Second International Conference on Geotextiles

Deuxième Congrès International des Géotextiles

AUGUST 1–6, 1982

1–6 AOUT, 1982

LAS VEGAS, NEVADA

U.S.A.

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COMPTES-RENDUS

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Volume IV (to be published January 1983) includes the following: the proceedings of the opening and closing sessions, the transcripts of formal questions and answers, an index of authors and paper topics, and errata.

Le volume IV (à paraître en janvier 1983) comprend: les comptes rendus des sessions d'ouverture et de clôture, les transcriptions des questions et réponses, les errata et un index des auteurs et titres de communications.

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Overcoming Psychological Hang-Ups is Biggest Drainage Challenge

Vaincre les préjugés est le problème prioritaire du drainage

Drainage systems, often incorporating synthetic textiles, are helping to make Civil Engineering projects safe from damaging actions of water. Great progress has been made in recent years, but several archaic and unrealistic negative beliefs and attitudes are hampering progress. Overcoming these "hang-ups" is a bigger challenge than designing good drainage systems. Upmost examples are the following: (a) The belief that drainage itself is not necessary, not practical, or too expensive, (b) The belief that nearly every drainage problem can be solved by the use of blends of sand and gravel containing moderate amounts of fines, and (c) A general reluctance on the part of specialists to try any new idea or product that does not have a long experience record. Until these psychological hang-ups can be overcome, the potential benefits of drainage and synthetic textile products in them cannot be fully realized. Meetings such as the 1st International Conference on the Use of Fabrics in Geotechnics and the present one can do a lot to open the eyes of people designing engineering works needing good drainage systems.

Les systèmes de drainage, qui comportent souvent des textiles synthétiques, contribuent à protéger les ouvrages de génie civil des effets destructeurs de l'eau. De grands progrès ont été faits ces dernières années, mais un certain nombre de préjugés archaïques, inexacts et négatifs font encore obstacle au progrès. Surmonter ces idées préconçues est une tâche plus considérable que de calculer de bons systèmes de drainage. Les exemples les plus criants sont les suivants: a) l'idée selon laquelle le drainage lui-même n'est pas nécessaire, pas efficace, ou trop cher; b) la notion selon laquelle tout problème de drainage ou presque peut être résolu au moyen de mélanges de sable et de gravier contenant une fraction modérée de fines; c) la réticence généralisée de la part des spécialistes à essayer toute idée nouvelle et tout produit qui n'a pas des références nombreuses et anciennes. Tant que ces préjugés ne seront pas éliminés, les avantages potentiels de l'emploi des textiles synthétiques dans le drainage ne pourront pas être pleinement reconnus. Des manifestations comme le 1er colloque international sur l'emploi des textiles en géotechnique et comme le présent congrès peuvent faire beaucoup pour ouvrir les yeux de ceux qui établissent les projets des ouvrages de génie civil où un bon drainage est nécessaire.

INTRODUCTION

Drainage, often with the aid of synthetic textiles, is doing a great deal to make Civil Engineering projects safe from detrimental actions of water. Significant progress has been made in the past few years, but a number of unfounded negative beliefs and attitudes (hang-ups) are greatly interfering with progress, with the result that many engineering works create unnecessary hazards to the public, and deteriorate prematurely from the effects of water. Several prime examples discussed in this paper are the following:

(1) The belief that drainage itself is not necessary, is not practical, or is too expensive is a prime hang-up. This attitude has resulted in widespread practices that eliminate drainage as a design consideration in some important areas of engineering. Two areas are discussed here: (a) One example is the thousands of miles of dikes and thousands of small dams which have been built without drains. All of these structures would be safer with good drains. Upgrading existing structures and providing drains in the ones built in the future would be of great benefit to people in virtually every part of the world. (b) The second example is the common practice of designing pavements as "strong" but undrained systems. In this area alone, a lack of drainage is causing premature damage that is costing taxpayers throughout the world countless billions of dollars a year.

(2) The belief that nearly every drainage problem can be solved by the use of drains constructed of blends of sand and gravel containing not more than 5% of fines (material finer than 0.074 mm (No. 200 sieve) is a hang-up of major proportions. This widespread fallacy is a major obstacle to the use of the textiles in engineering projects requiring drainage, as it implies that they are seldom needed. Single-layer (and even some multiple-layer) drains constructed with sand and gravel blends seldom have the conductivity (k x thickness) needed to accommodate all of the water entering drains and thus protect Civil Engineering works from damaging actions of water. Quantities of water needing to be removed by drains for engineering works should always be estimated by appropriate calculations with Darcy's law and flow nets, or other suitable methods. Such calculations nearly always show the need for a layer of open-graded (narrow size-range) aggregate in the conducting part of a drain. And this mandates the use of some kind of filter--either specially processed good quality aggregate or a suitable textile--to prevent clogging of the open-graded layer.

(3) A general reluctance on the part of specialists of all kinds to try any new idea or product that does not have a long track record has impeded progress in all fields of technical and medical work. It has kept the textiles out of many projects where they might have been of significant benefit. A coordinated educational program is needed to overcome negative attitudes about textiles in drainage systems.

FIRST HANG-UP:

The belief that drainage is not necessary, is not practical, or is too expensive.

Drains are routinely designed for large earth dams, concrete dams, drydocks, large retaining walls, basements for large buildings, and many other major Civil

Engineering works needing protection from water. But they are almost never provided for many small, "unimportant" structures or to remove surface water entering pavements. An unwillingness to even consider drainage systems for many facilities is a hang-up of major size that can be blamed for huge economic losses and the existence of greater hazards to the public than would exist if the majority of these works were well drained. Examples are:

(a) Levees and Small Dams. Any water-impounding dam or levee that is not provided with a drainage system is susceptible to the development of concentrations of seepage on the downstream slope and beneath the downstream toe, as shown in Fig. 1. Any such structure is

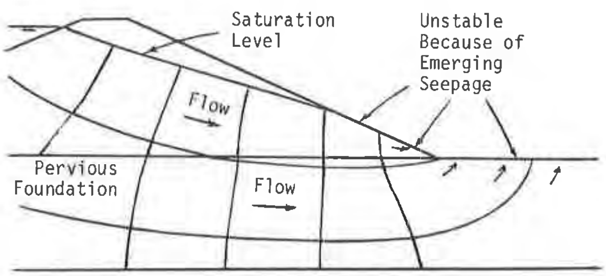

Fig. 1 Cross section through a typical dam or levee having no drainage system.

potentially likely to fail from seepage at some time. Although any dam or levee designer will probably admit that good drainage is a nice idea, he may say it isn't worth the cost for the countless small dams and thousands of miles of small levees around the world. Hopefully as more and more people become aware of the fact that the probability of failure of any dam or levee can be substantially reduced by a drainage system constructed at its landside toe, funds will be made available to upgrade the safety of many of these structures not having drains.

Dams and levees that have not been provided with good drainage systems can fail from seepage with little or no advance warning, because undermining ("piping") from seepage can be occurring without much external evidence. One such small irrigation dam in a Western State had been given its regular annual safety inspection and reported "safe for continued use". That night it failed by piping, culminating years of undermining by seepage that had gone unnoticed. Seepage exit areas that are not covered with good filters and drainage layers can loose significant amounts of materials that are washed away and not even noticed, as was the case with this project, until a break-through occurs and a rapid failure ensues. When all important seepage exits are protected with good filters and surcharged with clean drainage aggregate and gravel fill, failures of this kind can be virtually eliminated because the soil particles are trapped by the filters, and the piping actions are not allowed to start.

Figure 2(a) and (b) shows two applications of geotextiles in drains for levees and small dams. Figure 2(a) shows a substantial toe drain that controls seepage in both the dam and the foundation. A suitable synthetic cloth or filter fabric protects an open-graded drainage layer from clogging, and another fabric keeps dirt out of its upper sides. To insure adequate permeability the open-graded layer should contain no material finer than about 1 cm. size. A pipe at the bottom conducts the seepage to gravity outlets or to sumps for removal by pumps. Most of the volume of the trench can be any stable earth fill, suitably compacted. A drain of this kind can be used for upgrading the seepage safety of countless miles of levees, and thousands of small dams with seepage problems. It is also a good type for new dams or levees to be constructed on permeable foundations.

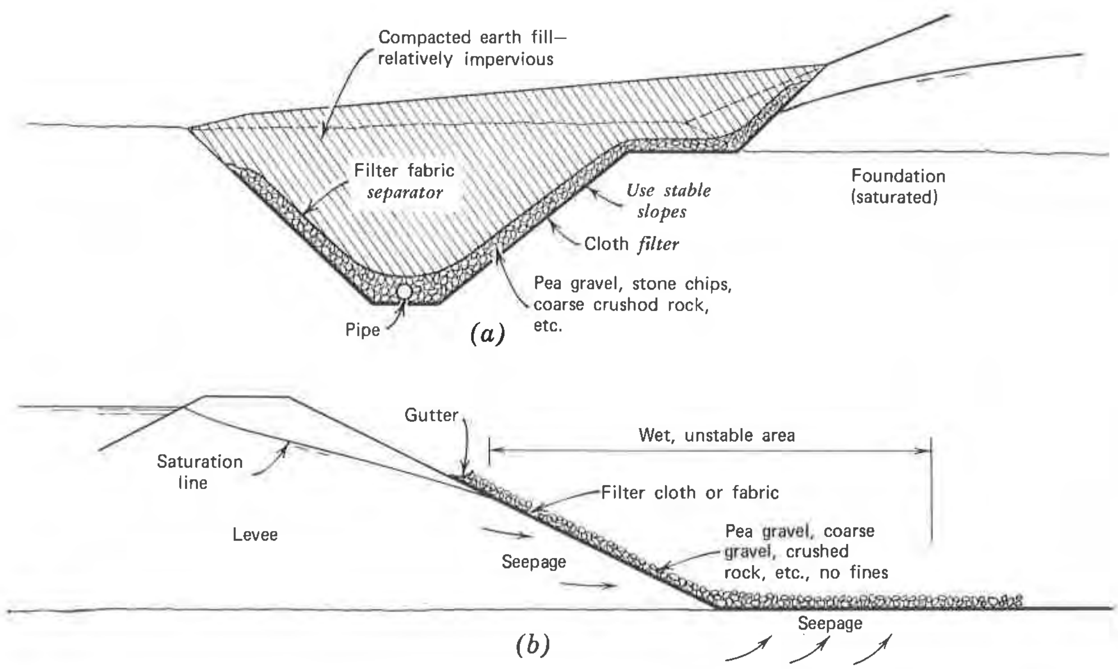

Fig. 2 Illustration of two potential uses for fabrics in drains for dams and levees. (a) A toe drain controlling seepage through dam and foundation, (b) A temporary measure for preventing imminent failure. (After Fig. 5.16 of "Seepage, Drainage, and Flow Nets," 2nd Ed., H. R. Cedergren, Copyright © 1977, John Wiley & Sons, Inc. N. Y. Reprinted by permission of John Wiley & Sons, Inc.)

Figure 2(b) shows a type of filter blanket that can be used to provide protection to levees and small dams with shallow seepage problems, such as minor sloughing, pin boils, and other shallow instability problems caused by seepage. If stockpiles of suitable porous aggregates and suitable fabric filters are kept on jobsites, and shallow seepage conditions should develop under high water stages, protective blankets of this kind can be quickly placed on troublesome areas. The fabric should have sufficient permeability to allow free flow of seepage into the aggregate layer which should be coarse gravel, crushed stone, railroad ballast, or comparable good-quality, highly permeable material. If additional weight is needed, any stable granular material can be placed over the drainage layer, provided a filter layer is placed where necessary to keep fine soils out of the coarse layer. This is not a recommended treatment for deep-seated instability problems, unless it is heavily ballasted.

Before the development of the synthetic fabrics, drains of the kinds shown in Fig. 2 would have been constructed as graded filters (1), with a fine aggregate filter layer being placed against the soil surfaces on which the drains were to be constructed, and one or more coarser layers to provide for water removal and weight (2), (3). Many water-impounding structures have been provided with drains constructed of durable, natural mineral grains; however it can be difficult to obtain filter aggregates fine enough to provide filter protection, yet permeable enough to freely remove all of the incoming water without the build-up of excessive head. Also, if the filter aggregates are placed on wet, soft ground, as for the levee in Fig. 2(b), the filter material tends to mingle with the wet soil, and its permeability is greatly reduced. Under such conditions, if a selected filter fabric is carefully rolled out over the surface needing protection, it serves as a separator, holding the soil in place, and preventing it from entering into the open-graded drainage layer and reducing its permeability.

The use of filter fabrics had a great deal of impetus in regions in which problems developed with sand and gravel filters that did not meet specification needs, and in cases where a lack of space made it difficult to place graded filters. In places where strong currents and heavy wave action would physically remove filter aggregates under large rock used for riprap and breakwaters, the synthetic fabrics have been particularly helpful. Barrett (4), Dunham and Barrett (5), and other workers describe early shore-erosion protection structures such as stone seawalls and jetties that used the fabrics to hold fine soils in place and thus prevent undermining of these works. Seemel (6) presents a good summary of the development and use of the fabrics. Numerous papers in the First International Conference on the Use of Fabrics in Geotechnics describe usages of fabrics in projects around the world.

In both of the examples given in Fig. 2, the drains are in exterior parts of the structures, and are accessible for removal and replacement if problems should develop over the life of a project.

If the belief that drains are not needed in many small structures such as levees and small dams can be overcome, drainage systems--often with the incorporation of fabrics-can be of great benefit in upgrading the seepage safety of countless structures around the world. An outstanding example of the use of fabrics to help upgrade seepage safety of existing structures is the dike for the Florida Power & Light Company's cooling water reservoir in Florida, where 1.5 million square yards (1,250,000 m^2) of nonwoven fabric was used in constructing a drain of the general type shown in Fig. 2(a) (7).

(b) Pavements. The modern belief of most pavement designers that internal drainage is not needed is a psychological hang-up of gigantic proportions, and one that defies explanation, as it cannot be justified on any engineering or economic basis. Because of it pavements deteriorate 3 to 4 times faster (annually) than if they were well drained.

Historically, road builders have believed in good drainage; however hardly any modern road designers do. As a consequence, nearly all of the important pavements that have been built in the past several decades have failure mechanisms built right into them. During the periods of time that structural sections are filled with water, the rates of damage (per traffic impact) can be hundreds to thousands of times greater than when there is little or no free water to be acted upon by traffic and climate. For centuries road builders have known that coping with the water that gets into pavements and the soils under them is the biggest obstacle to having long-lasting, trouble-free pavements. Even the Ancient Romans built their famous road system above the surrounding terrain to help eliminate water damage. In 1820 John L. McAdam (8) said that ". . . if water pass through a road and fill up the native soil, the road whatever may be its thickness loses support and goes to pieces." And, "The erroneous opinion . . . that (by) placing a large quantity of stone under the roads, a remedy will be found for the sinking into wet clay or other soft soils . . . (so) that a road may be made sufficiently strong artificially to carry heavy carriages though the subsoil be in a wet state . . . has produced most of the defects of the roads of Great Britain." His complaint is valid today.

Shortly after enactment of the U.S. Federal Aid Act of 1916, pavements were designed on the basis of the Soil Classification (A-1, A-2, etc.) and a designers experience and judgment. With the advent of modern Soil Mechanics methods, pavement designs have been based almost entirely on strength factors obtained by making tests on saturated samples of base and subgrade materials. Designers have tended to believe that these methods guarantee that any and all problems with water are automatically eliminated. Build pavements sufficiently "stout" and there is no need for drainage, is the idea. Since the loads applied in the tests are generally "static" and traffic impacts are "dynamic" one might expect shortcomings in the designs using these methods. Much of the damage to pavements is caused by the pore pressures and actions in the water impacted by heavy vehicles; other severe damage is caused by climatic actions on the trapped water, such as freezing, "D"-cracking, blow-up, shrinkage cracking, premature oxidation, and the break-out of chunks of pavement to create the well-known "pot holes". Most of these actions do not occur at all in well drained pavements.

In the period from about 1950 to 1962 several highly instrumented and documented "road tests" were made to determine what makes pavements break up and what can be done about it. The primary (but almost totally ignored) finding in these tests was that during periods when free water was trapped in the test pavements, each traffic impact produced up to hundreds and thousands of times more damage than when there was no free water in their sections. In the WASHO road test (9), damage rates were up to 70,000 times greater (per impact) with free water than with no free water present; in the AASHO road test (10), the wet damage rates were 10 to 40 times greater than the dry; in the Univ. of Ill. Circular Test Track experiments (11) they were 100 to 200 times greater with free water than without.

Those planning the Road Tests were thinking only in terms of finding the strongest combinations of pavement and base materials to resist damage, not at all in terms of eliminating the free water with good drainage. As a consequence, not a single one of the hundreds of combinations tested was well drained! And so the prevailing practice of pavement designers continued to be to design pavements "stout", but not even think of drainage as a viable design concept.

Confidence in the "un-drainage" philosophy has been so high that many designers look with disfavor on anyone who dares to question this approach, and deeply resent any suggestion that drainage is a better concept. As part of the work carried out to develop the FHWA's "Guidelines for the Design of Subsurface Drainage Systems for Pavement Structural Sections," (12) field interviews were conducted with State Highway engineers throughout the U.S. in the 1971-72 period. Comments made by persons interviewed probably represent a good cross-section of the opinions of pavement designers everywhere. One of those interviewed said, "I have nothing but contempt for anyone who thinks pavements can be drained." In a major Western state, a top pavement designer said, "But, of course, it is neither necessary, practical, nor economical to drain pavements." In all of these states, pavements were breaking up prematurely from traffic and undrained water.

An engineer in one state interviewed said, "Anyone who thinks pavements can be drained is a fool." He said they had tried drainage and it just doesn't work. They had meticulously compared hundreds of miles of "drained" pavements with many miles of similar, but "undrained" pavements and couldn't detect any difference in performance. On inquiry, we found that a "drained" pavement was a stretch of road (on their standard low-permeability sand base) with a narrow cross drain every 300 to 400 feet (91 to 122 m) with a drain pipe in a trench backfilled with concrete sand. Such a drain could not have had much influence in draining water out of more than a strip 2 or 3 feet (0.6 to 0.9 m) wide above each one of the drains; therefore there should have been no noticeable difference between "drained" and "undrained" pavements. Yet, these engineers were so firm in their convictions they were almost willing to come to blows with anyone disagreeing with them. What a hang-up they had!

In 1977 the U.S. Department of Transportation (DOT) released a report of a major study of the condition of our Nation's highways (13). That report said that out of a $450 billion total outlay expected for American roads between 1976 and 1990, $329 billion will be needed just to keep our roads in their 1975 condition. Applying information I have gathered in investigations of deteriorating highway and airfield pavements across the United States (12), (14), I estimate that the modern belief that pavements don't need to be drained can be blamed for about 2/3 of the repair and replacement costs facing our nation, which represents an unnecessary and avoidable loss of at least $15 billion a year (15) to American taxpayers, and on a world-wide basis, at least a trillion dollars over a 40 year period (16). Even though it is not possible to pin-point the exact losses caused by this hang-up, it must be evident that they represent a severe drain on already overburdened taxpayers.

Taxpayers, public officials, and members of the public media have become alarmed at the rapid deterioration of our "Magnificant Pavement System" that was supposed to represent the best thinking and modern technology, and consumed vast sums of materials, energy, and money. With over 116 million potholes jarring American drivers, and their cars and trucks, an emergency pothole filling bill by Congress was decried as "A Poor Choice of Patchwork" by Engineering News-Record (17). A U.S. News & World Report feature (18) says that Congress' increase in the allowable loads on federally-aided roads in 1974 from 73,280-lb. to 80,000-lb. was an "unwise" act by our legislators. It says that American roads--the most expensive public works undertaking of all time--are being battered to pieces. Numerous other national publications have expressed concern over the deteriorating pavements. Though the increase in loads voted by Congress has been a large factor in the accelerated damage to our undrained system, it would have had a great deal less impact had these pavements been well drained.

Many things contribute to pavement failure; however water is by far the greatest contributing factor (other than traffic impacts). The antiquated "un-drainage" philosophy is a hang-up that must be overcome, before significant improvement in the pavement deterioration dilema can be expected.

SECOND HANG-UP:

The belief that blends of sand and gravel containing a few percent of fines are suitable for all drainage needs.

This ill-conceived belief is a hang-up of major proportions, and one that is an obstacle to the use of the fabrics in drains as it implies they are seldom needed.

Human actions are often a matter of simply following a "popular" practice, no matter how rational or how irrational it may be. Popular practices, like a pendulum, often swing from one extreme to another. Beliefs about what kinds of aggregates are good drainage materials have gone through pendulum-like cycles over the years. After about 1750 there was a period when designers of roads and other engineering works believed that the coarse rock and boulders employed in "French Drains" were ideal materials. Under favorable conditions these drains often served their purpose, at least in part. But, when the large head-size boulders and rocks were placed in trench drains in erodible silts, fine sands, and the like, the drains often became clogged the first time the adjacent soils became saturated. So this kind of material fell in bad repute because of the piping and clogging it produced.

As engineers became concerned over the need to prevent piping, there developed a tendency to use blends of sand and gravel in drains without determining if the blends were permeable enough to remove the water without excessive build-up of head in the drains. They began to believe that concrete sand and other materials containing less than about 5% of fines (silt and clay) were satisfactory for virtually every drainage need. This widespread belief (hang-up) has led to the design and construction of many dams, levees, roads, and other Civil Engineering works that are poorly drained. It persists in spite of world-wide experience proving that this belief belongs in the fairy-tale world.

If errors in thinking about drainage requirements are to be eliminated, we must not forget that every seepage and drainage situation follows specific laws of Nature, and depends on physical factors such as coefficient of permeability, hydraulic gradient, and area of cross-section in which water is flowing. Seepage and drainage are quantitative problems, not qualitative. Each problem has its own specific solution. To ensure that drains will be able to remove the water reaching them, the inflows must be estimated with seepage principles and the required permeabilities and dimensions of drainage layers must also be calculated using the same fundamentals (3),(19), and (20).

Even the simplest "thumb-nail" calculation that uses reasonable values for permeabilities, gradients, and dimensions will almost always show the need for a layer of highly permeable, open-graded aggregate in drains for Civil Engineering works. And this dictates the use of filters--either special aggregates meeting accepted filter criteria, or suitable textiles--to prevent piping and clogging actions, together with a layer or zone of highly permeable aggregate that removes the water.

The filter requirements of drains have been described in many books and publications (21), (22), (23), etc., and the desired properties of fabric filters are discussed in recent publications (24), (25), etc.. Although the discharge needs of drains have had hardly any attention at all until recently, the principles that can be used in making these determinations (largely Darcy's law and flow

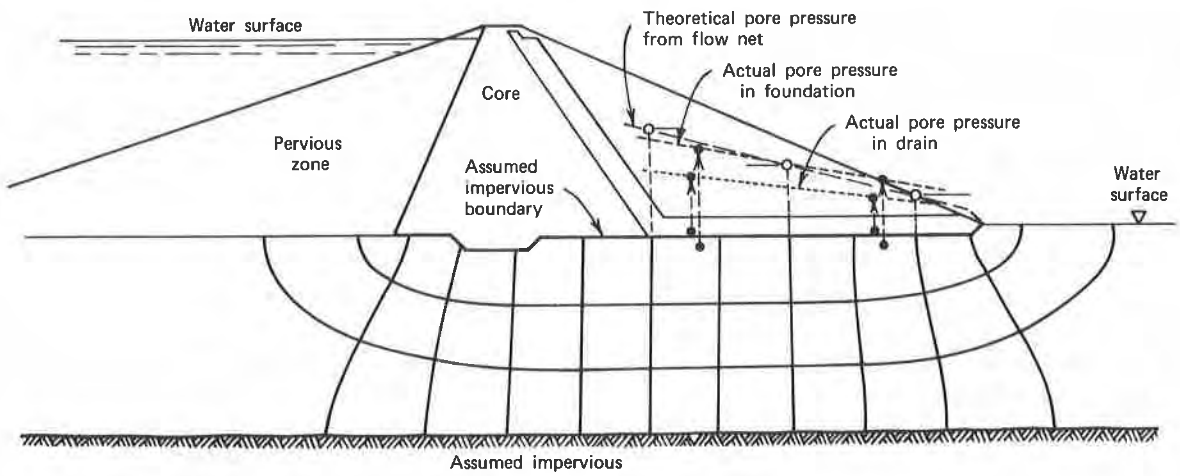

Fig. 3 Uplift pressures that built up under an earth dam with an expensive but ineffective drain were not measurably influenced by the drain (From Embankment-Dam Engineering, Casagrande Volume; 2nd Chapter, by H. R. Cedergren, "Seepage Control in Earth Dams." Copyright © 1973, John Wiley & Sons, Inc. New York. Reprinted by permission of John Wiley & Sons.

nets) have been described in many publications over the past 40 years or so (26), (27), etc.

The dam illustrated in Fig. 3 epitomizes the extremes to which the sand and gravel blend idea has been used in drains. Here, a 60 m high earth dam was built in 1965 in a Western state of the U.S. Designed and built by a major dam design firm, it has a very costly and complex drain system that provided no discernible benefits in controlling uplift pressures under the dam. All drain zones were constructed in three layers, with outer "fine filters" on both sides of an inner "coarse filter". All zones were allowed to contain up to 5% of fines, on the presumption that this amount of fines is satisfactory in drains. After water had stood in the reservoir for several months near the position shown, pore pressures in the drain and in the foundation built up to the levels shown. In order to better understand the problem I constructed the flow net shown, on the assumption that the drain was completely impervious (accepting no foundation seepage). Piezometric heads obtained from the flow net are also shown in Fig. 3. It can be seen that they agree almost exactly with those measured, showing that the drain in fact has no effect on uplift pressures. This dam is a remarkable illustration of the invalidity of the premise that blended aggregates containing not more than about 5% of fines are suitable drainage aggregates.

A great many of the dams I have been asked to investigate because of seepage problems, have had problems caused because drains were either not used, or those that were contained too many fines and did not have the levels of permeability needed to remove the water without excessive head build-up. Only when the discharge needs of drains are properly estimated, and the drains are designed to carry these amounts of water can such deficiencies be eliminated.

Not being willing to analyze drains as conveyors of water, while being willing to follow a misguided concept is a major psychological hang-up that needs to be combated by extensive educational and promotional programs.

THIRD HANG-UP:

The unwillingness to try a new material or idea that does not have a long track record.

Progress in all areas of technology (and medecine) has been hampered by this kind of hang-up. Ideas that happen to be popular remain in vogue for years while eminently superior ideas remain untried, largely because of psychological hang-ups and a "fear of the unknown."

Specialists in all fields have a reluctance to try new ideas that have not been accepted by their peers. A person who follows a practice used by his predecessors or fellow workers (no matter how bad) is generally not blamed if something goes wrong. But let him try something new or innovative (no matter how fundamentally superior it may be) and if problems develop he is usually called "reckless", "irresponsible", or at the minimum "careless". There is often a tendency to be extremely critical about something new and to look "through rose-colored glasses" at the conventional, no matter how poor its record.

This kind of behavior has gone on throughout recorded history. When Galileo furnished evidence that proved the earth revolves around the sun and is not the center of the universe as was believed in his time, he was forced to spend the last years of his life under house arrest after being tried by the Inquisition in Rome for suggesting such a radical idea. Engineers, doctors, and other specialists have stubbornly ignored new ideas that later proved vastly superior to ideas that had been popular at a given time. The potential benefits of open-graded aggregates and synthetic fabrics in drains for engineering works are not being fully realized because of mental or psychological hang-ups such as are discussed in this paper.

Ideas eminently superior to prevailing ideas and practices have been rejected, even ridiculed. When Dr. Simmeliveis in Vienna suggested in 1880 that simple sanitation (washing hands, etc.) could reduce Streptococal infections he was scoffed at and not allowed to operate. Not until 1920 were his cleanliness ideas accepted by the medical profession. In about 1780, Dr. Benjamin Rush of Philadelphia said Cholera and Typhoid were spread by contaminated well water and bottled water. He was ridiculed and his idea lay unaccepted for more than 150 years! Even in the 1920's many children were dying from infantile diarrhea because of inaction and refusal to accept a concept that was very sound. Examples of this kind of human behavior can be found in all major fields.

Why do experts in all fields resist new ideas? Largely because it is "safe" to stay with an accepted procedure--no matter how poor its record. "Don't rock the boat" is an expression that aptly expresses the general attitude about making changes or trying something new. It is very hard to overcome.

One of the problems that needs to be overcome with the synthetic fabrics is the fact that people generally have come to look upon the synthetic products as being rather fragile and of limited life expectancy. Many of the synthetic products such as water hoses and black plastic sheeting become brittle and badly deteriorated in just a few years. Even though much of the deterioration is caused by exposure to sun light, and the fabrics in engineering will be protected from such exposure, there is a tendency to look upon the fabrics skeptically in relation to long-time performance.

Engineers in decision-making administrative positions sometimes don't keep themselves informed about new products or ideas, and reject an idea simply because it is new to them. In 1974 I reviewed a near-failure of a dam in California, caused by an ineffective internal drainage system containing aggregates with 6% of fines. Seepage had caused the near collapse of this dam when its reservoir was quickly filled. I recommended a new toe drain of the general design shown in Fig. 2(a), and such a drain was designed. Just before the opening of bids, an engineer with an agency having control over part of the funds said he would not permit any plastic materials to be used. A redesigned drain with vertical walls was subsequently built with no fabrics used. Severe caving problems had to be fought and the redesigned drain cost substantially more than my original design with fabrics. Unfortunately, this kind of reactionary attitude is altogether too common. It is a handicap to progress.

SUMMARY COMMENTS:

Only part of the potential benefits of good drainage systems and good drainage products is being realized because of some unrealistic and unfounded beliefs (hang-ups). Examples of three major areas where psychological hang-ups are impeding progress as discussed in this paper are:

(1) The belief that drainage itself is unnecessary, impractical, or too costly,

(2) The belief that aggregate blends of sand and gravel materials that provide good filter protection will automatically provide good drainage, and

(3) A general reluctance to try any new idea or product before it has a long experience record.

Collectively these misguided concepts are restricting progress in drainage and the use of new products in drainage systems. Major educational and promotional programs are needed to overcome these hang-ups.

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Performance of Geotextiles in Stabilization of Clay Slopes in Italy

Comportement des géotextiles dans la stabilization de pentes argileuses en Italie

The use of geotextiles in drainage and landslide correction works has been increasing in Italy since the last ten years. A concise description of constructive schemes and of site arrangement for draining trenches is preliminarly reported. Then, the drainage system designed for correcting a landslide in clayey soils, adopting a polyester non woven fabric, is described. The effectiveness was controlled after 30 months: the geotextile, the surrounding soil and the draining gravelly and sandy material were sampled and tested in the laboratory; results of permeability and tear tests, both on contaminated and on "virgin" fabric, are reported. The functioning of the soil/fabric double layer as a filter and the satisfactory maintenance of geotextile properties were assessed.

L'emploi des géotextiles dans le drainage et dans la stabilisation des éboulements a crû en Italie depuis une dizaine d'années. Pour les tranchées drainantes, on rapporte préliminairement une concise description des schèmes de construction aussi que du leur arrangement en campagne. De suite, on décrit un drainage construit pour la stabilisation d'un éboulement en argiles, avec l'emploi d'un géotextile non-tissé en fibre polyester. Le comportement de l'ouvrage a été contrôlé 30 mois après: le géotextile, le sol environnant et la sable et gravier du drainage ont été échantillonés et soumis à essais en laboratoire; on rapporte les résultats des essais de permeabilité et de déchirure sur des échantillons de géotextile, soit vieux, soit nouveaux. Le fonctionnement de la double couche sol/géotextile tel que un filtre et la conservation satisfaisante des proprietés du géotextile ont été confirmés.

1 INTRODUCTION

The use of geotextiles in drainage works, specifically for the stabilization of landslides or unstable slopes, has been increasing in Italy since the last ten years. At present, geotextiles are currently employed as a protection against contamination ond occlusion of the water-bearing material, in partial substitution of traditional graded filters, made of purposely selected natural soils: the main advantages are safety and economy, thanks to a shorter time that is necessary for their setting in place.

In this paper, a summary of the evolution of drainage technology and a brief description of constructive schemes and modalities for draining trenches and of their arrangement for slope stabilization are preliminarly reported. Successively, a case history concerning the correction of a landslide crossed by an important oil pipeline in Northern Italy is told: an exploration pit was excavated after 30 months since the construction of a draining trench; the surrounding natural soil, the contaminated geotextile and the water-bearing material were closely examined, sampled and tested in the laboratory. The functioning of the self-established double layer "geotextile / adjacent soil" as a filter and the satisfactory maintenance of the main geotextile properties were assessed.

2 DRAINAGE TECHNOLOGY IN ITALY

The following considerations are restricted to drainage trenches; other deep draining structures, such as galleries, pits, vertical drains or sub-horizontal pipes, are out of the scope of this paper.

2.1 Evolution of Drainage Technology

The progress achieved in design and construction of drainage trenches in Italy is synthesized in fig. 1. The oldest drains (since 19th Century to 2nd World War) consisted of shallow trenches, filled with coarse gravel and cobbles, without sand (fig. 1 a); the use of such drains, also provided with a bottom collector pipe (fig. 1 b), but designed without following any filter design criterium, continued to about 1950 ÷ 1955.

In order to avoid a too rapid clogging and loss of efficiency, the well known criteria for selection of filter material (1) were introduced; graded filters in natural soils (clean sand with gravel in the main part of the drain, fine uniform gravel around the collector pipe) began to be adopted; besides, the filetr material was protected against surface water by backfilling the upper part of trenches with clay (fig. 1 c).

In many cases, such drains are provided with a concave concrete slab at the bottom (fig. 1 d), stepped for high slope of the sliding surface (2).

During the last ten years, these traditional drains have become more and more expensive, owing to the increasing costs of natural filter materials; on the contrary, the use of geotextiles in substitution of natural sand removes the uncertainties related to grain size composition of the sand itself and reduces the construction times. Therefore, modern trenches with geotextiles (fig. 1 e) result technically more effective and economically advantageous, in comparison with traditional trenches with graded filters in natural soils.

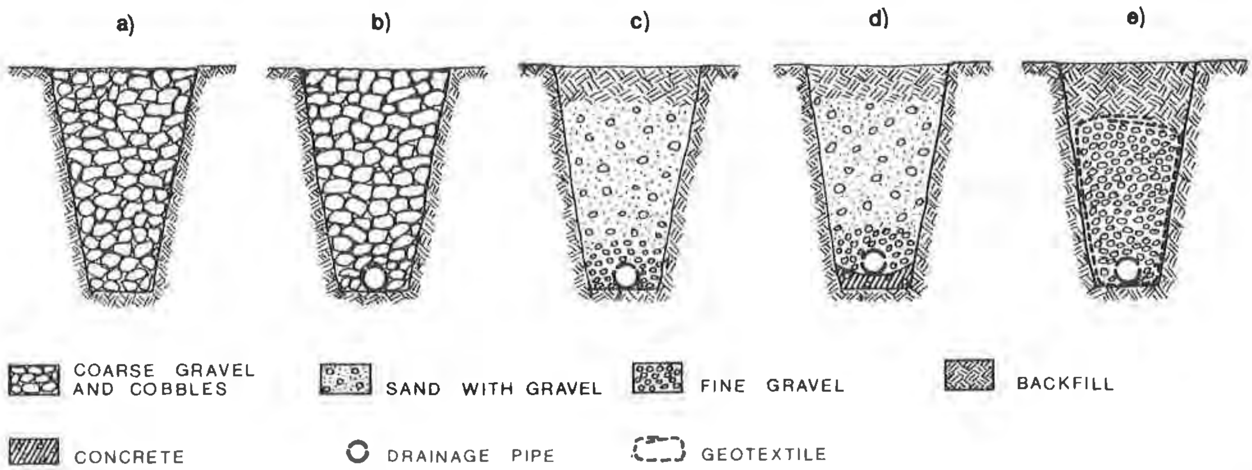

Fig. 1 Evolution of drainage trenches in Italy (transversal sections): a) - b) obsolete drains; c) - d) drains with graded filter in natural soil; e) modern drain with geotextile.

2.2 Current Drainage Technology

According to the typical transversal section (fig. 1 e), draining trenches generally include the following basic components:

- coarse granular soil, as hydraulically pervious material, in the main part of the trench;
- geotextile sheet, as filter around the draining body;
- collector pipe at the base of the trench;
- surface backfill with impervious soil (clay), in order to prevent the trench against surface water and against agricultural works (minimum thickness = 1 m).

As for variability of basic components:

- many kinds of non-woven fabrics, with a preference for polyester fibers, are employed (alternatively, needle-felt or spun-bonded), the mass per unit area varying from 300 to 500 g/m^2;
- the grain size composition of draining materials ranges from coarse gravel to coarse and medium sand;
- reinforced concrete, steel or PVC, perforated pipes are employed to collect the drained water (diameter varying from 200 to 300 mm).

Several criteria for geotextile selection can be found in the literature (3), depending on the opening size of the geotextile itself (O_{90} or O_{98}) and on some grain size characteristics of the adjacent soil (diameters d_{85} or d_{50}, coefficient of uniformity $U = d_{60}/d_{10}$). Nevertheless, natural soils are generally so disuniform and heterogeneous, that such criteria can be applied only with rough approximations.

Longitudinal dimension, depth and planimetrical arrangement of drainage trenches depend on geometrical and hydrogeological characteristics of the landslide to be stabilized. Lengths up to 100 m and depths of about 6÷8 m can be attained; as a general rule, these draining structures are disposed parallel to the direction of maximum slope, so that they cannot be damaged by subsequent slope movements.

3 A CASE HISTORY

In order to control the performance of geotextiles in landslide correction, a slope movement near the small town of S. Cristoforo (Alessandria District - Northern Italy) was selected.

The hillside was crossed by a pre-existing oil pipeline (from Genoa to the River Po Basin - diameter d = 800 mm), whose economical importance justified detailed investigations and landslide corrective works.

3.1 Description of the Landslide

The investigated area lies on the northern side of a hill, constituted by overconsolidated, fissured, marine clays (Lower Pliocene).

The geometrical parameters of the sliding mass are:

- angle of slope to horizontal $\beta = 8°$ (mean value);
- maximum length L = 200 m;
- maximum breadth B = 230 m;
- maximum thickness D = 9 m (as resulting by borings and by the subsequent excavation of drainage trenches).

The following representative lithological sequence was recognized:

a) 0 ÷ 5 m: reddish brown clayey silt, with traces of sand and elements of gravel (colluvial soil);

b) 5 ÷ 8 m: light brown silt with clay and traces of sand (in-situ weathered zone of the marine formation);

c) > 8 m: grey-blue silt with clay (unweathered marine formation).

In general, the deepest slip surface passed along the contact between soils (b) and (c), but locally entered in the grey unweathered soil (involved in the sliding mass by a softening process).

This landslide can be classified as a periodical mass movement, whose stability is governed by the residual angle of shearing strength ϕ'_r (11 ÷ 13°) and by the height of water level above the slip surface.

The piezometric level, measured during the last movement of the slide (spring 1979), was only 2 ÷ 3 m deep in the main part of the area; besides, some ponds of water could be observed in the upper part of the landslide. Therefore, it was outstanding that the slope stability could be increased by appropriate drainage works.

3.2 Drainage System

As the pipeline trend was close to the axis of the sliding area, the following draining system was proposed:

a) a central draining trench, near and parallel to the pipeline itself, 200 m long;

b) two lateral drains, each about 50 m long (not considered in this paper).

For the former and most important drain, the use of geotextiles was proposed, in order to reduce the construction times and the consequent risks for the pipeline. The (uncommon) transversal section of this drain is represented in fig. 2.

In order to avoid clogging of the geotextile, and bearing in mind that soils to be drained were constituted in prevalence by silt and clay fractions, a fabric characterized by low values of the "filtration diameter" (4) was selected: its commercial name is "Terbond TF" and it is produced in Italy by ANIC-Fibre.

According to well established classification criteria (5), Terbond TF 500 can be described as follows: non-woven felt; mechanically matted; polyester fibers (density ρ_f = 1380 kg/m^3); mass per unit area μ = 500 g/m^2; thickness T_g equal to 4.0 and 1.9 mm, respectively under 2 and 200 kPa.

As for concerns hydraulic properties, the coefficient of permeability (both normal and planar) is of the order of 10^{-3} m/s.

The filtration diameter was determined on the basis of tests formerly suggested by Fayoux (4), slightly modified by ANIC-Fibre Laboratories (6). The results are reported in fig. 3: the filtration diameter, defined as the dimension corresponding to d_{98} for the soil passed through the tested geotextile, resulted about 60 µm.

For the draining body, the most appropriate material (fine to medium gravel) was available only in small amount and a compromise between technical and economical exigencies became necessary. Uniform gravel (d = 10 ÷ 30 mm) was put in place only around the collector pipe; the main part of the trench was filled in with alluvial, clean sand with gravel (fig. 2): this natural material was economically available from the bed of River Lemme, close to the sliding hillside.

The grain size characteristics of the natural soils and of the draining materials are reported in fig. 4. The coefficient of permeability of the alluvial sand, determined by falling head permeameter, resulted ranging from

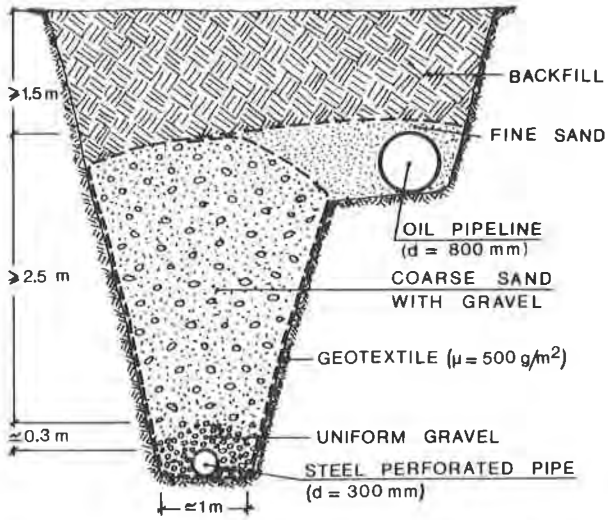

Fig. 2 Transversal section of the drainage trench.

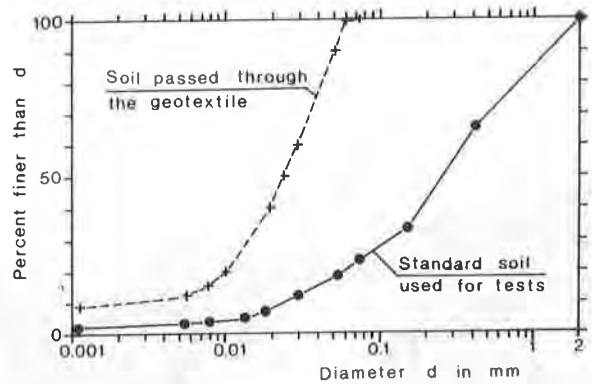

Fig. 3 Results of the test for determining the filtration diameter of the geotextile.

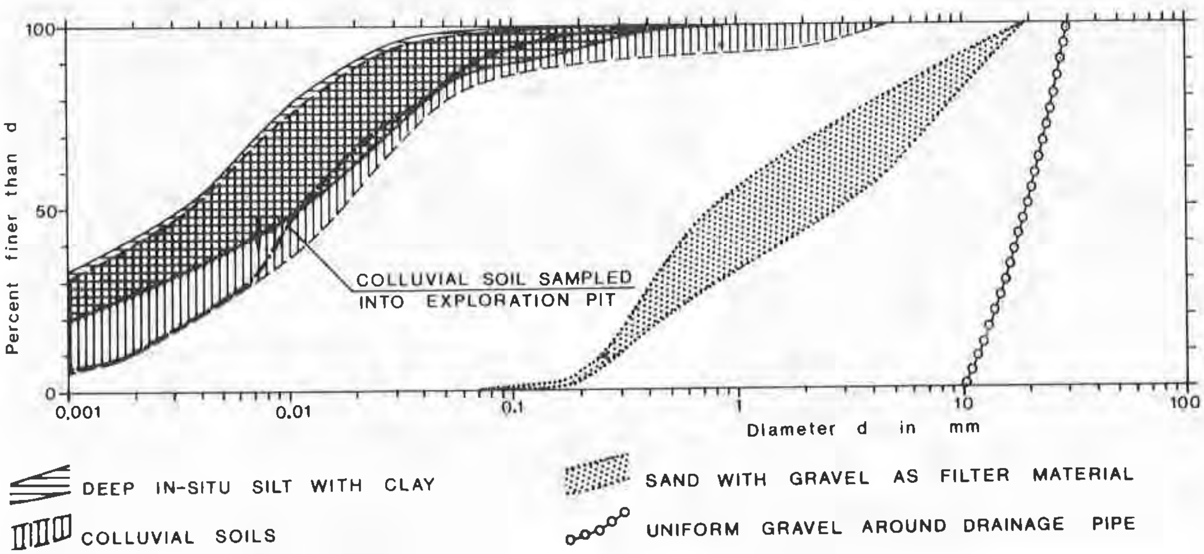

Fig. 4 Grain size characteristics of the natural soils and of the draining materials.

1 to 2 . 10^{-3} m/s; for the uniform gravel, k can be computed according to Allen Hazen's formula.

The construction was completed within July, 1979, according to the following procedure:

a) general excavation until the pipeline base level, in order to allow small movements and detensioning of the structure;

b) partial excavation until the bottom level of the drain (each portion no longer than 10 m, to reduce risks for the pipeline);

c) filling in the excavated portion and excavation of the next one (see photo - fig. 5);

d) covering the pipeline with protective fine sand and general backfilling.

3.3 In-situ Control

The effectiveness of the draining system on the slope stability was ascertained during the years 1979÷1981, when the hillside underwent long rainy periods, without any movement. However, a control pit was excavated in February, 1982, in order to control the conditions of the geotextile and of the adjacent soil after about 30 months since construction.

The exploration pit, situated in the lower part of the landslide, was excavated (initially by backhoe, then by hand operator), to a depth of about 3.5 m; the upper backfill, the geotextile, the underlying draining material and the lateral original colluvial soil were put in evidence (see photo - fig. 6).

The water table was not achieved within the explored depth (3.5 m); this fact finds his explanation partly in a satisfactory performance of the drainage system as a whole, partly in the scarceness of precipitations during last winter 1981/82. However, is was clear that the trench had really acted as a drain; a thin layer (about 1 mm thick), formed by silt-sand particles vould be observed on the external surface of geotextile; many pores of the fabric itself appeared blocked and clogged; besides, a blackish layer, about 20 mm thick, could be seen on the drainage sand (originally grey), at the sand/fabric interface.

The polyester fabric was found apparently unaltered and not attacked by roots or by animals.

3.4 Laboratory Tests

The external colluvial soil, the geotextile and the internal sand were sampled for laboratory tests.

Fig. 5 View of the drainage trench along the pipeline during construction.

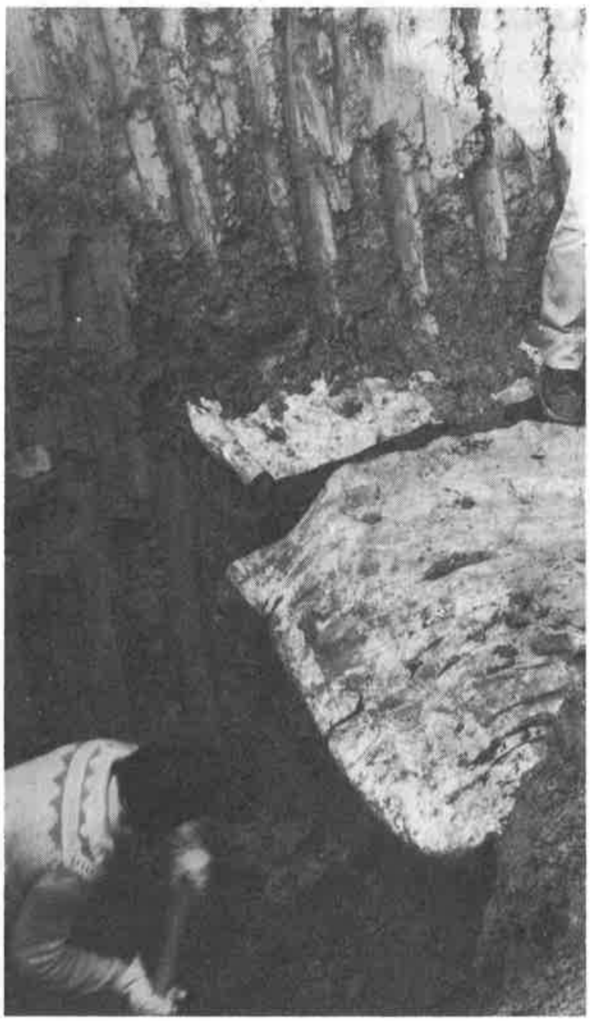

Fig. 6 Excavating the exploration pit.

The colluvial soil, sampled on the trench side opposite to pipeline, was constituted essentially by silt fraction (about 80 %), with minor amounts of clay and sand. The grain size curve of this sample is reported in fig. 4, in comparison to the envelope of all the granulometrical curves that had been previously determined for colluvial soils; in particular, the percentage of soil retained to 0.074 mm sieve was 8.6 %.

A block of the same colluvial soil was taken at the contact with fabric. A small amount of soil was scraped by a knife from the contact surface and a sieve analysis carried out: the percentage of soil retained to 0.074 mm sieve resulted 10.2, gaving a proof of relative enrichment in coarser particles in proximity of the upstream geotextile surface.

As for regards sand with gravel forming the draining body, laboratory tests took into consideration the apparent differences between the blackish outer layer and the inner grey mass. On the basis of qualitative analysis by immersion for 24 hours in 3 % solution of NaOH, the black layer resulted to contain a remarkable amount of organic substances (likely of vegetal origin). Owing to natural variability of alluvial material, and in order to distinguish eventual differences due to the presence of geotextile, granulometrical analyses were performed only on fractions passing to 2 mm sieve. Under such limitation, sieve analyses gave the following results:

- black layer: 7.1 % passing to 0.074 mm sieve;
- grey sand: 3.2 % passing to 0.074 mm sieve.

The relative enrichment in finer particles in proximity of the downstream geotextile surface resulted evident.

Finally, laboratory tests were carried out on fabric samples, contaminated after 30 months since their burying, and, for comparison, on "virgin" geotextile samples.

The coefficient of permeability was measured both normally (k_n), both in the plane of geotextile (k_p), under normal pressures ranging from 50 up to 218 kPa. The results are summarized in fig. 7: the measured values are conform to the values declared by the Productor and the decrease of k_n (or k_p) due to contamination appears very limited.

The fabric resistance was controlled by trapezoid tear test (ASTM D2263). The breaking force F_T was measured respectively on virgin and on aged samples, both longitudinally (L) and transversally (T). The measured values are summarized in table 1.

Table 1. Results of trapezoid tear tests (ASTM D2263).

Breaking force	virgin fabric	aged fabric
$F_{T(L)}$	850 N	700 N
$F_{T(T)}$	600 N	470 N

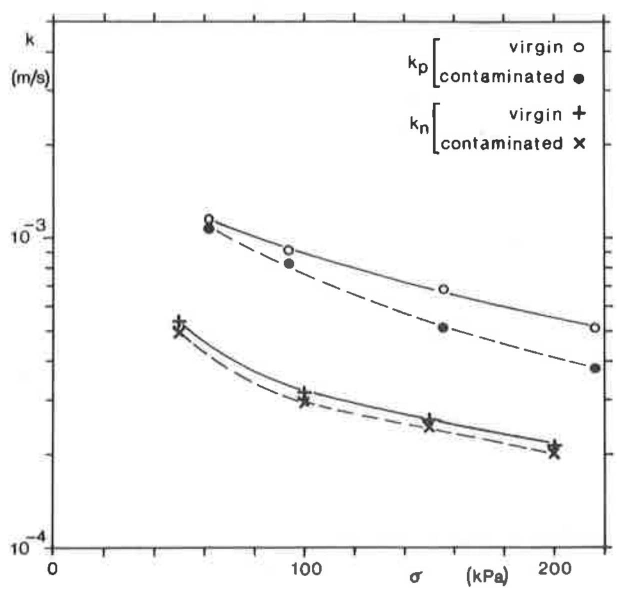

Fig. 7 Coefficient of permeability k versus normal pressure σ, for "virgin" and contaminated samples (k_p planar, k_n normal to geotextile).

The diminution with reference to initial values can be partly ascribed to variability of production; however, such a loss of resistance (about 20 %) is tolerable for practical purposes and is in agreement with results published for other polyester geotextiles (7).

4 COMMENTS AND CONCLUSIONS

From direct inspection of soil/fabric interface and from grain size analyses on sampled soils, the self-establishing of a thin layer (less than 3 mm), impoverished of fine particles, adjacent to the upstream face of geotextile, was proved. Along the downstream face of geotextile, the draining sand was found enriched in fine particles: the thickness of this partially clogged layer was found to be about 20 mm. This investigation strengthened that the so-modified assembly "soil/geotextile/sand" can act as a graded filter, and after a short time can reduce and stop any further piping of fine particles.

Accordingly, from the case history previously described it was also clear that the particular polyester fibre non-woven geotextile named Terbond TF 500 had supported more than 2 years of burying, with tolerable losses of the hydraulic and mechanical properties.

For future investigations, the following two topics can be proposed:

- soil/fabric shear angle and its eventual evolution with time;
- influence of soil water chemism on hydraulic and mechanical behaviour of the assembly soil/fabric/drain.

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The Use of Geotextile Fabrics in Pond Construction Beneath an Impermeable Membrane (Geomembrane)

Utilisation de géotextiles sous une membrane imperméable dans la construction des bassins

Geotextiles are commonly being used as underlining for geomembranes in pond construction. The geotextile provides puncture protection, gas release and abrasion resistance. This paper describes field usage of geotextile/geomembrane systems and outlines field experiments conducted to define a geotextile selection process. Laboratory tests were developed and are described that proved capable of readily defining pass/fail criteria for various combinations of geotextile/geomembrane/soil/load conditions. It was found that thick, needlepunched, nonwoven fabrics in a weight range of 400-600 g/m^2 provide the optimum combination of strength, durability and lateral transmissivity to perform satisfactorily as an underlining in large pond construction.

On utilise couramment les géotextiles comme doublures de fond de géomembranes dans la construction de bassins. Les géotextiles protègent contre les perforations, l'échappement de gaz et les frottements. Ce rapport décrit comment les mécanismes des géotextiles/géomembranes sont utilisés sur les chantiers, et trace les grandes lignes des expérimentations menées en vue de définir un processus de sélection de géotextiles. Il est décrit ici les essais en laboratoire que l'on a réussi à mettre au point pour permettre de définir commodément d'après des critères précis l'acceptabilité de diverses conditions pour les combinaisons de géotextiles/géomembrane/sols/charges. On a pu trouvé que des matériaux épaix, poinconnés à l'aiguille, non-tissés, dans la gamme de poids de 400 à 600 grammes par mètre carré donnent la meilleure combinaison de résistance, durabilité et facilité de transmission latérale, qui permet leur emploi comme doublures de fond dans la construction de grands bassins.

INTRODUCTION

The use of geotextile fabrics in combination with geomembranes (impermeable linings) has been rapidly gaining in worldwide recognition and importance to the design engineer. There are two primary applications developed at this time requiring the use of heavyweight (>400 g/m^2) geotextile fabrics: (a) for abrasion and puncture protection of geomembranes in solid waste landfills--including bottoms, sidewalls and covers, and (b) for both puncture protection and gas relief beneath geomembranes in liquid containment ponds. This report will deal primarily with the latter application as specified and utilized in the United States market.

Geomembranes of varying chemical composition are well established products for pond linings (1) and are being increasingly specified by design engineers--particularly for containment of toxic wastes (e.g., sodium cyanide solution catch basins in a gold ore heap leaching process). Additionally, recent studies at Texas A&M University have shown that the containment of organic fluids (basic, neutral polar and neutral nonpolar) and organic acids have been demonstrated to cause substantial increases in the permeability of clay liners (2). This evidence will undoubtedly cause an even more frequent use of geomembranes in the future. However, this increased use of geomembranes heightens several functional concerns of the designer. First, the membrane manufacturers are very explicit in requiring that the installation contractor prepare the subgrade in a finished manner that is extremely smooth in order to prevent the possibility of puncture by sharp rocks or stones protruding up into the lining. Second, many subgrades contain organic wastes that emit gasses during decomposition which can be trapped beneath the lining and cause sections of the membrane to lift and float within the pond structure. Third, since many lined ponds are constructed over existing cracked surfaces such as concrete or asphalt, wind and water forces cause severe abrasion of the geomembrane by the spalled or rough textured surface. Each of these three real and potentially catastrophic problems can be eliminated by the use of a porous, yet strong, geotextile underlining fabric.

A relatively thick, porous nonwoven geotextile comprised of polypropylene (or polyester in nonalkaline environments) can easily be placed between the subgrade (base) and geomembrane to provide a cushion against puncture, provide a lateral conduit for release of trapped subgrade gasses and provide abrasion protection against a rough surface. In addition, the geotextile fabric provides a clean environment for field seaming of the geomembrane panels which reduces the incidence of pond leakage caused by blowing sand or soil fouling the chemically bonded seams.

This paper will describe several installations using geotextiles in combination with geomembranes and detail laboratory procedures developed to aid in the selection of the proper geotextile for the intended end-use.

FIELD EXAMPLES

In 1980, a uranium ore processing plant was being constructed in central New Mexico. Within this complex were three leachate evaporation ponds that totaled over 70 acres in size. The porous subsoil and sulfuric acid leachate required the use of a butyl rubber lining material (geomembrane) having both cushion and gas relief protection. The owner/designer considered two alternative designs for the underlining system: (1) 30.5 cm of treated sand, or (2) a 400 g/m^2 spun-bonded needlepunched, nonwoven polypropylene geotextile fabric. The geotextile fabric was selected because it provided all of the required properties of strength, permeability and thickness and it was less costly to place than the sand. Fibretex Grade 400 geotextile fabric (3) marketed by Crown Zellerbach Corporation was chosen and installed on this project in August 1980. All three ponds were subsequently filled with leachate and have operated since that time without interruption.

Figure 1. Photograph of Butyl Geomembrane Being Installed Over Polypropylene Geotextile.

Recent governmental EPA regulations on the storage of toxic fluids have recommended the use of double-lined ponds or expensive monitoring systems to insure against any possible leakage of the contained fluid into the surrounding water table. Thick, heavyweight geotextile fabrics are now being specified as the separation member between two geomembranes (Figure 2A) to act as a collection medium for lost fluid, to provide a space for leakage monitoring and to aid in the protection of the lower membrane during placement of the upper membrane. The geotextile is particularly effective on steep slopes (>2:1) where sand is often impossible to place.

Another common use for geotextiles is to provide an abrasion-resistant layer over existing cracked concrete surfaces or rough textured asphalt (Figure 2B). The larger cracks (>0.5 cm) are filled with grout or asphalt and then the geotextile is placed, with adjacent panels overlapped (30 cm). The geomembrane is then placed directly over the geotextile with care to insure that all exposed rough surfaces are well padded. Particular attention should be directed to the pond's top edges where wave action can be most severe.

In all three examples, the geotextile and geomembrane are securely planted together in a trench at the top of the berm and then backfilled and compacted to prevent eventual pull-out.

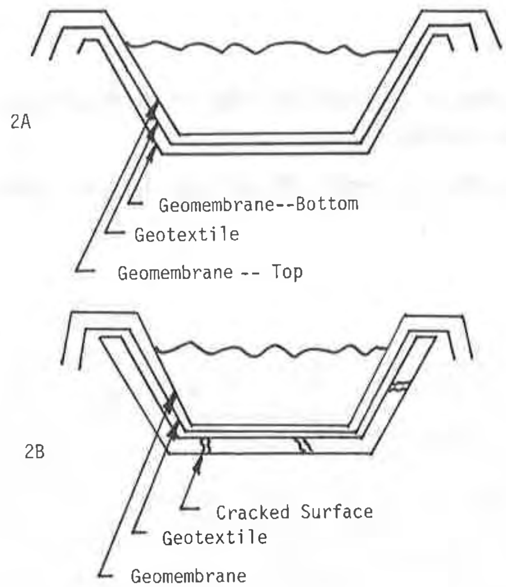

Figure 2. Installation of Geotextile in (A) Double-Lined Pond and (B) Rough Surfaced Pond.

FIELD TESTS FOR CUSHIONING

In an attempt to define a selection process that would identify the correct geotextile for puncture protection from heavy wheel loads, a field evaluation was conducted at an open pit coal mine in West Virginia. At this site, a polyvinyl chloride (PVC) geomembrane was used to contain and cover a large quantity of overburden removed from the mine. Since large ore trucks would be traversing areas underlined with the geomembrane, there was concern that puncture might occur thus releasing acidic runoff into the surrounding water supply. The use of a geotextile to protect the geomembrane from puncture was therefore investigated.

For the test, the base soil was leveled with a bulldozer, covered with 0.1 m of clean fill, and then compacted with three passes of a roller. The base was very "springy" under the roller movement, indicating unstable subsurface conditions. The geotextiles used in this study were needlepunched, spun-bonded polypropylene nonwoven fabrics with basis weights of approximately 400 g/m^2 (Geotextile A) and 600 g/m^2 (Geotextile B). These were cut into test strips of approximate dimensions: 1.2 m by 4.6 m. The fabric samples to be used under the liner were laid on the soil parallel to each other and separated by 0.6 m gaps. A single large piece of 0.51 mm (20 mil) PVC geomembrane was then placed over the geotextile samples and the soil. The fabric samples to be laid over the geomembrane were then put in place and their positions outlined with spray paint to allow identification of fabric position after the test when the soil overburden would be removed. The geotextile samples and exposed geomembrane were covered with fill so that, after compaction, there were 0.53 m of soil and crushed rock over the membrane. A four-wheel, 45,350 kg (50 ton) ore carrier was then run repeatedly over the test area. The wheels sank to within 0.15 m of the membrane. The soil was removed and damage to the geomembrane evaluated. Table 1 indicates the arrangement of geotextile samples over and under the geomembrane and the degree of damage resulting from the ore carrier wheel load. It is clear that the geotextile provided significant protection to the geomembrane thus significantly reducing puncture damage. The best protection was provided by Geotextile B (600 g/m^2) on the top and Geotextile A (400 g/m^2) under the geomembrane. Use of a single layer of Geotextile B on the top of the membrane or Geotextile A both above and below the membrane yielded slightly poorer results. Use of one layer of Geotextile B, on top of the liner, gave somewhat less protection. The degree of protection correlated directly with increasing fabric basis weight, thickness and protection from both sides. Test locations where the membrane was afforded no geotextile protection showed varying degrees of damage, with some showing complete membrane disintegration. This field trial clearly demonstrated the utility of thick needlepunched nonwoven fabrics to reduce the propensity of geomembrane puncture failure and pointed up the need for top and bottom layers where very heavy loading over poor subsoil are encountered.

TABLE I

FIELD TEST--GEOTEXTILE PROTECTING GEOMEMBRANE FROM PUNCTURE WHEN SUBJECTED TO LOADING FROM 45,000 Kg (50 TON) ORE CARRIER

Test Location	Identity and Position of Geotextile(1): Over Geomembrane	Under Geomembrane	Extent of Puncture Damage
1	B	A	None
2	None	B	Some
3	B	None	Very Slight
4	A	None	Slight
5	None	A	Some
6	A	A	Very Slight
7	None	None	Varying Degrees Up to Membrane Disintegration

(1) Geotextiles A and B are needlepunch spun-bonded polypropylene nonwoven fabrics of basis weight 400 g/m^2 and 600 g/m^2, respectively.

LABORATORY TESTS FOR CUSHIONING

Field trials as described above are too expensive and too time consuming for use in screening the performance of many different geomembrane-geotextile combinations. Therefore, a laboratory test method was developed that could simulate cyclic compression loading of the geomembrane against the soil-aggregate environment of interest. After subjection to the aggregate environment, the degree of damage was quantitatively determined by measuring the degree of air leakage through the geomembrane sample. The required load and number of cycles to simulate ore transport or equipment movement over the proposed site was achieved using an Instron testing machine cycling to the required compression load.

This test method was used to simulate geomembrane abuse expected during transport and dumping of ore onto a containment site used for heap leaching of gold ore. The experimental assembly of aggregates, geotextiles, and geomembrane mounted on the compression cell of an Instron testing machine is shown in Figure 3. The geomembrane, unprotected or covered top and/or bottom with geotextile, was placed on a 2 cm thick layer of 2 cm diameter aggregate. A 4 cm thick layer of finer, 0.5 cm diameter aggregate was then placed on the top of the membrane. Quartzite gold ore from a New Mexico mine was used as the aggregate. The assembly was placed on top of the compression cell of an Instron testing machine. A solid steel cylinder, 103 cm^2 in area and protected with a rubber gasket material was attached to the Instron crosshead bar to deliver compression force to the top of the aggregate assembly. The testing machine was then allowed to cycle 18 times/minute between 68 and 363 kg (150 lbs and 800 lbs) load to deliver 258 kPa - 1378 kPa (37.5 psi - 200 psi) compression force to the aggregate covering the plastic membrane. After cycling 30 minutes, the system was disassembled for damage evaluation. The system was then reassembled and cycling continued for a second 30-minute period. Damage was evaluated both visually and by measuring air flow under 69 kPa (10 psi) pressure through the geomembrane with a Sheffield Porosimeter.

Figure 3. Apparatus to Estimate Geotextile Protection of Plastic Pond Liner Membrane During Aggregate Compression Loading.

Geomembrane resistance to failure was found to be dependent upon geomembrane type, geomembrane caliper, type of geotextile used for cushioning and basis weight of that geotextile. Results for a wide selection of geomembranes, both unprotected and protected top and bottom with geotextiles, are summarized in Table 2. The utility of nonwoven geotextiles to increase geomembrane resistance to failure from aggregate cutting, puncture and abrasion was clearly demonstrated. The liners tested included low density polyethylene (LDPE), medium density polyethylene (MDPE), high density polyethylene (HDPE), alloy of high density polyethylene (HDPE) and ethylene propylene rubber (EPDM), polyvinylchloride (PVC), oil resistant polyvinylchloride, chlorosulfonated polyethylene (CSPER, with polyester fiber reinforcement) and chlorinated polyethylene rubber (CPER, with polyester fiber reinforcement). In most cases, two calipers of each type of plastic liner were tested. The effectiveness of no geotextile or Geotextiles C, A, and B, respectively, needlepunched spun-bonded polypropylene nonwoven fabrics having 300 g/m^2, 400 g/m^2, and 600 g/m^2 basis weights were evaluated.

The results in Table 2 suggest significant differences in the resistance of plastic liners to aggregate damage and significant differences in the degree of protection afforded by the different basis weights of the geotextile. The three 1.52 mm (60 mil) plastic liners tested showed significant resistance to failure even without geotextile protection. However, the use of Geotextile C (300 g/m^2) seemed to provide some protection for 1.52 mm (60 mil) LDPE and MDPE. All other plastic membranes besides the 1.5 mm (60 mil) products failed without geotextile protection. Geotextile C (300 g/m^2) on both sides yielded promising results with HDPE 0.76 mm (30 mil), LDPE 0.76 mm (30 mil) and CPDE 0.76 mm (30 mil). Note however that the latter two liners had failures in one of two replicates with Geotextile A (400 g/m^2) used on both sides. Geotextile A on both sides yielded sufficient protection to minimize leakage through MDPE 0.76 mm (30 mil), Alloy (HDPE + EPDE) 0.76 mm (30 mil), Alloy 1.02 mm (40 mil), PVC 0.51 mm (20 mil) and PVC 0.76 mm (30 mil). Geotextile C on both sides protected CSPER 0.91 mm (36 mil). The PVC oil resistant liner was the most difficult to protect using geotextiles.

Geomembrane protection via a single layer of geotextile above the membrane was also explored. The most promising results were seen with use of a single layer of Geotextile B (600 g/m^2) where a significant reduction in leakage was observed for all the geomembranes except PVC 0.51 mm (20 mil), PVC 0.76 mm (30 mil), and PVC oil resistant 0.76 mm (30 mil). Note that these results suggest that the compression cycling test for 1080 cycles was somewhat more extreme than the field test described above. In that experiment one layer of Geotextile B over 0.51 mm (20 mil) PVC provided a very high degree of protection.

Limited experimentation using this cyclic compression test method was carried out to compare the effectiveness of different types of geotextiles to protect several different types of geomembranes during 30 minutes of cyclic compression loading against aggregate. Table 3 compares results for the unprotected membrane; the membrane protected with Geotextile D, a spun-bonded, nonwoven polypropylene fabric of 136 g/m^2 basis weight and caliper of 0.38 mm (15 mils); and the membrane protected with Geotextile C, a needlepunched, spun-bonded polypropylene fabric of 300 g/m^2 basis weight and caliper of 2.29 mm (90 mils). As seen above, the unprotected geomembrane failed in all cases. Response for geomembranes covered top and bottom with the geotextiles was dependent upon both the type of geomembrane and the type of geotextile. For CPER (polyester fiber reinforced) both types of geotextiles provided acceptable protection. However for LDPE, MDPE, and CSPER (polyester fiber reinforced), the thick needlepunched nonwoven geotextile yielded much greater protection than observed with the thin nonwoven. For the two PVC liners neither geotextile afforded sufficient protection to prevent significant damage as indicated by air leakage. However, use of Geotextile A , the 400 g/m^2 analog of Geotextile A, protected both geomembranes.

Cyclic compression testing was also used to compare the cushioning effectiveness of geotextiles versus sand to protect the geomembrane. Results for a 0.76 (30 mil) PVC and a 0.91 mm (36 mil) CSPER (polyester reinforced) geomembrane are shown in Table 4. Both fail without protection. Similar protection was observed using one inch of sand under the membrane or a layer of Geotextile C, 300 g/m^2 basis weight nonwoven on the top and bottom of the membrane. Thus, with either of these systems, CSPER did not fail but PVC was severely damaged. Both geomembranes were protected by use of Geotextile A, 400 g/m^2 basis weight nonwoven.

Thus, a laboratory test method was developed to simulate geomembrane and geotextile-geomembrane resistance to repetitive loading such as generated by dumping or transport in heap leach mining. This method used cyclic compression loading generated by a tensile testing machine to subject the candidate geomembrane system to the cutting, punching, and abrasive action of the aggregate. Results demonstrated that geomembrane type, geomembrane caliper, geotextile caliper, and geotextile basis weight were important factors that determine resistance to damage. The cyclic compression loading method was also used to compare the relative protecting or cushioning effectiveness of sand and geotextiles. Conclusions from this test method were in agreement with limited field evaluations that also identified geotextile basis weight and caliper as important factors to preventing geomembrane failure.

GAS TRANSMISSION

Gas build-up under the geomembrane with possible flotation or rupture of the membrane can be prevented by use of a thick, nonwoven geotextile to allow gas transmission to the outside. Fluid permeability in the plane of the geotextile determines its utility in this application. Planer air permeability values were obtained via modifying a Frazier apparatus (4) by placing a piece of plastic film over the fabric so air flow was made to move through the plane of the fabric. The geotextile was held under 44.8 kPa (6.5 psi) compression pressure to simulate the force of 4.6 m (15 feet) of water down on the geomembrane and fabric. A pressure difference of 124 Pa was maintained through the fabric. Since the Frazier apparatus is calibrated for air movement through an area 156 cm^2 (4.91 in^2) the indicated air flow must be adjusted for the cross-sectional area of the geotextile that the air actually moves through. This area is determined by the diameter of the apparatus and the fabric caliper under the compression force. Thus, the resulting permeability was the flow per cross-sectional area of the fabric. Multiplication by caliper gave air transmission per linear dimension of fabric edge. Air transmission values for several fabrics are shown in Table 5. While these values are useful to rank geotextiles, it is unclear how well they predict the flow of gas from the center of the pond through the fabric to the edge of the pond. For a 4.6 m (15 ft) deep pond, there is a 44.8 kPa (6.5 psi) pressure differential to drive the gas compressed in the fabric out through the edge. Further work to quantify the gas transmissivity of geotextiles such as has been done by Koerner and Sankey with water as the fluid (5) seems justified.

TABLE 2

GEOMEMBRANE PUNCTURE RESISTANCE--GEOTEXTILE PROTECTION ON BOTH SIDES OF THE MEMBRANE. RESULTS AFTER 60 MINUTES OF CYCLIC COMPRESSION LOADING, 1080 CYCLES TOTAL

Geomembrane		Geomembrane puncture resistance as indicated by minimized air flow (cm^3/min) through the membrane after cyclic compression loading. Protection by indicated geotextile(1)			
Type	Caliper	Unprotected Membrane	Fabric C (300 g/m^2)	Fabric A (400 g/m^2)	Fabric B (600 g/m^2)
Low Density Polyethylene (LDPE)	0.76 mm (30 mil)	400+	2.0	98 3.0(2)	2.0
Low Density Polyethylene	1.52 mm (60 mil)	17	2.0	2.0	No Result
Medium Density Polyethylene (MDPE)	0.76 mm (30 mil)	400+	201	2.0	2.0
Medium Density Polyethylene	1.52 mm (60 mil)	37	2.0	3.0	No Result
High Density Polyethylene (HDPE)	0.76 mm (30 mil)	400+	0.0	0.0	0.0
High Density Polyethylene	1.52 mm (60 mil)	2.0	1.0	400 1.0(2)	0.0
Alloy of High Density Polyethylene and Ethylene Propylene Rubber (Alloy)	0.76 mm (30 mil)	400+	311	2.0	1.0
Alloy of High Density Polyethylene and Ethylene Propylene Rubber	1.02 mm (40 mil)	400+	245	0.0	0.0
Polyvinyl Chloride (PVC)	0.51 mm (20 mil)	400+	400+	3.0	1.0
Polyvinyl Chloride	0.76 mm (30 mil)	400+	400+	2.0	1.0
Oil Resistant Polyvinyl Chloride	0.76 mm (30 mil)	400+	328	148 2.0(2)	176 2.0(2)
Chlorinated Polyethylene Rubber With 10x10, 1000 Denier Polyester Fiber Reinforcement (CPER)	0.87 mm (35 mil)	400+	2.0	400+ 3.0(2)	3.0 1.0(2)
Chlorosulfonated Polyethylene Rubber With 10x10, 1000 Denier Polyester Fiber Reinforcement (CSPER)	0.91 mm (36 mil)	400+	107	233	0.0

(1) Geotextiles A, B, C are needlepunched, spun-bonded polypropylene nonwoven fabrics of indicated basis weight.
(2) Results from a second experiment.

TABLE 3

COMPARISON OF GEOTEXTILES TO IMPROVE PUNCTURE RESISTANCE OF GEOMEMBRANE--RESULTS AFTER 30 MINUTES OF CYCLIC COMPRESSION LOADING, 520 CYCLES TOTAL

Geomembrane	Geomembrane puncture resistance as indicated by minimized air flow (cm^3/min) through the membrane after cyclic compression loading. Protection by indicated geotextile.		
	Unprotected Membrane	Geotextile D(1) Both Sides of Liner	Geotextile C(2) Both Sides of Liner
LDPE, 0.76 mm (30 mil)	400+	400+	2.0
MDPE, 0.76 mm (30 mil)	400+	400+	21
PVC, 0.76 mm (30 mil)	400+	400+	302
PVC, Oil Resistant 0.76 mm (30 mil)	400+	400+	302
CPER, 0.87 mm (35 mil) (10x10, 1000 denier polyester fiber reinforcement)	400+	4.0	2.0
CSPER, 0.91 mm (36 mil) (10x10, 1000 denier polyester fiber reinforcement)	400+	400+	1.0

(1) Geotextile D is a spun-bonded polypropylene nonwoven fabric of basis weight 136 g/m^2 and 15 mil caliper.

(2) Geotextile C is a needlepunched spun-bonded polypropylene nonwoven fabric of basis weight 300 g/m^2 and 90 mil caliper.

TABLE 4

USE OF GEOTEXTILES OR SAND TO IMPROVE PUNCTURE RESISTANCE OF GEOMEMBRANES--RESULTS AFTER 30 MINUTES OF CYCLIC COMPRESSION LOADING, 520 CYCLES TOTAL

Geomembrane	Geomembrane puncture resistance as indicated by air flow (cm^3/min) through the membrane after compression loading. Protection by indicated geotextile.		
	Unprotected Membrane	Sand 1" on Bottom Side of Liner	Geotextile C(1) Both Sides of Liner
PVC, 0.76 mm (30 mil)	400+	400+ 400+(2)	302
CSPER, 0.91 mm (36 mil) (10x10, 1000 denier polyester fiber reinforcement)	400+	2.0	1.0

(1) Geotextile C is a needlepunched, spun-bonded polypropylene nonwoven fabric of basis weight 300 g/m^2 and 90 mil caliper.
(2) Results from a second experiment.

TABLE 5

GEOTEXTILE PLANER GAS PERMEABILITY-- MODIFIED FRAZER METHOD(1)

Geotextile-- Basis Weight(2)	Gas Transmission Per Meter of Fabric Edge (L/s-m)	Per Foot of Fabric Edge (ft^3/min ft)
E-150 g/m^2	0.59	0.38
F-200 g/m^2	0.79	0.51
C-300 g/m^2	1.67	1.08
A-400 g/m^2	1.32	0.85
B-600 g/m^2	2.11	1.36
G-500 g/m^2	1.05	0.68
H-300 g/m^2	0.96	0.62

(1) Fabric under 44.8 kPa (6.5 psi) compression to simulate 4.5 m (15 ft) of water. Pressure difference of 124 Pa was present through the fabric.

(2) Geotextiles E, F, C, A, B are needlepunched, spun-bonded polypropylene nonwoven fabrics. Geotextile G is needlepunched, spun-bonded polyester nonwoven fabric. Geotextile H is a needlepunched staple polypropylene nonwoven fabric.

SUMMARY AND CONCLUSIONS

Both field and laboratory tests have been conducted in order to optimize geotextile fabric properties for use as underlining in geomembrane pond construction. The geotextile has been demonstrated to be functionally useful as a cushion against puncture, as a gas release medium and as an abrasion resistant layer over rough surfaces. Laboratory test methods were developed which simulate field puncture problems, various combination of geotextile/geomembrane were evaluated and test data reported. From this work, it can be concluded that a thick, nonwoven, needle-punched polypropylene geotextile can be used to provide the essential protection functions noted above for pond construction. Commercial use of geotextiles as pond underlining can be expanded without fear of failure or deterioration if care is exercised in the geotextile/geomembrane selection.

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The Use of Geotextile for Wrapping Large Depth Drain

Utilisation d'un géotextile en tranchée drainante de profondeur importante

For the construction of a motorway in the department of Hautes Pyrénées-France-it had to make a large excavation (10 m deep) in satured clays (W_L 30 % Ip 14) and to digout a network of 8 draining ditches, each 3.5m deep and streching over a total lenght of 7 Km.

This communication describes the inventive concept of the work and the method of calculation used for determining its dimensions.

Then follows a description of the applied technology in wich a nonwoven-needled geotextile. (SODOCA AS 250) operating as filter has enabled the work to be completed in the best technical and economical conditions.

Finally, four month after the end of the earthworks. the slopes do not show any disturbance.

La construction d'une section d'autoroute dans le département des Hautes Pyrénées a nécessité la réalisation d'un grand déblai (de 10 m de profondeur) dans des argiles saturées (W_L 30 %. Ip 14) et la mise en place d'un réseau de 8 tranchées drainantes de 3,5 m de profondeur chacune sur une longueur totale de plus de 7 Km.

Cette communication décrit la conception originale de l'ouvrage et la méthode de calcul utilisée pour son dimensionnement. Elle décrit également la technique de mise en oeuvre dans laquelle l'emploi d'un géotextile non tissé aiguilleté (SODOCA AS 250) comme filtre a permis de réaliser l'ouvrage dans les meilleures conditions techniques et économiques.

Enfin 4 mois après la fin des terrassements, il est fait part des constatations qui mettent en évidence le bon fonctionnement de l'ouvrage.

La réalisation d'un tronçon d'une longueur de 7 kms de l'autoroute A 64, entre la fin du plateau de Lannemezan et Capvern, dans le département des Hautes Pyrénées (France), a nécessité l'éxecution d'un déblai de plus de 10 mètres de hauteur dans des formations constituées par " les argiles de Lannemezan ".

L'étude géotechnique préalable confiée au Laboratoire régional des Ponts et Chaussées de Toulouse a permis de préciser, d'une part, la nature, l'état et les caractéristiques mécaniques, et, d'autre part d'étudier les conditions de stabilité des pentes des talus de déblais et de proposer les conditions particulières de réalisation d'un réseau de tranchées drainantes comportant un filtre géotextile.

1- CARACTERISTIQUES DES SOLS

1-1. Caractéristiques géotechniques :

Le déblai est entièrement situé dans les argiles de Lannemezan et les sédiments du Pontieu correspondant à des argiles peu plastiques et à des limons argileux (fig. 1) dont les caractéristiques moyennes sont :

W_L = 34 Ip = 14 pour les premières, et

W_L = 43 Ip = 16 pour les secondes.

Le poids volumique moyen du sol sec est sensiblement le même pour les deux matériaux $\gamma_d \simeq 19$ kN/m3.

Ces sols sont généralement saturés, leur teneur en eau est toujours voisine de 30 % et parfois supérieure. Ce sont des sols fins, argilo-siliceux humifères, très acides (absence totale de calcaire) où l'on peut rencontrer localement des passages très graveleux avec des galets de diamètres importants (supérieurs à 400 mm) et peu de fines. Vis à vis des problèmes d'extraction et de réemploi en terrassements, ce sont des matériaux, d'après les Recommandations pour les Terrassements Routiers (1) de classe A_2 et plus généralement C_1, avec une matrice argileuse prépondérante.

1-2. Caractéristiques mécaniques et hydrauliques :

. Essais in situ : à partir des résultats de sondage scissométriques et pénétrométriques, la cohésion apparente moyenne C_u est de l'ordre de 60 k Pa.

La perméabilité en place, déterminée à partir d'un essai d'eau Lefranc est en moyenne de $K = 10^{-8}$ m/s. Sur toute la longueur du déblai, la nappe phréatique est pratiquement en surface par rapport au terrain naturel.

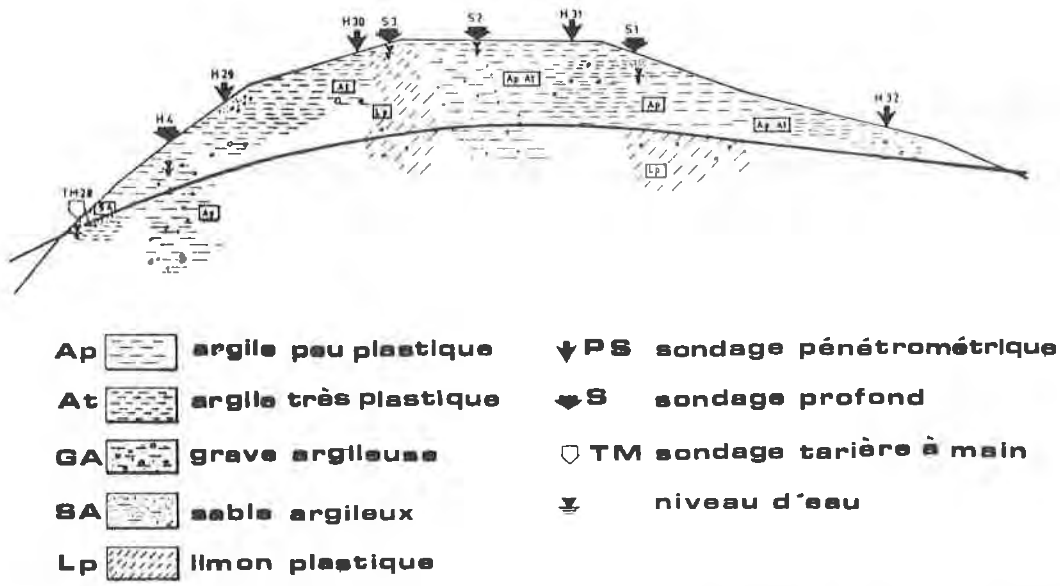


Fig. 1 : Profil en long géotechnique du déblai

. Essais en laboratoire : les essais à court terme (scissomètre de laboratoire) ont confirmé les résultats des essais in situ. Les essais triaxiaux (essai consolidé non drainé) ont permi de déterminer les caractéristiques intrinsèques du matériau qui sont homogènes avec une cohésion effective $0 \leqslant C' \leqslant 15$ kPa et un angle de frottement effectif $33 \leqslant \phi' \leqslant 35°30'$.

1-3. Problèmes de stabilité

La réalisation de la tranchée perturbe les conditions initiales de répartitions des contraintes et des pressions au sein du terrain.

A court terme, les calculs de stabilité effectués en prenant en compte une cohésion apparente C_u = 60 kPa, ont montré que pour un angle à la base inférieur à 53°, le coefficient de sécurité est constant et égal à 1,73. Ces résultats permettaient donc de conclure que pendant les terrassements proprement dit, la stabilité à court terme du talus de 10 m de hauteur ne posait pas de problème ; mais les travaux s'effectuant sous la nappe phréatique , il était toutefois nécessaire de prendre des dispositions constructives pour recueillir et évacuer les eaux au fur et à mesure des terrassements.

Les débits à évacuer étant faibles dans les zones homogènes ($K = 10^{-8}$ m/s) mais pouvant atteindre localement 22 l/s/ml dans les zones très caillouteuses ($K = 10^{-4}$ m/s).

Pour les valeurs moyennes de l'angle de frottement effectif $\phi' = 33°$ et une cohésion effective, c' = 5 kPa, la stabilité à long terme (coefficient de sécurité FS $\geqslant$ 1,3) ne pouvait être assurée pour des pentes de talus avec un angle à la base extrêmement faible, ce qui entraînait alors des déblais considérables. Un dispositif de rabattement de la nappe a donc été étudié. L'étude a été faite par analogie électrique (fig. 2). Un réseau de drainage constitué, sur chaque talus par quatre tranchées drainantes parallèles à l'axe de l'autoroute et de 3,50 m de profondeur a été retenu. Avec des pentes de talus de Cotg . β = 2/1, l'ensemble du dispositif permet de résoudre le problème de stabilité à long terme, et assure un coefficient de sécurité théorique à long terme de 1,4 (fig. 3).

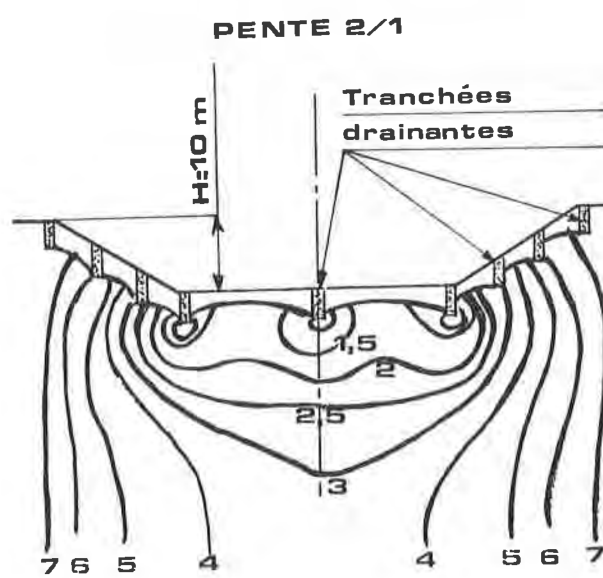


Fig. 2 : Réseau d'équipotentielles.

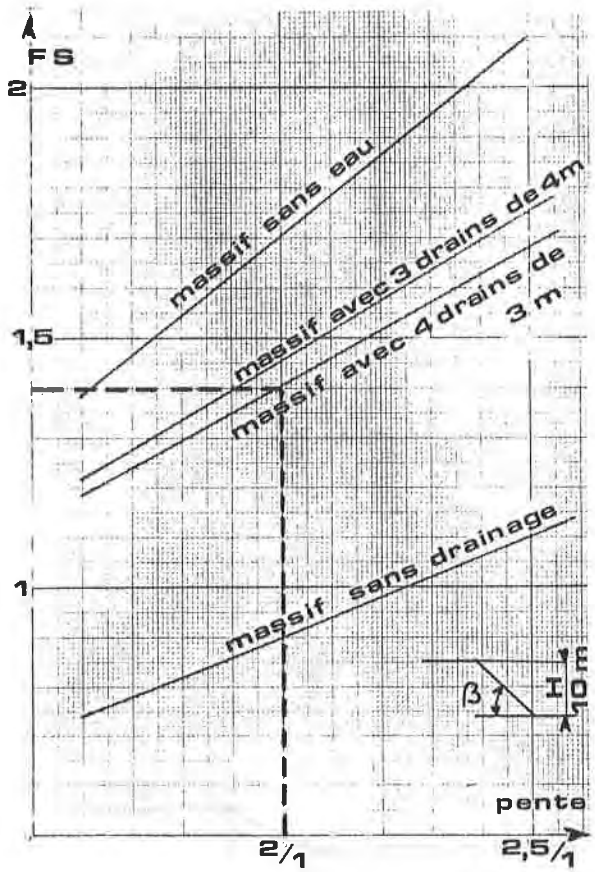


Fig. 3 :Variation du coefficient de sécurité théorique FS en fonction de la pente du talus.

2 - CARACTERISTIQUES DES TRANCHEES DRAINANTES

D'après les perméabilités évaluées par l'essai Lefranc, $K = 10^{-8}$ m/s, le débit à capter par drain et par mètre linéaire est de $Q = 10^{-4}$ l/s/ml.

Le matériau de remplissage doit respecter des normes bien précises si l'on veut obtenir un fonctionnement satisfaisant de l'ensemble, celles-ci ne doivent pas être modifiées à la mise en oeuvre. La définition de ces caractéristiques a été faite en appliquant les règles de filtre de Terzaghi :

$$(1) \quad \begin{array}{l} \dfrac{F\,85}{} \geqslant 2 \\ 4 \text{ à } 5\ B_{15} < F_{15} < 4 \text{ à } 5\ B_{85} \\ F_{60} < 2.F_{40} \end{array}$$

A partir du sol de base B, le fuseau granulométrique dans lequel doit s'inscrire le matériau est assez large, ce qui laisse le choix entre une granulométrie uniforme ou étendue. Compte tenu des possibilités en matériau de la région, le choix s'est porté sur une grave propre à granulométrie assez continue, associée à un géotextile.

Le rôle du filtre en géotextile étant dans ce cas d'assurer la permanence du fonctionnement de la tranchée drainante:

- en evitant toute contamination du matériau drainant, aussi bien à la mise en oeuvre que pendant son fonctionnement,
- en permettant à l'eau de passer sans entraîner d'augmentation importante de la pression interstitielle.

Pour son dimensionnement, les règles suivantes ont été appliquées. La première compare la dimension des pores du géotextile avec les dimensions du sol à filtrer

$$(2) \quad O\ 95 \leqslant B_{85}$$

On utilise un seul diamètre O_{95} qui est, par définition tel que 95 % des pores aient un diamètre inférieur à O_{95} et 5 % un diamètre supérieur.

La valeur de O_{95} (2) étant déterminée à partir de la dimension des pores du géotextile obtenue en faisant passer à travers celui-ci un matériau de granulométrie connue et appropriée en suspension dans l'eau (3).

La seconde règle porte sur le contraste de perméabilité

$$(3) \quad \text{Kn filtre} \gg 100\ \text{K sol à protéger}$$

Enfin, étant donné les conditions particulières de remblaiement dans les tranchées drainantes de grande profondeur risquant de soumettre, lors du déversement du matériau, le géotextile à des efforts localisés importants, il a été recherché un compromis entre les caractéristiques du filtre et le mode de déversement.

3 - REALISATION DES TRANCHEES DRAINANTES

Dans le déblai d'une longueur d'environ 1000 m, il a été réalisé 3.500 mètres linéaires de tranchée de 3m 50 de profondeur et 0,80 m en moyenne de large, ce qui a nécessité l'emploi de 35.000 m2 de géotextile. Une partie des tranchées initialement prévue a pu être réduite par modification de l'ouverture en tête du déblai (création d'un emprunt par exemple).

Afin de limiter au maximum les venues d'eau, le terrassement a été conduit en réalisant au préalable les drains de chaque côté de la fouille de déblai, puis l'extraction des matériaux jusqu'à une profondeur inférieure de 0,50 m à la profondeur de la tranchée drainante. La figure 4 résume le procédé.

Les caractéristiques des matériaux utilisés et répondant aux critères de dimensionnement fixés en (1) et (2) sont les suivantes :

- Matériau de remplissage :

  Il s'agit d'une grave naturelle extraite dans l'eau et provenant d'un emprunt situé à environ 15 km du tracé, dans les alluvions récentes de la Neste (fig. 5). Sa granulométrie est assez grosssière, elle comporte des caillou de gros diamètre, le pourcentage de grains inférieurs à 0,08 mm est faible (inférieur à 3 %), et son équivalent de sable élevé (Es supérieur à 70).

Le géotextile utilisé est un nontissé aiguilleté en filaments continus de polypropylène de marque SODOCA AS 250. Ses caractéristiques moyennes sont données dans le tableau ci contre.

Tableau 1 : Caractéristiques du SODOCA AS 250.

	Valeur	Classe C.F.G. (2)
- Résistance à la traction en kN/m	12,75	4
- Allongement à la rupture ε_R %	100	12
- Résistance à la déchirure kN	1,2	7
- Permittivité Kn/e S^{-1}	0,6	7
- Transmittivité Kt.e m2/s	4.10^{-7}	6
- Porométrie O 95 μm	86	7

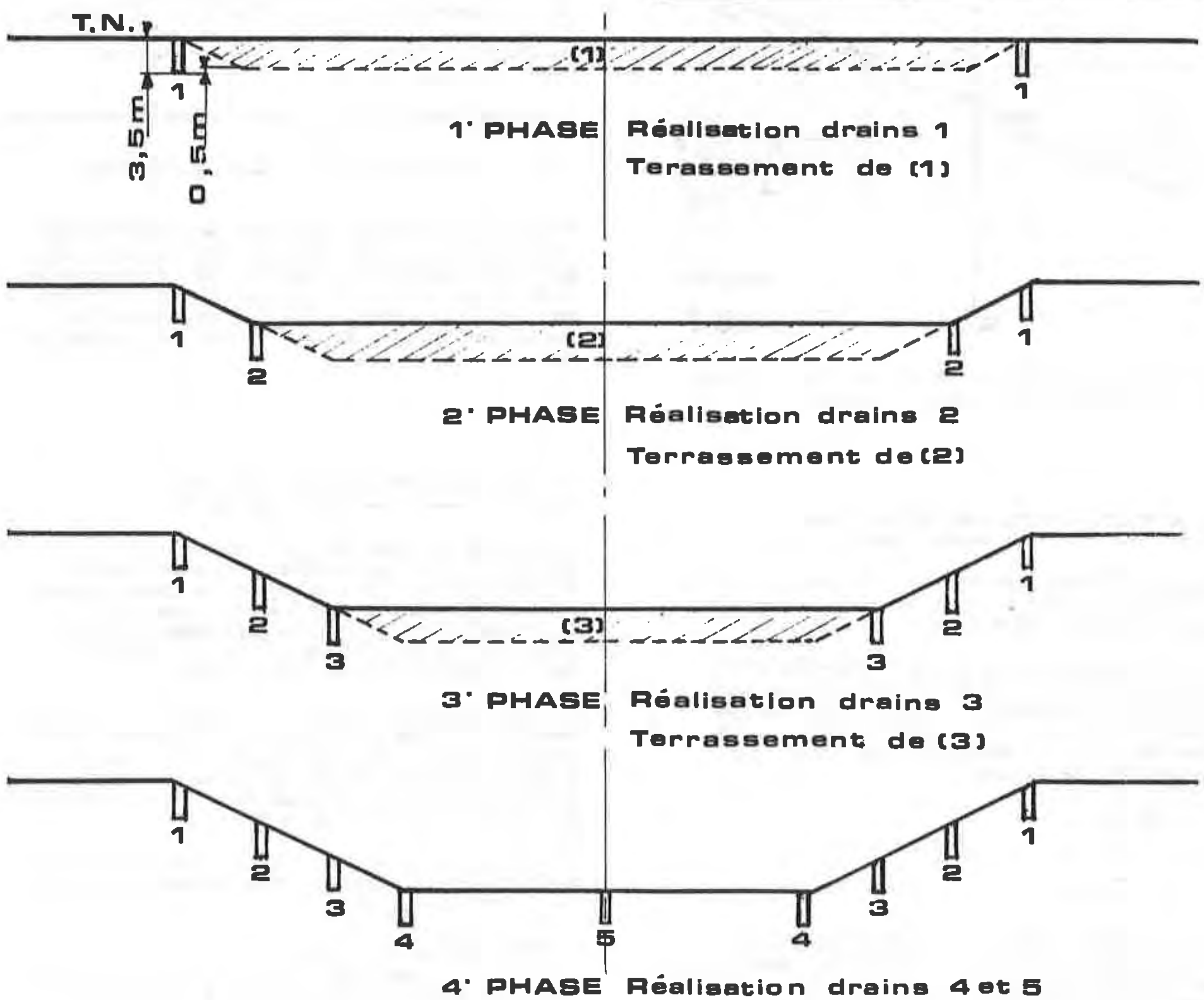
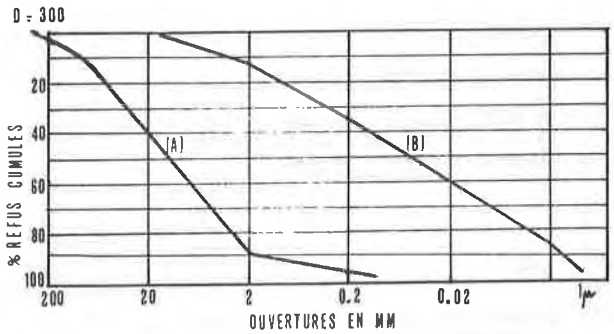


Fig. 4 : Mode de réalisation des tranchées.



(A) MATERIAU DRAINANT
(B) MATERIAU DE BASE

Fig 5 Caractéristiques granulométriques moyennes du matériau drainant et du matériau de base.

La réalisation proprement dite des tranchées a été conduite comme suit :

- Exécution de la tranchée à la pelle hydraulique jusqu'à sa profondeur maximale
- Mise en place du géotextile par bandes successives d'une largeur correspondante à celle du produit livré soit 5,30 m, préalablement découpées à une longueur permettant de recouvrir la totalité de la paroi de la tranchée y compris la longueur nécessaire au recouvrement, soit 10 m environ (fig. 6).
- Déversement de la grave naturelle à l'avancement des travaux afin d'éviter tout risque de désordre dans la tenue des parois de la tranchée (fig. 7).

Fig. 6

Fig. 7

Les recommandations à la mise en oeuvre ont porté essentiellement sur la nécessité d'assurer une parfaite continuité de la tranchée drainante et de veiller à l'excellent recouvrement des bandes de géotextiles (0,5 m minimum) et enfin d'éviter toute pollution lors de sa mise en oeuvre.

Actuellement, quatre mois environ après l'achèvement des terrassements, on constate un bon fonctionnement général du système de rabattement successif de la nappe phréatique. Pendant les travaux, les venues d'eau ont pu ainsi être évitées. Les talus ne présentent aucun désordre (fig. 8), ce qui n'a pas été le cas comme nous avons pu le constater, sur un autre talus de pente identique, dans les mêmes types de terrains et pour lequel cette méthode n'avait pas été adoptée dès l'exécution.

Fig. 8

Les difficultés rencontrées à l'exécution des tranchées portent sur :

- les caractéristiques du matériau drainant (présence de cailloux de diamètre trop important, propreté du matériau insuffisante Es < 40)
- recouvrements défectueux du géotextile, largeur insuffisante et même sens de recouvrement contraire au sens d'écoulement
- déchirement accidentel du géotextile lors du déversement du matériau drainant.

Ces quelques incidents ont nécessité une reprise des zones défectueuses.

CONCLUSION

L'utilisation d'un géotextile comme filtre dans la réalisation de tranchées drainantes dimensionnées pour le rabattement successif de la nappe phréatique dans un déblai de 10 m de hauteur a permis de réaliser l'ouvrage routier dans les meilleures conditions techniques et économiques.

Cette technique a permis d'utiliser des matériaux locaux sans élaboration complexe et coûteuse.

Leur mise en oeuvre a pu être effectuée simplement évitant ainsi le recours à des méthodes longues et onéreuses comme l'étayement qui aurait été nécessaire en cas de mise en oeuvre d'un filtre composite.

Une attention toute particulière doit toutefois être portée sur les conditions pratiques de mise en oeuvre, surtout en ce qui concerne le mode de liaison des bandes de géotextile, une couture étant à notre point de vue préférable à un recouvrement.

Enfin, on devra s'assurer que les caraetéristiques du matériau de remplissage sont toujours compatibles avec les caractéristiques mécaniques du géotextile et en particulier il faudrait limiter la présence de cailloux de dimensions trop importantes tout en essayant de privilégier les matériaux naturels propres à granulométrie resserrée.

REMERCIEMENTS

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The Development of Fin Drains for Structure Drainage

Le développement des drains en épi pour le drainage des ouvrages

Fin drains, a combination of a plastic core and a synthetic fabric, are now becoming widely used in ground and structure drainage. The plastic core is required for its hydraulic conductivity and the surrounding fabric operates as a filter. Sometimes the combination is completed by including a plastic pipe. This paper examines their use as a replacement for conventional materials in structural drainage. The hydraulic conductivity and filtration properties of conventional drainage materials are reviewed briefly as a background to the technical demands placed on fin drains. Laboratory testing methods to examine the hydraulic flow and filtration properties of fin drains under varying conditions are presented. A limited number of fin drains are put forward together with their published specifications. Comment on fin drain applications is made with a view to illustrating the installation systems employed and to identify the advantages to be gained from using this type of structure drainage.

Les drains en épi des combinaisons d'une âme en plastique et d'un géotextile en fibres synthetiques, sont de plus en plus utilisés pour le drainage des structures et des terrains. La conductivité hydraulique necessite l'âme en plastique et le géotextile joue le rôle d'un filtre. Un tuyeau en plastique est parfois utilisé pour compléter l'ensemble. Cette communication examine l'exploi des drains en épi pour remplacer des materiaux conventionnels pour le drainage des structures. Le permeabilité et les propriétés de filtration des materiaux conventionnels sont examinées en bref comme fond aux exigences techniques aux drains en épi. Les essais de laboratoire examinent le débit d'eau et les propriétés filtrantes de ces drains sous des conditions variées sont presentés. Quelques-uns de ces drains sont presentés avec leurs propriétés publiées. Des applications sont décrite pour illustrer les systèmes de pose et pour montrer les avantages que ce type de drain peut offrir.

1 Introduction

In 1968 the US Joint Highway Research Advisory Council sponsored the development of a prefabricated underdrain system in the Civil Engineering Dept of the University of Connecticut. The object was to reduce or eliminate the problems associated with the use of conventional aggregate filters. The prefabricated system of a plastic core fitting into a slit plastic pipe and protected by synthetic fabric was described in detail by Healy and Long at the Paris Conference in 1977 (1). The current availability of a range of synthetic fin drains is evidence of the success of this American development.

This discussion limits itself to the drainage of structures, an application ideally suited to fin drains which offer:-

i a readily available material with known filtration and hydraulic flow properties;

ii easy installation, thereby offering construction economies;

iii a protection of any water proofing applied to the exterior of the structure.

The objectives of the paper are to identify the drainage and filtration requirements of structural drainage and to ascertain how well fin drains meet these demands.

There is a discussion of the testing of fin drains and their components as well as mention of a limited number of products and applications.

2 Hydrostatic Pressure Effect on Buried Structures

It is important to control the hydraulic activity within the backfill against buried structures. A rising water level due to seepage within the backfill can produce a hydrostatic pressure on the structure much greater than the active thrust exerted on the structure by the unsaturated soil alone (fig. 1). As a result it is important to reduce or eliminate water pressure by the provision of drainage. This drainage layer is normally placed immediately against the structure with a drainage outlet at the base either through weep holes in the structure or by means of a pipe or a combination of both.

Apart from reducing pressure such drainage will also prevent:-

i softening and subsequent loss of strength of cohesive material;

ii control water movement through fissures resulting from dry weather;

iii reduce or eliminate the pressure effect resulting from frost.

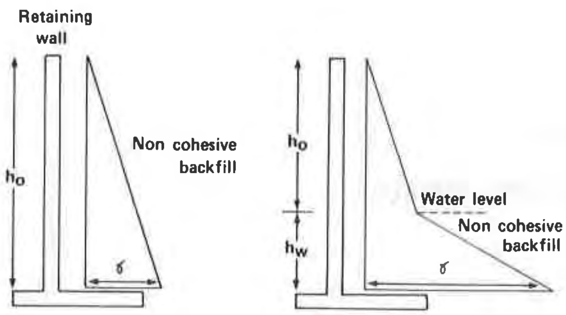


Fig. 1. EFFECT OF INCREASED WATER LEVEL ON EARTH RETAINING STRUCTURES.

Example - sand backfill, dry.

$$\sigma = Ka \times \gamma d \times h$$
$$= 0.22 \times 18 \times h$$
when $h = 10$ m
$$= 39.6 \text{ kN/m}^2$$

Example - sand backfill, partially saturated

$$= (Ka \times \gamma d \times h_o) + (Ka_{sat} \times \gamma_{sat} \times h_w)$$
$$= (0.22 \times 18 \times h_o) + (1 \times 27 \times h_w)$$
when $h_o = 7$m and $h_w = 3$m
$$= 108.72 \text{ kN/m}^2$$

3 Drainage Design Requirements

a Hydraulic conductivity

To provide an indication of the flow rates which might be expected for structural drainage, it is useful to examine some national examples and specifications.

United Kingdom

Two situations are examined using agricultural and urban drainage design criteria for a structure of 4 m height and a run off area of 4 m per lin metre of structure.

Agricultural drainage

Designs are based on M.A.F.F. recommendations (2), which uses 0.9 x rainfall at a specified probability. For this example the design rainfall relates to Buckinghamshire, which would occur over 5 consecutive days when the soil is already saturated and which is exceeded 10 times in 20 years. This rainfall would be 6 mm/day and therefore run off would be 5.4 mm/day. Total run off for this example would be 5.4 mm x 4 m resulting in 21.6 L/day m = 0.00025 L/s m.

Urban drainage

Assuming a storm subsequent to backfilling and prior to road surfacing and completion of positive surface drainage. A heavy rainfall at a rate of 50 mm/hour for a period of 5 minutes would result in a flow rate of 0.057 L/s m. (3)

West Germany

According to DIN 1185 agricultural drainage requires an uptake of 2 L/s hectare corresponding to a flow of 0.02 L/s m for a 100 m run off.

For dam construction (DIN 1184) to cope with water seepage a drainage standard on subgrades of low to very high permeabilities, varies from 0.01 to 0.05 L/s m respectively.

In DIN 4095, Drainage of subsoil for the protection of structures, no flow rates are quoted, but drainage materials are recommended, refer Fig. 2.

Sweden

The recommended drainage for buildings is 0.03 - 0.06 L/s m (4).

b Filtration

Having introduced a drainage system into a structure design, it is important to ensure it remains effective over the lifetime of the structure. Problems can arise if soil particles migrate into the drain leading to reduced flow and perhaps eventual blockage.

Soil is a structured mass of particles and movement of water can disturb the equilibrium, leading to soil instability. Dynamic seepage forces are capable of physically transporting particles. Migration increases as particle size decreases until cohesive forces bind together fine grained soils, eg clay. Well graded soils are more stable than uniform or gap graded materials. Adequate references are available to decide which soils cause filtration problems (5, 6). Broadly speaking particles in the range of fine sands to medium/coarse silts and where velocities are high enough to transport them can ultimately cause clogging.

Having decided a filter is necessary, design recommendations are available for conventional granular filters (7) which may have to be multi layer to be truly effective. An example of such a design quoted in DIN 4095 is shown (Fig 2).

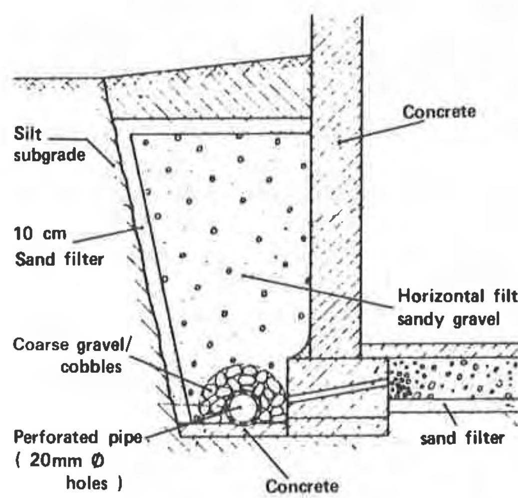


Fig. 2. EXAMPLE OF TWO STAGE FILTER DRAIN. Din 4095.

4 Conventional Structure Drainage Materials

To provide a background to fin drain property requirements it is useful to review materials which are currently being specified.

UK

The Dept of Transport specify the following permeable backing to earth retaining structures, (8).

i A minimum thickness of 300 mm of Type A material complying with Clause 505..

ii Porous no-fines concrete, cast in situ, 225 mm thick complying with the requirements of Clause 1617.

iii Precast porous concrete blocks complying with the requirements for Type B2.8 of BS 2028, 1364 laid in stretcher bond with dry joints in 225 mm thick walling.

iv When the filling adjacent to the structure is pulverised fuel ash, the permeable backing shall be a minimum thickness of 300 mm of sand with a grading within Zone 1 or 2 of Table 2, BS 882 or other approved material.

Measurements were taken on items i and iv using uncompacted materials from within the grading bands specified, using a soil/fabric permeameter (12). Mean results are shown in table 1. Items ii and iii offer higher permeabilities, but their filtration properties, particularly iii, cannot be specified.

TABLE 1. PERMEABLE BACKING TO EARTH RETAINING STRUCTURES. (U.K.)

Material	Layer Thickness	Evaluation Method	Vertical Flow Capacity	Transverse Permeability (Gradient i = 1)	D_{50}	D_{10}
	mm		L/s m	mm/s	mm	mm
Type A granular filter	300	Measured	0.21	0.7	2.2	0.4
		Specified	-	-	max 12.0 min 0.8	0.7 0.15
Zone 1 sand	300	Measured	0.12	0.4	1.2	0.2
		Specified	-	-	Zone 1 max 1.8 min 0.8	0.4 0.15
					Zone 2 max 1.0 min 0.45	0.3 0.15

W. Germany

DIN 4095 (Subsoil drainage for buildings) puts forward the following:-

i a granular filter of sandy gravel in silty sand soils;

ii as in i. but surrounded by a 10 cm layer of filter and in silty soils.

It recommends that the grading of these filter materials should follow criteria established by Terzaghi.

5 Specification Parameters for Fin Drains in Structure Drainage

Fin drains must offer sufficient hydraulic conductivity to ensure there is no increase in pressure exerted by the soil (fig 1) and possess a filter which will ensure long term maintenance of drainage performance.

a Hydraulic Conductivity - vertical

With a knowledge of the permeability of the backfill/adjacent soil and rainfall intensity it is possible to calculate the flow required within the fin drain to avoid hydrostatic pressure building up in the backfill.

Most drainage demand next to the structure will exist when excavated material is used as backfill. Unless well compacted this backfill will have a higher pore volume than when in its original state and could be fissured. Eventually, the excavated material will return to its original condition resulting in backfill and adjacent soil having similar permeabilities. This situation, especially if the soil is not of high permeability will require an outlet for water against the structure leading to either pipe and/or weep holes at the base of the structure.

If the backfill is of higher permeability than the adjacent soil, water will rapidly gravitate to the drainage outlet at the base as soon as it enters this more permeable zone. In this situation the demand on the fin drain is less critical which can be regarded as a reserve capacity to the permeable backfill.

Muth (9) puts forward drainage outflow rates (Fig 3) based on the two situations above.

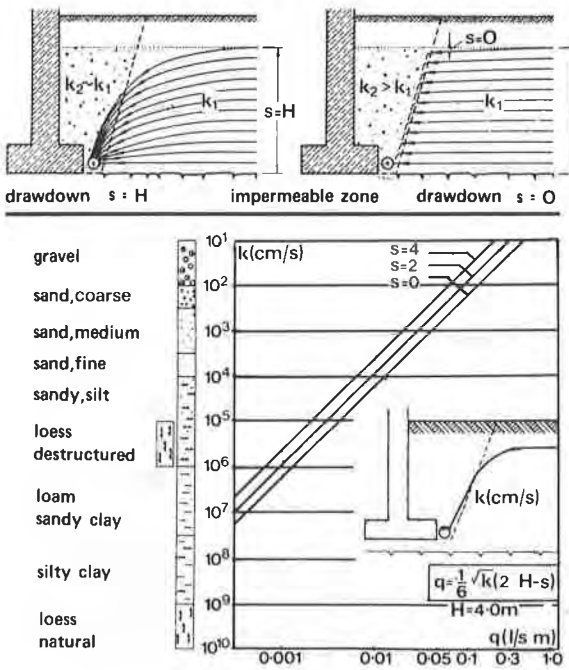


Fig. 3. DRAINAGE OUTFLOW DEPENDENT ON SOIL TYPE (MUTH).

b Hydraulic Conductivity - horizontal

Horizontal structural drainage, eg underground car parks, requires separate consideration due to the slight falls (3% - 10%) resulting in lower flow rates through the fin drain. Regular outflows into pipes are required to ensure the fin drain is emptied before it reaches full capacity.

It is important to place the fin drain on a flat base to ensure continual water flow and no ponding.

c Filtration

Considerable research has resulted in design criteria being drawn up for geotextile filters (13, 14, 15). Reference to these guidelines will enable a suitable choice of filter to be made. Knowing the particle size distribution of the backfill it is possible to choose a filter with suitable pore sizes to prevent excessive piping and with significantly higher permeability even under pressure, than that of the soil.

It is worth noting that soil is seldom a uniform homogenous material, but can be of the utmost variety with characteristics changing from natural to destructured state, from layer to layer and from one area to another. As a result filtration criteria rarely provide an exact solution, but form a useful guide.

d Compression

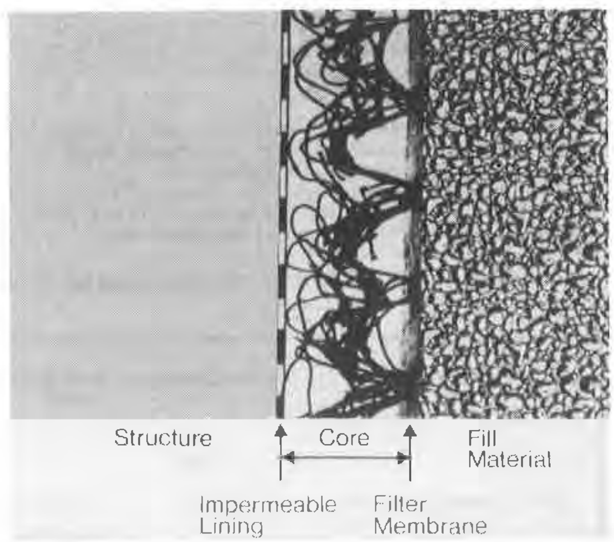
The depth at which a fin drain is installed will have a bearing on its performance, its hydraulic conductivity, and with thicker filters even its permeability being reduced as pressures increase.

Pressure on the drain will depend on the type of backfill, its compaction and moisture content. At 5 m depth backfill pressure will be in the region of 35 - 45 kN/m^2.

6 Examples of Fin Drains and their Properties

Enkadrain - manufactured by Enka BV, Netherlands.

Weight	- 750 g/m^2
Thickness	- 20 mm
Filter	- Colback, non woven, heat bonded filaments comprising copolymers nylon 6/polyester
Core	- Nylon 6 monofilament
Roll Size	- 30 m x 1.0 m
Hydraulic Capacity	- Vertical

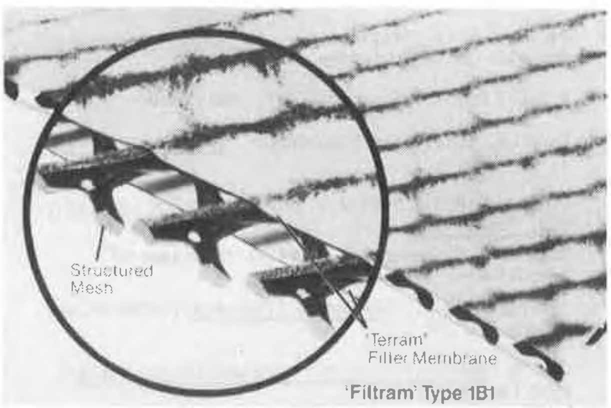


Pressure	Thickness	Installation Depth (sand)	Permeability	Hydraulic Flow (i=1)
kN/m^2	mm	m	mm/s	L/s m
0	20	0	50.0	10.4
15	9	2.5	20.0	2.8
50	5	7.0	7.0	1.1

Information extracted from published literature.

Filtram - manufactured by ICI Fibres, UK.

Weight	- 980 g/m^2
Thickness	- 4.5 mm
Filter Fabric	- 'Terram', non woven, heat bonded continuous filaments comprising polyethylene/polypropylene
Core	- Polyethylene
Roll Size	- 25 m x 1.6 m
Hydraulic Capacity	- Vertical



Pressure	Thickness	Installation Depth (sand)	Hydraulic flow (i = 1)
kN/m^2	mm	m	L/s m
0	4.5	0	0.5
50	4.3	7	0.48
100	4.1	15	0.4

Filter (quality 1B1)

Permeability (100 mm head) - 36 L/s m^2
Pore size O_{90} - 0.11 mm
O_{50} - 0.07 mm

Eljen - available from Eljen Development Corporation, USA, and comprises:-

i a flexible dimpled styrene sheet providing channels each side of 3 mm depth, sheets normally 3 m x 1.5 m;

ii a non woven thin polypropylene filter fabric;

iii a perforated plastic pipe (100 mm) inserted along the base of each section.

For lengths less than 15 m in sand and 60 m in clay the core has sufficient hydraulic conductivity to obviate the need for a pipe.

Maximum hydraulic efficiency is unimpaired providing installations do not exceed 12 m, quoting a perpendicular pressure of 95.8 kN/m^2.

Hydraulic capacity (i = 1)

0.43 L/s m 95.8 kN/m^2

Filter

Thickness < 1.0 mm
Open Area $> 10\%$
Max. pore size $\not> 0.8$ mm
$<$ 10% of the holes $>$ 0.2 mm
permeability - (50 mm head) - 30 mm/s

Information extracted from published literature.

7 Laboratory Testing Methods for Fin Drains

a Hydraulic flow

Extensive testing (9) has been undertaken in West Germany on a variety of synthetic drainage systems. One piece of testing apparatus (fig 9) consists of a wave tank with the sample under test providing a horizontal hydraulic flow path. A normal pressure can be applied to the sample. If required a particle layer can be incorporated which under compression simulates soil pressure in the filter surface.

ICI use a similar horizontal planar flow test and is developing a vertical flow apparatus.

b Filtration

To ensure the filter component of the fin drain has adequate permeability and filration properties in relation to that of the adjacent soil a range of tests have been developed.

The filter permeability must be higher than that of the soil to ensure no build up of hydrostatic pressure. It can be argued that to quote a K value for a very thin fabric ($<$ 1 mm thickness) is hardly appropriate when in reality such a value refers to flow through a mass (soil). It is more useful to determine the flow rate through the fabric. To this end the ASTM is drawing up a geotextile permittivity testing method to measure flow rates. Other permeability tests have been developed by (11) and ICI.

With regard to filtration properties the pore size distribution of the filter must be known to ensure compatibility between the fabric and adjacent soil. Wet (10,11) and dry (ICI) using static eliminators have been developed.

Apparatus is available to test the soil/fabric system. The ICI method has been reported (12) and measures the flow of the system over a period at a defined hydraulic gradient. Modifications now enable the measurement of piping to be made.

c Compression

As detailed (9) compression representing soil pressure can be applied to the fin drain during hydraulic flow testing.

Any dry test must simulate the effect of soil pressure on the filter and must be capable of identifying the remaining core space available for water conductivity. Ideally subsequent calculations must take into account the nature of the water flow at varying gradients.

8 Comment on Fin Drain Installations

A Underground Structures

A buried structure requires external waterproofing protection and an exterior drainage layer. Frequently this waterproofing is an impermeable membrane onto which a stone drainage layer has to be placed. The membrane will require protection especially during the construction period. Conventionally the drainage layer will be loose or cement bound gravel. A gravel layer on an underground roof increases the weight on the structure, which has to be taken into account in the design.

A fin drain overcomes these problems. It provides a protection for the waterproofing membrane. It obviates the need for shuttering to contain any sloping or vertical stone layer. Its lightweight is an advan-

-tage in reducing the roof loading. To this list one must add the advantages relating to handling and installation time.

B Bridge Abutments and Retaining Walls

In the UK the normal drainage layer behind abutments comprises porous concrete blocks or, when filling with PFA, a 300 mm wide layer of sand (refer section 4). The transport and handling onto site can be expensive and time consuming; their installation is equally labour intensive and costly. Likewise a sand layer poses constructional problems.

A fin drain is an attractive alternative in that it is a lightweight drainage system available in roll form. Designers recognise the advantage of being able to specify a product, which is not subject to any local limitations of material supply.

The fin drain has to be held in place prior to backfilling. Installation systems vary from suspending the material loosely from the top of the structure to using adhesive or nailing methods.

Jointing of fin drains is normally undertaken by overlapping the filter material onto the adjoining sheet. Some products are provided with a 10 - 15 cm overlap along one edge.

C Pipe Connections

The pipes used with fin drains are often plastic in keeping with the character of this drainage system ie being light in weight and easy to handle. With thin fin drains an integrated system is possible with the fin drain being inserted into a slit pipe. Another form of integration is to continue the filter fabric protecting the drainage core around a perforated or porous pipe thereby preventing contamination. Three design systems are shown below. (Fig 4)

9 Conclusion

From this brief review it can be seen that fin drains are capable of meeting the demands of structure drainage. Where conventional material possesses high hydraulic conductivity it generally lacks specific filtration properties. Fin drains are also versatile and can be modified to meet specific requirements. Being cheap to transport and internationally available they offer designers the opportunity to specify material which do not pose local site problems of availability.

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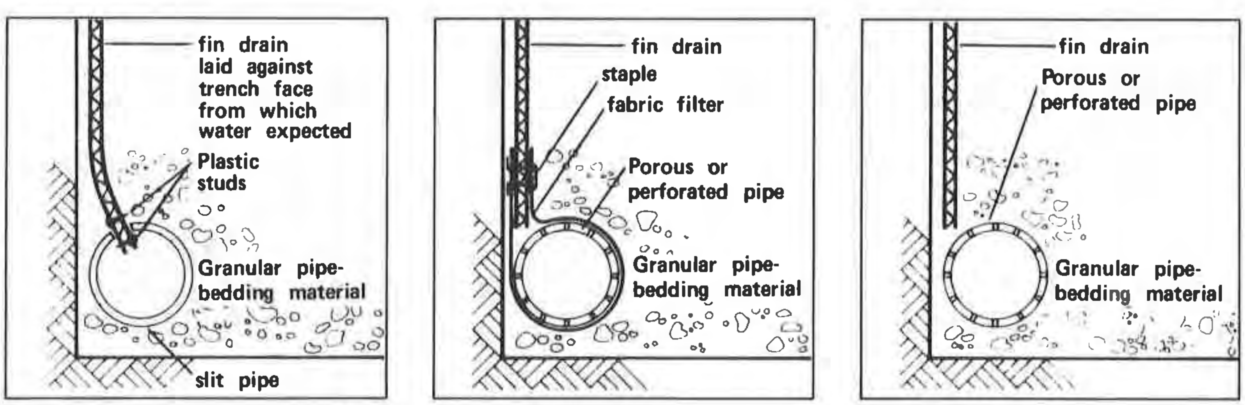

Fig. 4. EXAMPLES OF FIN DRAIN / PIPE COMBINATIONS.

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Earth Fills Consolidation Using Fabrics: Computation by Means of Homogenization Method

Consolidation des remblais au moyen de textiles: Etude par homogénéisation

Horizontal non woven fabrics periodically distributed in an earthfill increase the velocity of the consolidation process. This case has been investigated by a numerical approach [1] and in a previous paper [2] we used an analytical method for the same problem. These results are unattractive for practical purpose.

In the present paper we use the homogenization method which consists of replacing the soil-fabric system by an anisotropic equivalent material.

The water pressure at the center of the fill is given as a function of an adimensional time. This result is then compared to results presented in previous papers [1] and [2].

L'accélération de la consolidation des remblais au moyen de nappes textiles horizontales disposées périodiquement a été abordée soit numériquement [1], soit analytiquement [2]. L'utilisation des résultats ainsi obtenus est lourde dans la pratique.

Nous proposons ici, au moyen de la méthode d'homogénéisation, de remplacer le composite sol-textile par un milieu équivalent anisotrope.

Les résultats présentés sous la forme d'un graphique donnant la pression interstitielle au centre du remblai fonction d'un temps adimensionnel, permettent une utilisation immédiate. Ils sont d'autre part comparés à ceux de [1] et [2].

1. Introduction

Les textiles non-tissés peuvent être utilisés dans un remblai soit comme nappe anticontaminante, soit comme armature pour accroître les propriétés mécaniques du massif, soit enfin pour jouer le rôle de drain et accélérer ainsi la consolidation. C'est ce dernier usage qui nous intéresse ici. Les études menées dans ce domaine soit sur le plan numérique [1] soit analytiquement [2] conduisent à des résultats lourds à manipuler dans la pratique. Nous nous proposons ici de donner une méthode rapide d'évaluation de l'efficience des nappes textiles et de la consolidation des remblais, d'une part au moyen d'hypothèses simplificatrices sur la consolidation, du même type que celles faites en [1] et [2], d'autre part en remplaçant le milieu hétérogène sol-textile par un milieu équivalent au moyen de l'homogénéisation.

Nous présentons dans le paragraphe suivant le problème étudié : un remblai de sol consolidant sous son propre poids ou une surcharge placée en son sommet, où sont périodiquement disposées des nappes horizontales de textile. Nous rappelons ensuite les approches de [1] et [2] et enfin posons le problème aux limites abordé ici.

La troisième partie présente -sans démonstration- la détermination du matériau équivalent au système sol-textile. Les résultats obtenus sont partiellement classiques mais soulignons qu'ils sont obtenus ici dans un cadre plus général et sont complètement justifiés par la démarche suivie. Ils sont ensuite utilisés dans une quatrième partie pour calculer le pourcentage de consolidation fonction du temps pour le matériau équivalent. Celui-ci est donné sous forme d'une courbe d'exploitation aussi aisée que dans la théorie de TERZAGHI.

Le cinquième paragraphe aborde enfin la caractérisation de l'efficience du textile et le domaine de validité de la solution homogénéisée. Ceci est obtenu au moyen de l'introduction de deux paramètres adimensionnels. La valeur du second de ces paramètres, qui sépare le domaine de validité de la solution homogénéisée de la solution en drain parfait, est ensuite mise en évidence en comparant les résultats obtenus à ceux de [2] qui sont opérationnels pour les deux types de comportement.

2. Position du problème

Considérons un remblai (coupe représentée fig. 1) de longueur suffisante pour que le problème puisse être considéré à deux dimensions. Des nappes de textiles d'épaisseur e sont disposées horizontalement (z = constante) séparant ainsi des couches de sol d'épaisseur constante 2H. Chaque nappe peut ainsi collecter l'eau qui s'écoule du sol soit par gravité soit sous l'effet de la surcharge due aux couches de sol supérieures. La largeur du remblai est 2B mesurée suivant la variable x.

Le problème de consolidation étudié ici présente plusieurs caractéristiques importantes qui vont guider notre démarche dans la suite :

- la structure du système sol-textile est périodique suivant la verticale, la période vaut 2H + e.
- la perméabilité du textile k_T est, par construction très supérieure à celle du sol k_S :

$$\frac{k_S}{k_T} \ll 1$$

- la géométrie de la période est fortement discontinue ; l'épaisseur de la couche de sol 2H est très supérieure à celle de la nappe de textile e :

$$\frac{e}{2H} \ll 1$$

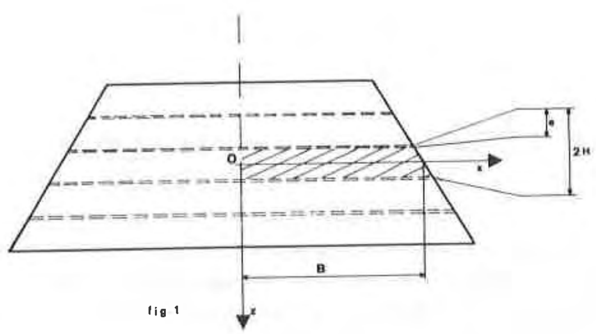

fig 1

Pour aborder le problème nous admettrons valables les hypothèses suivantes (TERZAGHI)

- la loi de comportement du sol et du textile est l'élasticité linéaire infinitésimale isotrope. Le matériau composant le sol a une compressibilité négligeable devant celle du squelette assemblage des grains.
- le liquide interstitiel (de l'eau en général) est incompressible
- les perméabilités sont considérées comme constantes et isotropes au cours du processus.

Ainsi, à tout moment, nous supposons vérifié :

$$\sigma = \sigma' + u$$

où σ est le tenseur des contraintes totales

σ' représente le tenseur des contraintes intergranulaires, indépendantes de la pression interstitielle u.

Nous appellerons $\bar{\sigma}$, $\bar{\sigma}'$ et $\bar{u}$ les accroissements de ces quantités à partir de l'état initial d'équilibre. On aura donc également :

$$\bar{\sigma} = \bar{\sigma}' + \bar{u}$$

D'autre part, le liquide filtrant vérifie la loi de DARCY classique :

$$\vec{v} = - k \,\overrightarrow{grad}\, h \quad \text{où}$$

$\vec{v}$ est la vitesse d'écoulement au sens de DARCY

k la perméabilité du matériau considéré

et h la charge hydraulique :

$$h = \frac{u}{\gamma_w} - z \qquad (\gamma_w \text{ poids volumique du liquide.})$$

Dans ces conditions les équations qui régissent la consolidation s'écrivent, en déformation plane :

$$(1) \quad \begin{cases} G\,\Delta \bar{U} + \dfrac{G}{1-2\nu}\dfrac{\partial \bar{\varepsilon}}{\partial x} - \dfrac{\partial \bar{u}}{\partial x} = 0 \\ G\,\Delta \bar{W} + \dfrac{G}{1-2\nu}\dfrac{\partial \bar{\varepsilon}}{\partial z} - \dfrac{\partial \bar{u}}{\partial z} = 0 \\ \dfrac{k}{\gamma_w}\,\Delta \bar{u} = \dfrac{\partial}{\partial t}\bar{\varepsilon} \end{cases}$$

où $\bar{U}$ et $\bar{W}$ sont les composantes du déplacement, suivant x et z

$\bar{\varepsilon} = \dfrac{\partial \bar{U}}{\partial x} + \dfrac{\partial \bar{W}}{\partial z}$ est la variation du volume,

G le module de cisaillement du squelette et ν son coefficient de poisson :

$$G = \frac{E}{2(1+\nu)} \qquad \text{avec E module d'Young.}$$

Les coefficients G , E et ν prennent les valeurs correspondantes au sol ou au textile suivant que les équations sont écrites pour le sol ou pour le textile.

Les équations (1) sont astreintes de plus à vérifier des conditions initiales : $\bar{U}$, $\bar{W}$ et $\bar{u}$ sont nuls au temps t = 0 où nous supposons commencer la consolidation. D'autre part, au niveau d'une couche, nous devons écrire la continuité des déplacements $\bar{U}$, $\bar{W}$ aux interfaces sol-textile ainsi que la continuité du flux de liquide filtrant (cf. figure 2).

$$(2) \qquad k_T \left.\frac{\partial \bar{u}}{\partial z}\right|_T = k_S \left.\frac{\partial \bar{u}}{\partial z}\right|_S \quad \text{pour } z = H$$

La symétrie pour x = 0 permet d'écrire

$$\frac{\partial \bar{u}}{\partial x} = 0 \qquad \bar{\sigma}'_{xz} = 0 \qquad \bar{\sigma}'_{xx} = 0 \qquad \text{pour } x = 0$$

$$\text{et} \qquad \bar{u} = 0 \qquad \bar{\sigma}'_{xx} = 0 \qquad \bar{\sigma}'_{xz} = 0 \qquad \text{pour } x = B$$

Enfin nous pouvons considérer, pour la couche étudiée qu'à l'instant initial t = 0 est appliquée une surcharge

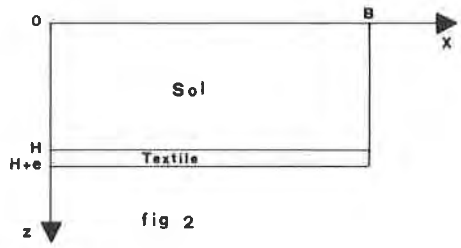

fig 2

$\overline{\sigma}_o$ correspondant au poids des couches supérieures, surcharge maintenue constante pour $t > 0$.

La résolution de (1) avec les différentes conditions initiales et aux limites mentionnées ci-dessus est délicate. Rappelons deux travaux concernant cette étude.

En [1] BOURDILLON remplace le système (1) par l'équation découplée

$$(3') \qquad \frac{k_s}{\gamma_w}\,\Delta\bar{u} = \frac{1}{K_s}\,\frac{\partial\bar{u}}{\partial t}$$

où $K_s = \dfrac{E_s}{3(1-2\nu)}$ est le coefficient de compressibilité volumique du squelette. Le textile est supposé ne pas se déformer à $t > 0$. L'équation (3) suppose que la contrainte isotrope totale varie peu en fonction de la pression interstitielle (Hypothèse de RENDULIC). Ceci permet de découpler les équations donnant les déplacements et la pression. On trouvera en [1] le traitement numérique de (3).

En [2] nous avons abordé analytiquement le problème avec des hypothèses légèrement différentes. Le remblai est supposé tel que $\frac{H}{B} \ll 1$ si bien qu'en première approximation l'écoulement dans le sol se fait suivant la verticale :

$$(4) \qquad \frac{k_s}{\gamma_w}\,\Delta\bar{u} = \frac{1}{E'_{oed\,s}}\,\frac{\partial\bar{u}}{\partial t}$$

où $E'_{oed\,s} = \dfrac{E'(1-\nu')}{(1+\nu')(1-2\nu')}$ est le module oedométrique.

Dans le textile, qui ne se déforme pas, la surpression interstitielle $\bar{u}$ supposée indépendante de z vérifie l'équation, avec les notations de la figure 2

$$k_T\, e\,\frac{\partial^2\bar{u}}{\partial x^2} = 2\,k_s\left.\frac{\partial\bar{u}}{\partial z}\right|_{s,\,z=H}.$$

La solution est alors obtenue sous la forme d'un développement en série double

$$(5) \qquad \frac{\bar{u}(x,z,t)}{\bar{\sigma}} = \sum_{n=0}^{\infty}\sum_{p=0}^{\infty}\frac{(-1)^n\,16\sin\left(H\lambda_{pn}^{1/2}\right)\cos\left(\frac{2n+1}{2B}\pi x\right)}{(2n+1)\pi\left(2H\lambda_{pn}^{1/2}+\sin\left(2H\lambda_{pn}^{1/2}\right)\right)}$$

$$\times\cos\left(\lambda_{pn}^{1/2}\,z\right)e^{-C_{vs}\lambda_{pn}t}$$

avec $C_{vs} = \dfrac{k_s\,E'_{oed\,s}}{\gamma_w}$ et les λ_{pn} solutions de

$$\lambda^{1/2}\,\mathrm{tg}\left(\lambda^{1/2}H\right) = \frac{(2n+1)^2\pi^2}{8}\,\frac{e\,k_T}{B^2 k_s}$$

La comparaison de ces résultats avec ceux de [1] est bonne. Cela tient en partie au fait que les équations (3) et (4) sont semblables : le rapport du module de compressibilité volumique au module eodométrique $\frac{K}{E'_{oed}}$ varie en effet peu en fonction du module de POISSON ν

$$\frac{K}{E'_{oed}} = \frac{1+\nu}{1-\nu}$$

Toutefois l'exploitation de (5) est lourde. Nous nous proposons ici de remplacer le système sol textile par un milieu fictif équivalent obtenu par homogénéisation, puis moyennant des hypothèses du même type que celles formulées ci-dessus, d'obtenir une solution sous une forme plus simple.

3. Recherche d'un milieu fictif équivalent

Nous nous bornerons ici à donner les grandes lignes de la méthode d'homogénéisation qui conduit (cf. [3], [4]) à la détermination d'un milieu fictif équivalent et à en donner les applications au problème du talus. La méthode s'adresse à des milieux de structure périodique quand les dimensions $O(\ell)$ de la période sont petites par rapport aux dimensions $O(L)$ du problème aux limites étudié. Dans notre cas particulier, cela revient à admettre que 2H + e est petit par rapport à la hauteur totale du remblai. L'hypothèse est donc plutôt mal vérifiée mais nous appliquerons toutefois la méthode. Celle-ci consiste à utiliser le petit paramètre $\varepsilon = \frac{\ell}{L} \ll 1$ et à chercher les inconnues $\overline{U}$, $\overline{W}$ et $\bar{u}$ périodiques de période ℓ , sous forme de développements asymptotiques en puissance de ε . Ces développements, portés dans les équations de la consolidation (1), conduisent par identification aux équations (6) vérifiées par les limites quand $\varepsilon \to 0$, des déplacements $\overline{U}$, $\overline{W}$ et de la pression $\bar{u}$. Du fait que les dérivées par rapport au temps sont petites, ces équations sont ici de même structure que (1) mais les coefficients de perméabilité et élastiques du milieu fictif ne sont plus isotropes, le milieu est orthotropique de révolution d'axe z :

$$(5) \qquad \begin{cases} \dfrac{\partial}{\partial x_\ell}\left(a_{ijk\ell_{eq}}\,\overline{\varepsilon}_{ij}\right) - \dfrac{\partial\bar{u}}{\partial x_k} = 0 \\[2ex] \dfrac{\partial}{\partial x_j}\left(k_{ij_{eq}}\dfrac{\partial}{\partial x_i}\,\bar{u}\right) = \dfrac{\partial}{\partial t}\,\overline{\varepsilon} \end{cases}$$

où $(a_{ijk\ell})_{eq}$ est un tenseur élastique équivalent, orthotrope de révolution autour de l'axe $x_3 = z$

- $(k_{ij})_{eq}$ un tenseur perméabilité équivalent, orthotrope de révolution autour du même axe et ε_{ij} le tenseur déformation appliqué aux déplacements. La perméabilité $(k_{ij})_{eq}$ a ses directions principales suivant x et z et le calcul conduit aux expressions par ailleurs bien connues

$$(7) \quad \begin{cases} k_{x\,eq} = \dfrac{e\,k_T + 2H\,k_S}{2H + e} \\[2ex] k_{z\,eq} = \dfrac{(2H+e)\,k_T\,k_S}{e\,k_S + 2H\,k_T} \end{cases}$$

Remarquons ici que l'homogénéisation de l'équation (3) ou de l'équation (4) conduit aussi à des perméabilités équivalentes données par (7).

Les coefficients $(a_{ijkl})_{eq}$ sont plus compliqués à déterminer et leur connaissance n'est pas nécessaire par la suite. Notons toutefois que le module oedométrique équivalent suivant z s'écrit :

$$(8) \quad E'_{oed\,z\,eq} = \frac{(2H+e)\,E'_{oed\,T}\,E'_{oed\,S}}{e\,E'_{oed\,S} + 2H\,E'_{oed\,T}}$$

Précisons d'autre part que (6) n'est valable que si les différents termes qui y apparaissent sont du même ordre de grandeur. Si nous appelons par convention ℓ une dimension microscopique et L une dimension macroscopique, le procédé décrit plus haut consiste à chercher une description macroscopique d'un processus microscopique plus compliqué. Ainsi les équations (1) de la consolidation sont elles-mêmes la description macroscopique d'une situation complexe à l'échelle des pores qui représente donc dans ce cas l'échelle microscopique. Les équations (1) peuvent en effet être obtenues par homogénéisation des équations de LAME écrites dans les grains du squelette et des équations de NAVIER-STOKES écrites dans le fluide interstitiel (cf. [5]).

4. Solution du problème homogénéisé

Pour résoudre le problème du remblai à partir des équations homogénéisées (6), nous faisons deux hypothèses :

- nous reprenons l'hypothèse oedométrique faite en [2] : nous négligeons les déplacements $\overline{U}$ dans la direction horizontale.

Il y a découplage des équations et la pression interstitielle est donnée par :

$$(9) \quad \frac{k_{x\,eq}}{\gamma_w}\frac{\partial^2 \overline{u}}{\partial x^2} + \frac{k_{z\,eq}}{\gamma_w}\frac{\partial^2 \overline{u}}{\partial z^2} = \frac{1}{E'_{oed\,eq}}\frac{\partial \overline{u}}{\partial t}$$

Nous pouvons admettre que $E'_{oed\,eq} \sim E'_{oed\,S}$ comme on le voit d'après (8) ; pour plus de détail on se reportera au paragraphe 5.

- les couches de sols consolident indépendamment les unes des autres, si ce n'est que la surcharge $\overline{\sigma_0}$ à l'instant $t = 0$ d'une couche donnée dépend de sa position dans le remblai : $\overline{\sigma_0}$ est en effet égal au poids des couches supérieures.

Ainsi, considérant une couche particulière (cf. fig. 3)

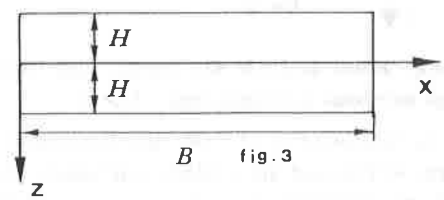

fig.3

de hauteur 2H (2H + e $\sim$ 2H), les conditions aux limites sont pour x = B et x = 0 :

$$\overline{u}(B, z, t) = 0 \qquad \frac{\partial \overline{u}}{\partial x}(0, z, t) = 0$$

Pour $z = \pm H$, la périodicité permet d'écrire :

$$\overline{u}(x, -H, t) = \overline{u}(x, +H, t)$$

D'autre part, il résulte de l'hypothèse oedométrique faite plus haut que :

$$\overline{u}(x, z, 0) = \overline{\sigma_0}$$

La solutions du problème aux limites ainsi posé s'obtient aisément par la méthode de séparation des variables :

$$(10) \quad \frac{\overline{u}}{\overline{\sigma_0}}(x, z, t) = \sum_{n=0}^{\infty} \frac{(-1)^n 4}{(2n+1)\pi} \cos\frac{(2n+1)\pi}{2B}\, x\; e^{-\frac{(2n+1)^2 \pi^2 E'_{oed\,S}\, k_{x\,eq}\, t}{4B^2 \gamma_w}}$$

dans laquelle on remarquera que la variable z n'intervient pas, tout comme la perméabilité $k_{z\,eq}$ verticale. L'équation (10) ainsi que les équations (9) et (6) est valable quand les différents termes intervenant dans (9) sont du même ordre de grandeur.

La solution se présente sous forme d'une série simple, d'accès plus aisé que la série double (5), solution fournie en [2]. Elle met de plus en évidence un temps adimensionnel

$$A = \frac{k_{x\,eq}\, E'_{oed\,S}\, t}{B^2 \gamma_w}$$

ce qui permet, dans le domaine de validité de (10) de donner le pourcentage de consolidation $\frac{\overline{u}}{\overline{\sigma_0}}$ dans l'axe du remblai en fonction du temps A pour tous les problèmes de remblais ce qui n'était pas le cas pour (5). On trouvera cette représentation sur la figure 4.

On remarquera enfin que la convergence de la série (10) est très rapide : deux ou trois termes suffisent en

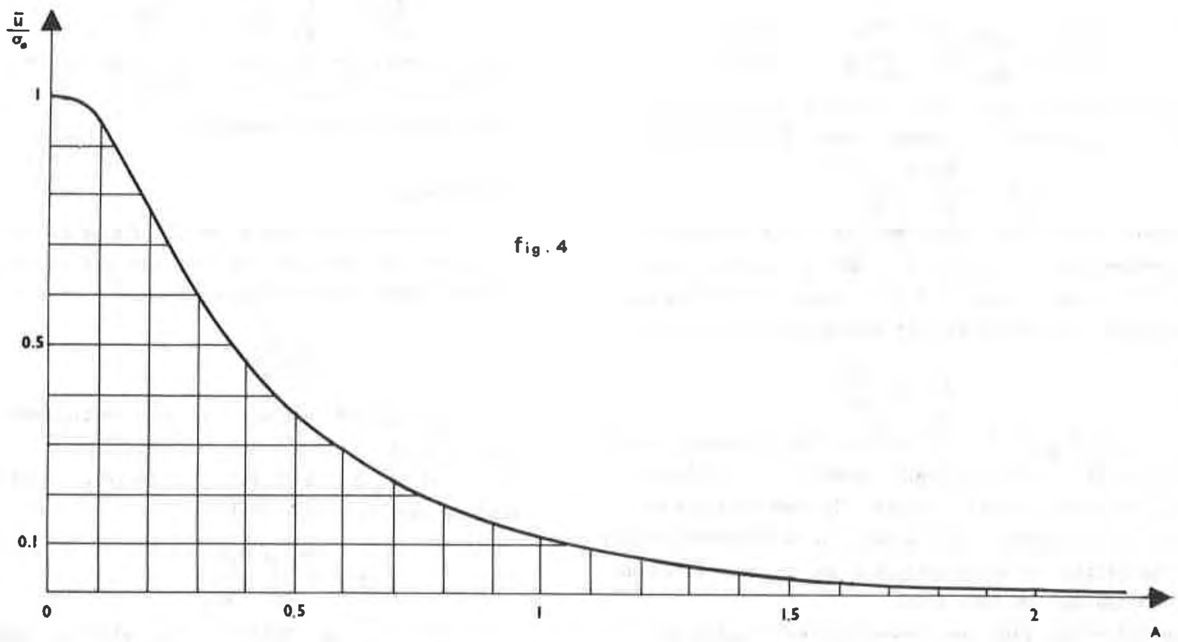

fig. 4

général pour obtenir une précision suffisante.

5. Efficience du textile - Domaine de validité de la solution homogénéisée

Comme nous l'avons indiqué ci-dessus, le milieu présente de fortes discontinuités. D'une part l'épaisseur de la nappe de textile est bien inférieure à celle de la couche de sol, ce que nous pouvons écrire :

(11) $\frac{e}{2H+e} = \eta$ où η est un petit paramètre adimensionnel.

D'autre part la perméabilité du textile est très supérieure à celle du sol. Utilisant le même paramètre η, nous écrivons :

(12) $\frac{k_T}{k_s} = \eta^{-\alpha}$ où α est un réel positif.

Pour rendre compte de l'effet de ces fortes discontinuités, portons (11) et (12) dans les expressions (7) des perméabilités équivalentes. Il vient pour η petit

$$k_{x\,eq} \sim k_s\,[\eta^{1-\alpha}+1]$$

$$k_{z\,eq} \sim k_s$$

Ainsi la perméabilité verticale est en première approximation celle du sol. Un raisonnement identique conduirait pour le module oedométrique vertical à

$$E'_{oed\,z\,eq} \sim E'_{oed\,s}$$

En ce qui concerne la perméabilité horizontale trois cas sont à distinguer.

- $0 < \alpha \ll 1$ $\quad k_{x\,eq} \sim k_s$ le textile ne joue pratiquement aucun rôle.
- α voisin de 1 $\quad k_{x\,eq} = O(k_s)$ et $k_{x\,eq} > k_s$ le textile améliore la perméabilité horizontale.
- $\alpha \gg 1$ $\quad k_{x\,eq} \gg k_s$: le textile améliore très fortement la perméabilité horizontale.

A la limite le textile se comporte comme un drain parfait. Dans ce cas on a :

(13) $$k_{x\,eq} \sim k_T\,\eta \sim k_T\,\frac{e}{2H}$$

L'efficience du textile se mesure aisément à partir de

$$\eta^{1-\alpha} = \frac{e\,k_T}{2H\,k_s} = C_1$$

si $C_1 < 1$ le textile n'est pas efficient

si $C_1 \geq 1$ le textile améliore la perméabilité ceci d'autant mieux que C_1 prend une valeur plus élevée. La validité de la solution homogénéisée (10) s'étudie à partir de (9) mise sous forme adimensionnelle en posant :

$$x = BX \,,\quad z = 2HZ \,,\quad t = \frac{B^2\,\gamma_w}{k_{x\,eq}\,E'_{oed\,s}}$$

Il vient :

$$\frac{k_{x\,eq}}{\gamma_w B^2}\,\frac{\partial^2 \bar{u}}{\partial X^2} + \frac{k_{z\,eq}}{\gamma_w 4H^2}\,\frac{\partial^2 \bar{u}}{\partial Z^2} = \frac{k_{x\,eq}}{\gamma_w B^2}\,\frac{\partial \bar{u}}{\partial T}$$

soit :

$$\frac{\partial^2 \bar{u}}{\partial X^2} + \frac{k_{z\,eq}\, B^2}{k_{x\,eq}\, 4H^2} \frac{\partial^2 \bar{u}}{\partial Z^2} = \frac{\partial \bar{u}}{\partial T}$$

La discussion se ramène donc à l'étude de l'ordre de grandeur du coefficient adimensionnel C_2 défini par :

$$C_2 = \frac{k_{x\,eq}\, 4H^2}{k_{z\,eq}\, B^2}$$

D'après les résultats précédents la limite de validité se trouve dans le domaine $C_2 \gg 1$ c'est à dire $\alpha \gg 1$ pour lequel on a (13). Compte tenu de ce que la perméabilité verticale est équivalente à celle du sol :

$$C_2 \sim \frac{k_T}{k_S} \frac{e\, 2H}{B^2} \qquad (14)$$

Ainsi, si $C_2 \leqslant 1$ la solution (10) est valable et si $C_2 \gg 1$ elle n'est plus valable : le textile se comporte alors en drain parfait : la consolidation se calcule classiquement par la solution de TERZAGHI donnant le pourcentage de consolidation d'une couche d'épaisseur 2H drainée par ses deux faces.

Pour déterminer plus précisément la limite entre les deux comportements -drain parfait ou non- , nous avons comparé graphiquement les résultats donnés par (10) et ceux donnés par (5) qui, rappelons le, sont valables pour les deux comportements. Nous avons ainsi étudié les différentes valeurs de C_2 en fonction du pourcentage de consolidation pour différents temps de consolidation (temps vrai). En effet le temps adimensionnel introduit plus haut ne peut être utilisé dans (5). La limite entre les deux comportements apparaît approximativement pour $C_2 = 0(10)$.

La valeur de C_2 donnée par (14) peut être obtenue directement par un raisonnement simpliste mais qui éclaire sa signification physique. Le débit Q_1 de liquide dans le textile par unité de longueur du remblai s'écrit, si on y suppose la pression $\bar{u}$ constante suivant z : $Q_1 = e k_T \frac{d\bar{u}}{dx}$ soit en première approximation avec $\bar{u}$ variant linéairement suivant x :

$Q_1 = e k_T \frac{\bar{u}_0}{B}$ où $\bar{u}_0$ représente la pression interstitielle au centre du remblai. Le débit Q_2 de liquide entrant dans le textile par unité de longueur du remblai provient de la filtration dans le sol ; en supposant $\bar{u}$ dans le sol indépendant de x , le débit Q_2 , vaut $Q_2 = B k_S \frac{d\bar{u}}{dz}$ soit avec la même hypothèse de linéarité :

$$Q_2 = B k_S \frac{\bar{u}_0}{2H}$$

Le rapport de ces deux débits s'écrit

$$\frac{Q_1}{Q_2} = \frac{k_T e\, 2H}{k_S B^2} = C_2$$

Quand C_2 est très supérieur à 1, le textile peut évacuer plus d'eau que ne lui en fournit le sol. Il travaille donc en drain parfait.

6. Conclusion

En résumé un remblai de sol drainé par des nappes de textile horizontales équidistantes est caractérisé par deux coefficients adimensionnels

- $$C_1 = \frac{e\, k_T}{2H\, k_S}$$

Si $C_1 < 1$ le textile ne joue pratiquement aucun rôle drainant.

Si $C_1 \geqslant 1$ le textile joue un rôle drainant qui augmente avec la valeur de C_1.

- $$C_2 = \frac{2eH}{B^2} \frac{k_T}{k_S}$$

Si $C_2 \leqslant O(10)$ le calcul du remblai se fait par la solution homogénéisée. Le pourcentage de consolidation s'obtient directement à partir du temps adimensionnel $A = \frac{k_{x\,eq}\, E'_{oed\,S}}{B^2\, \gamma_w} t$ avec

$$k_{x\,eq} = \frac{e k_T + 2H k_S}{2H + e}$$

en utilisant le graphique 4.

Si $C_2 \geqslant O(10)$ le calcul se conduit en considérant le drain parfait et on utilise les résultats de TERZAGHI

Si $C_2 = O(10)$ (en fait $1 < C_2 < 60$) il convient de procéder aux deux calculs et de comparer les résultats.

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Design of Geotextiles Associated with Geomembranes

Dimensionnement des géotextiles associés à des géomembranes

Geomembranes, used to line ponds, canals and dams, can be damaged by stresses caused by underpressures of liquids and gases or by mechanical actions. Since the first use of a geotextile associated with a geomembrane, designed in 1971 by the author, geotextiles have been increasingly used to drain liquids and gases beneath geomembranes and protect geomembranes from mechanical actions. In the first part of this paper, it is shown that geotextiles, used as filters or drains, do not have the drawbacks that granular materials, such as sand and gravel, have when they are in contact with geomembranes. In the second part it is shown that geotextiles not only prevent puncturing of geomembranes by sharp stones more efficiently than a sand layer would do, but also protect geomembranes from a large variety of mechanical actions. Several examples are presented to illustrate detrimental effects of underpressures and mechanical actions, and the use of geotextiles to alleviate these effects. Methods of design are outlined and design examples are given.

Les géomembranes, utilisées pour l'étanchéité des bassins, canaux et barrages, peuvent être endommagées par des sous-pressions de liquides ou gaz ou par des actions mécaniques. Depuis la première application de ce type, conçue en 1971 par l'auteur, des géotextiles ont été utilisés pour drainer liquides et gaz sous les géomembranes et protéger les géomembranes des actions mécaniques. Dans la première partie, on montre que les géotextiles, utilisés comme filtres ou drains n'ont pas les inconvénients présentés par les matériaux granulaires, sable et gravier, lorsqu'ils sont en contact avec des géomembranes. Dans la deuxième partie, on voit que les géotextiles non seulement sont plus efficaces qu'une couche de sable pour empêcher la perforation des géomembranes par les cailloux, mais aussi protègent les géomembranes d'une grande variété d'actions mécaniques. De nombreux exemples illustrent les effets néfastes des sous-pressions et des actions mécaniques, ainsi que l'utilisation des géotextiles pour atténuer ces effets. Des méthodes de dimensionnement sont présentées avec quelques exemples.

INTRODUCTION

Geomembranes (synthetic flexible impervious liners) are increasingly used to line ponds, canals and dams (hereafter generically called reservoirs). Problems can occur for several reasons including: (i) accumulation of liquid or gas beneath the geomembrane; and (ii) susceptibility of geomembranes to damage caused by mechanical stresses. Sometimes, problems apparently minor lead to catastrophic failures (Fig. 1).

Fig. 1 Dike failure due a sinkhole caused by water leaking from the pond.

Geotextiles can help solve the problems by draining liquid and gas beneath the geomembrane and protecting the geomembrane from mechanical stresses. The association of geotextiles and geomembranes is a natural one. Both materials are easy to install together due to their flexibility, and they have complementary properties: geotextiles provide strength and geomembranes provide imperviousness. First applications of the geotextile-geomembrane technique were made in 1971, in a series of ponds, as recommended by the author (1). Since then, geotextiles and geomembranes have been used together in hundreds of projects.

In this paper geomembrane problems solved using geotextiles are presented and methods of design are outlined.

1. LIQUID AND GAS DRAINAGE BENEATH GEOMEMBRANES

1.1 Presentation of the problem

Origin of liquids and gases. The origin of liquids accumulating under a geomembrane can be: (i) liquid from the reservoir, either leaking through the liner or overtopping the dike; (ii) water from the dike or the surrounding soil (groundwater, springs, precipitations seeping through dike, condensation of vapor). The origin of gases accumulating under a geomembrane can be : (i) gases generated by organic or polluted soil; (ii) air trappped beneath the geomembrane during installation; and (iii) air contained in the soil being forced to the surface by a rising groundwater table.

Fig. 2 Detrimental effects of fluids: (a, b) when water is pumped from the ditch, the geomembrane moves inward, pushed by the pressure of water entrapped underneath; (c) geomembrane uplifted by gas; (d) bank deformed by wave action after the dike has been softened by water leaking from the pond (in the middle of the bank, a pipe is protruding under the geomembrane); (e) the sand layer has been displaced by wave action and stones are in contact with the geomembrane; (f) slide under the geomembrane; (g) localized subsidence due to piping.

Fig. 3 Use of geotextiles in drainage systems associated with geomembranes: (a) geotextile being sewn to form a sleeve acting as a filter around a flexible perforated pipe used for draining gases; (b) plastic net (black, on the right of the photograph) acting as a drain, placed on a nonwoven filter (grey, on the left) and eventually covered by a geomembrane (roll in the background).

Detrimental effects of fluids. Direct effects on the geomembrane: (i) during installation, water from the soil, especially by condensation, can moisten the geomembrane, making seaming difficult; (ii) pressure of the accumulated fluids can lift the geomembrane, generating stresses in the geomembrane and disturbing the operation of the reservoir (Fig. 2a, b and c). Effects of excess water on the supporting ground include: (i) reduction of the strength of the soil thus lowering its resistance to wave action (Fig. 2d and e) or generating slides (Fig. 2f), both phenomena resulting in deformation of the bank; (ii) solution of salts (gypsum, etc.), erosion generated sinkhole (in karst) (Fig. 1), structure collapsing (in loess) or piping (in poorly filtered dikes) (Fig. 2g), inducing localized subsidence; and (iii) contamination of either soil or groundwater if the impounded liquid contains pollutants. All the above mentioned effects (except contamination) result in stresses in the geomembrane. Drainage is therefore necessary to prevent damage caused by stresses to the geomembrane.

1.2 Drainage beneath geomembranes

Materials used in drainage systems. There are two components in a drainage system: a drain, to convey the fluid, and a filter, to prevent clogging of the drain by the soil. Traditional drains are made of gravel or pipes (sometimes sand, when the discharge of water or gas is low); traditional filters are made of sand.

Under a geomembrane, rather thin layers of sand and gravel are often used to form large drainage blankets. However, sand layers and, to a lesser extent, gravel layers have many drawbacks: they are difficult to install on slopes; they are unstable on steep slopes (and not easy to stabilize, see Fig. 4a); they can be disturbed during construction by workers and bad weather conditions (such as wind); they can be eroded by water running under the geomembrane. Pipes also have drawbacks: they can protrude and harm the geomembrane if poorly installed or if the slope is deformed by wave action (Fig. 2d); they can originate piping if they are crushed or if their extremity is not properly plugged.

Geotextiles do not have the above mentioned drawbacks and are, therefore, increasingly used in the drainage systems associated with geomembranes where they act either as filters or as drains (Fig. 3). Geotextiles used as filters are the wovens and the nonwovens; geotextiles used as drains are the thick needlepunched nonwovens, the mats and the nets.

Design of geotextiles for drainage. Geotextiles used as filters must fulfill the filter criteria presented in (2). Geotextiles used as drains must have the transmissivity required by Darcy's formula:

$$Q/B = \theta\ i \tag{1}$$

where: Q = discharge to be conveyed (m^3/s); B = width of geotextile (m); θ = transmissivity of the geotextile (m^2/s); and i = gradient (dimensionless).

Typical values of hydraulic transmissivity are: $\theta_w = 10^{-7}$ to $10^{-6} m^2/s$ for thick needlepunched nonwovens, and $\theta_w = 10^{-4} m^2/s$ for nets. Transmissivity for a gas is derived from the hydraulic transmissivity by:

$$\theta_g = \eta_w\ \rho_g\ \theta_w/(\eta_g\ \rho_w) \tag{2}$$

where: η_w and η_g = water and gas viscosities respectively; ρ_w and ρ_g = water and gas densities respectively (Note: for air, $\theta_g = 0.07\theta_w$).

To determine the required transmissivity of a geotextile, the discharge to be conveyed and the gradient must be evaluated. Evaluation of the discharge depends on the origin of the fluid, such as water leaking through the geomembrane or flowing from the soil, gases emanating from the soil or air entrapped under the geomembrane (an example is presented in Appendix 1). The gradient depends on the pressure of the fluid and the geometry of the drain. The simplest case is the drainage of liquids on a downward slope by gravity where the pressure is equal to zero and the gradient is:

$$i = \sin\beta \tag{3}$$

where: β = slope.

Drainage of gases (because gravity is negligible) and drainage of liquids on the quasi-horizontal bottom of the reservoir (because the slope is negligible) require pressure and, therefore, evaluation of the gradient is more difficult.

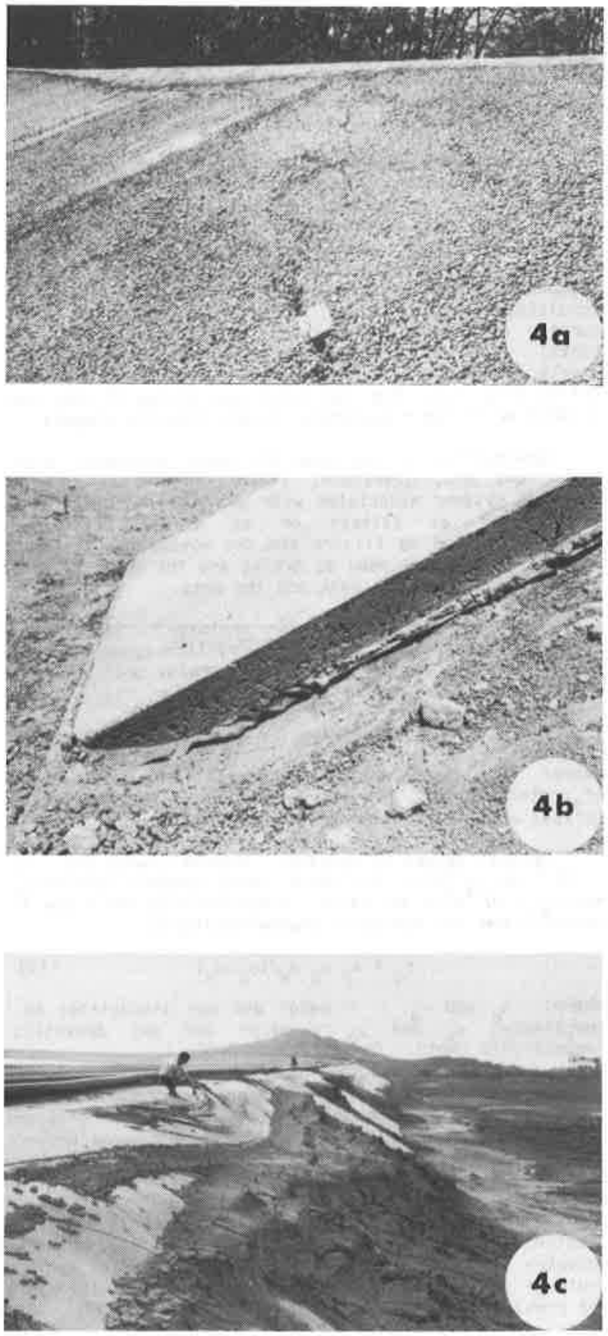

Fig. 4 Phenomena likely to damage a geomembrane by mechanical actions: (a) crack in a cement stabilized aggregate drainage layer (the geomembrane could burst through such a crack); (b) geomembrane wrinkled during the placement of fresh concrete in a full scale test; (c) geomembrane stretched and pulled out from its anchor trench by sediments having slid during a rapid draw-down of the reservoir.

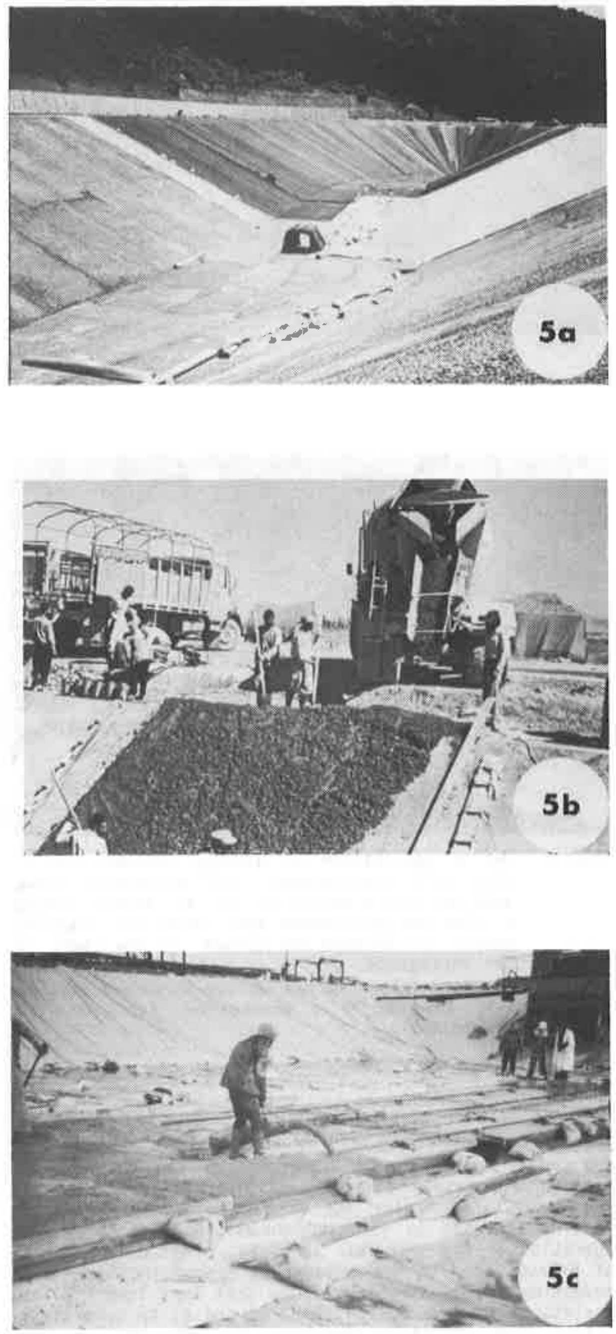

Fig. 5 Use of geotextiles to protect geomembranes: (a) geomembrane (black) placed on a geotextile (white) on a cracked cement stabilized aggregate drainage layer (see Fig. 4a); (b) concrete being poured on a geotextile protecting a geomembrane used to line a canal; (c) concrete being placed on a geotextile (white) protecting a geomembrane (grey).

2. PROTECTION OF GEOMEMBRANES AGAINST MECHANICAL ACTIONS

2.1 Presentation of the problem

Geomembranes can be subjected to various mechanical actions likely to have detrimental effects.

Distributed normal stresses. A geomembrane can be uplifted by liquids or gases accumulated underneath (as mentioned in Section 1.1) or be pushed into holes of the supporting material by the pressure of the impounded liquid. Holes can be either depressions (caused by deformation of the bank or localized subsidence, as explained in Section 1.1), or cracks in concrete (Fig. 4a). When uplifted or pushed into a hole, a geomembrane is stretched and eventually will burst.

Distributed tangential stresses. A geomembrane can be subjected to tensile stresses as a result of differential movements between the geomembrane and the adjacent materials. Such differential movements can be caused by: sliding of a concrete cover during its installation (Fig. 4b) or after its installation; sliding of an earth cover or of sediments during drawdown of the reservoir (Fig. 4c); differential settlements of the supporting subsurface; differential thermal expansion between the geomembrane and adjacent materials.

Concentrated stresses. A geomembrane can be punctured by falling objects, floating debris or ice (when it is not protected by a cover), sharp stones (either from supporting soil (Fig. 2e) or from cover material), sharp edges of concrete (either cracked concrete used as a supporting material or as a cover, or concrete plates used as a cover), vegetation (growing either under the geomembrane or in the earth cover if any). A geomembrane can be snagged by materials in contact which exhibit tangential movements as explained above. A geomembrane can be torn by animals or by ice adhering to it.

Repeated stresses. A geomembrane can be: (i) abraded by solid particles in suspension in the impounded liquid or by repeated tangential movements of the materials in contact; and (ii) subjected to fatigue by repeated actions of waves and turbulences generated by wind or agitators.

2.2 Protection of geomembranes

Methods of protection. Traditional methods used to protect geomembranes address almost exclusively problems generated by concentrated stresses. Traditional methods include: (i) preparation of support by eliminating protuberant stones and/or installing a sand layer, filling cracks in clay or concrete and eliminating angular shapes of concrete structures; and (ii) installation of a protective cover on the geomembrane (earth, concrete slab or blocks, etc.). Traditional methods have numerous drawbacks: elimination of stones, cracks, etc. is time consuming; a sand layer has the drawbacks listed in Section 1.2; installation of a protective cover may generate stresses (concentrated or distributed) harmful to the geomembrane (see Section 2.1).

Geotextiles are easy to install and do not have the above mentioned drawbacks. Depending on the considered case, geotextiles can perform various functions to protect geomembranes against mechanical actions: (i) a geotextile can act as a drain or a filter, as mentioned in Section 1.2, to drain liquid and gases and, therefore, prevent stresses generated by geomembrane uplifting; (ii) a geotextile can act as a tensioned membrane to support a geomembrane pushed into a hole by water pressure (Fig. 5a); (iii) a geotextile can act simultaneously as a reinforcement and a drain to provide additional shear strength to fresh concrete and drain the excess water expelled from the concrete during its placement in order to prevent this water from making the geomembrane surface slippery (Fig. 5b); (iv) a geotextile, with a low coefficient of friction, can act as a lubricator between a geomembrane and a concrete slab to facilitate differential tangential movements between a geomembrane and a concrete slab (Fig. 5c); and (v) a geotextile can act as an absorber to protect a geomembrane from concentrated stresses.

To date protection of geomembranes has been done mostly through the use of nonwoven geotextiles, in particular needlepunched. In fact, various types of geotextiles could be used, depending on the function to be performed: wovens and nonwovens (tensioned membrane); needlepunched nonwovens (reinforcement-drain); wovens and nonwovens, especially heatbonded (lubricator); nets with small openings and thick needlepunched nonwovens (absorbers).

Design of geotextiles for protection. The use of a geotextile acting as a tensioned membrane to bridge a hole under a geomembrane can be designed using the method presented in Appendix 2. The required strength of the geotextile increases if the water pressure and the diameter of the hole (or the width of the crack) increase, and/or if the allowable deflection (or elongation) of the geomembrane decreases.

An upper boundary of the required strength of a geotextile acting as a reinforcement for fresh concrete can be derived by writing and equation stating that the shear stress generated by the weight of the concrete is fully transmitted to the geotextile (assuming that the geotextile is properly anchored at the top of the slope). If the geotextile also acts as a drain, there is some friction on the slope and the tensile stress in the geotextile is reduced.

The design of a geotextile acting as a lubricator between a geomembrane and a concrete slab is extremely complex because tensile stress in the geotextile varies from one point to another. A detailed theoretical analysis has been made by the author and used for the design of a large canal. With friction coefficients of 0.7 (geomembrane/geotextile) and 0.5 (geotextile/concrete slab), the calculations show that, for a given differential tangential movement, the tensile stress in the geomembrane is five times smaller with than without a geotextile.

The use of a geotextile acting as an absorber to protect a geomembrane from concentrated stresses is not prone to theoretical analysis. A test, initially developed by the U.S. Bureau of Reclamation (3), consists of applying a controlled pressure over a geomembrane placed on stones. The same test, performed on a geotextile-geomembrane association shows a marked increase of puncture resistance provided by the geotextile (4). This is confirmed by the results of impact tests conducted with various objects falling on a geotextile-geomembrane association.

ACKNOWLEDGEMENTS

Photographs presented in this paper are from projects in which the author was involved either as a designer or as an expert witness. The author is indebted to J. S. Goldstein for valuable comments.

APPENDIX 1: DRAIN DESIGN

Let us consider a geotextile acting as a drain under a geomembrane on a slope. From a theoretical analysis conducted by the author for a project where leakage control was important, the required transmissivity θ of the geotextile (related to the discharge Q through a unit width B of geomembrane and the slope β, according to Eq. 1 and Eq. 3) is given by:

$$\theta = Q/(B \sin\beta) = \{k_m/(2\ T_m) + \pi\ \rho_w\ g\ n\ d^4/(256\ \eta_w\ T_m) + 0.146\ \pi\ ND^2\ \sqrt{g/z}\}\ (z/\sin\beta)^2 \qquad (4)$$

where: k_m = hydraulic conductivity of the geomembrane (m/s); z = depth under water level (m); T_m = thickness of geomembrane (m); β = slope (o); ρ_w = density of liquid (kg/m^3); g = gravity 9.81 m/s^2; n = number of pinholes per unit area (m^{-2}); d = diameter of pinholes (m); η_w = viscosity of liquid (kg/ms); 0.146 = numerical coefficient valid only in SI system; N = number of holes per unit area (m^{-2}); and D = diameter of holes.

For a given application, the curve (obtained using Eq. 4) giving the required transmissivity as a function of depth z must be compared to curves (obtained from tests) giving the transmissivities of one, two, etc. layers of the considered geotextile as a function of pressure (Fig. 6).

APPENDIX 2: TENSIONED MEMBRANE DESIGN

Using the tensioned membrane theory (5), a chart (Fig. 7) has been established by the author, giving a relationship between the following parameters: b = width of the crack (m); y = deflection of the geotextile-geomembrane (m); p = pressure of the liquid in the reservoir (N/m^2); ε = elongation of the geotextile-geomembrane; and α = force per unit width of the geotextile-geomembrane. A similar chart has been obtained in the case of a circular hole and a more complex chart in the case when the geotextile-geomembrane is not perfectly anchored at the edges of the crack and can slide. To use the chart, the (α,ε) curve of the considered geotextile must be drawn on the chart. Example: for a water pressure of 600 kN/m^2 (approximately 60 m of water) and a crack width of 5 cm (hence pb = 30 kN/m), the geotextile A would burst, the gotextile B would have an elongation larger than the geotextile C but would be subjected to a smaller tensile force.

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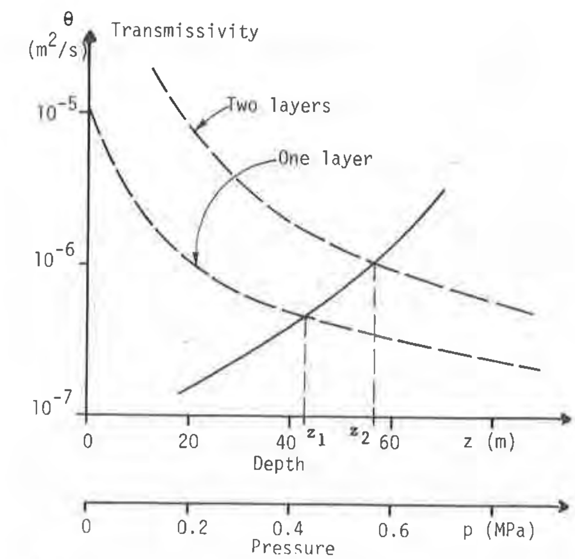

Fig. 6 Required transmissivity (solid line, from Eq. 4) compared to transmissivity of the considered geotextile (dashed curves, from tests). One layer of geotextile is required between depth 0 and z_1, and two layers between z_1 and z_2.

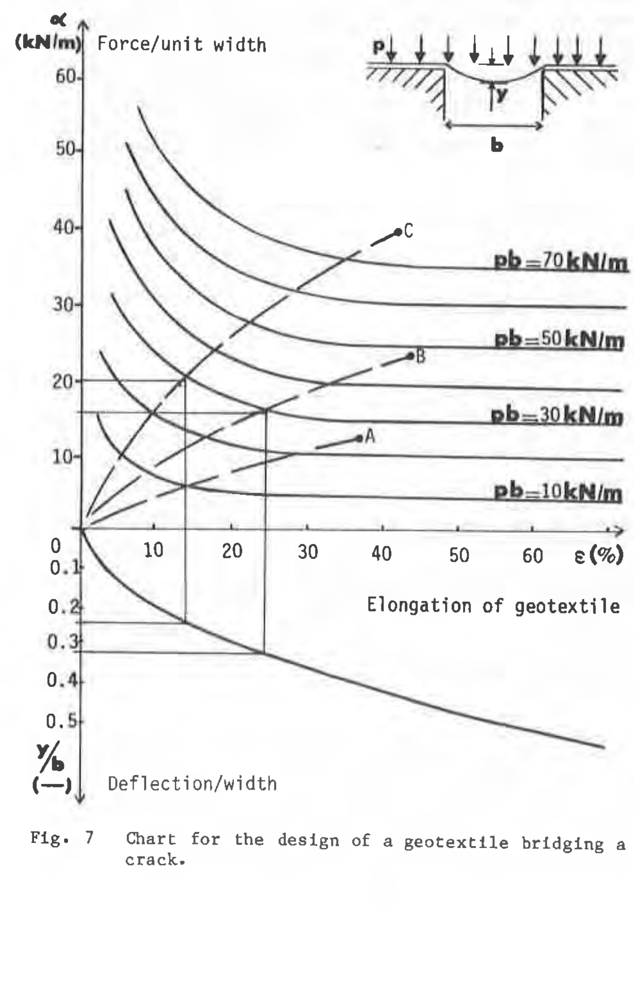

Fig. 7 Chart for the design of a geotextile bridging a crack.

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Behavior of Geotextiles as Filters Under Dynamic and Static Loadings

Conservation de la fonction filtre des géotextiles sous sollicitations dynamiques et statiques

When setting on a geotextile filter as a protection for a drain, this fabric is subjected to dynamical forces and afterwards, static compression loads by the upper soil layer with a quasi-steady flow.

Experiments were performed in order to reproduce critical conditions for the geotextile. Non woven heat bonded and needled punched geotextiles were used with a fine silicon dioxide powder.

From these experiments, the part of the initial moisture content at the time of the compaction operation is found to be very important. The filtration diameter, which is related to the size d_{98} of the passing soil depends strongly upon the type of loading : dynamic or static. Thus, these two tests appear to be usefully complementary.

The permeability measurements and the processing control of the passing soil lead us to conclude that, for the soils which were utilized, the tested geotextiles behave as good filters, and that the clogging up is avoided.

Le filtre géotextile utilisé comme protection de drain subit, lors de sa mise en place, des sollicitations dynamiques, et est soumis par la suite à la compression statique des terres susjacentes avec un écoulement quasi-permanent

Des essais de laboratoire ont été réalisés reproduisant ces sollicitations dans des conditions sévères pour le géotextile. Ils ont été conduits sur des non tissés thermoliés et non tissés aiguilletés avec un sol non cohérent très fin.

Ces essais ont permis de mettre en évidence le rôle de la teneur en eau lors de la phase de compactage. Le diamètre de filtration, correspondant au d_{98} du passat dépend fortement du type de sollicitation : dynamique ou statique. On montre ainsi la complémentarité des deux types d'essais.

Enfin ces essais permettent de conclure, à la suite des mesures de perméabilité et du contrôle des passats à la bonne conservation de la fonction filtre et au non colmatage des géotextiles considérés avec les sols testés.

INTRODUCTION

Un geotextile utilisé comme filtre dans une tranchee drainante doit permettre le passage de l'eau vers le drain tout en empêchant le transfert de particules du sol. Il est important de s'assurer que cette fonction filtre est conservee lors des deux phases principales de la vie du géotextile :

- la mise en place : le géotextile est soumis à des sollicitations dynamiques (compactage du sol),
- en service : le geotextile est comprimé statiquement et est soumis à un regime hydraulique quasi-permanent.

Afin d'etudier ces deux aspects, des simulations en laboratoire ont été realisees conjointement à I T F - LYON (essais dynamiques) et a l'Université de Grenoble (essais statiques) sur des eprouvettes sol-géotextile-drain identiques :

- le sol : de la silice broyée en poudre donc un materiau non coherent. Deux granulometries ont été retenues fig. 1. Toutes deux présentent un d_{95} analogue, 125 µm, mais le sol 1 a une granulométrie très etalée : son coefficient d'uniformité est supérieur à 20 alors qu'il n'est que de 1,5 pour le sol 2, très uniforme.
- le geotextile : un nontissé. Deux structures de masse surfacique semblable ont été testées :
TP, un thermolie TYPAR 3807, μ = 270 g/m2,
BD, un aiguilleté BIDIM U 34, μ = 280 g/m2.
- le drain : des billes de verre de 10 mm de diamètre.

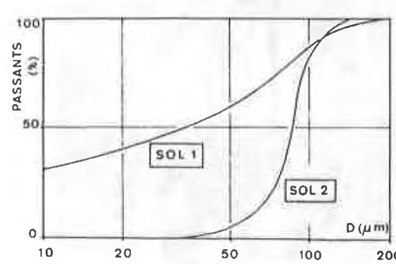

Fig. 1 - Courbes granulometriques des sols utilisés

1 ESSAIS DYNAMIQUES (Simulation de la mise en place)

1.1 Dispositif experimental

On utilise une cellule d'essais cylindrique de 185 mm de diamètre, fig. 2, où le géotextile, posé sur les billes de verre est pince à la périphérie. Le sol est prepare à une teneur en eau initiale W_o donnée et est "precompacté" a la mise en place sur le géotextile.

L'effort de compression dynamique est appliqué au moyen d'un dispositif mecanique conçu à cet effet, fig. 3. Il permet de laisser tomber une masse m d'une hauteur h, maximum de 1 m, avec une periodicité minimum de 4 s entre deux chocs. Les conditions choisies pour toutes les experimentations sont les suivantes :

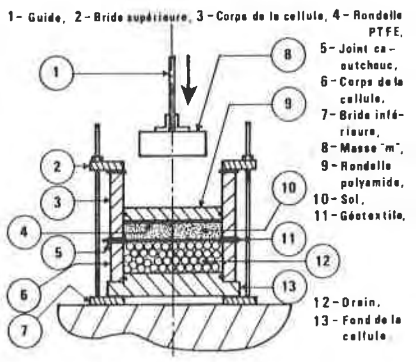

Fig. 2 - Cellule d'essais dynamiques

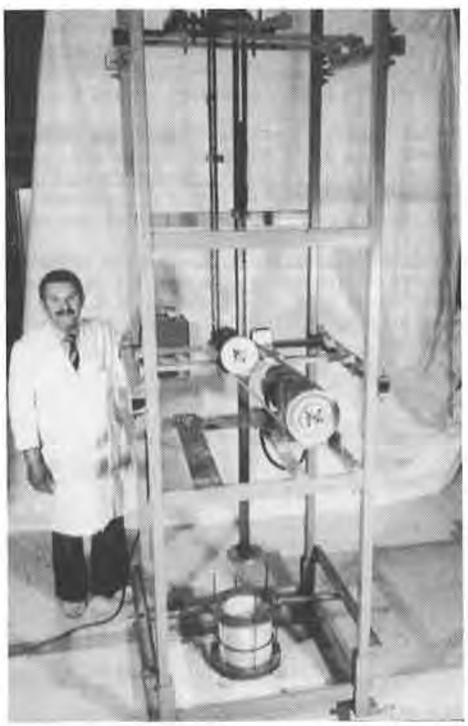

Fig. 3 - Dispositif de compactage

m = 8,9 kg,
h = 1 m,
periode : 10 s,
nombre de coups N = 100.

En faisant l'hypothèse de frottements négligeables, l'energie totale appliquée sur 1,5 kg de sol 1 ou 1 kg de sol 2 est de 325 kJ/m2. A titre indicatif, l'energie appliquee à 1 kg de sol de densité 2 lors d'un essai de compactage Proctor Normal est de 36 kJ/m2 et de 166 kJ/m2 au Proctor Modifié.
En cours d'essai, on mesure le tassement du sol. Après compactage, la teneur en eau à la surface du sol en son milieu et au contact du geotextile est déterminée par deux prelèvements à chaque niveau. Les billes du drain sont lavees, l'eau est conservee et évaporée à l'etuve ce qui permet d'obtenir la fraction de sol passante (soit M1 la masse correspondante par unité de surface de textile).
Dans certains cas, on effectue un essai identique, mais sans prelèvement en fin de compactage : on fait alors une mesure de perméabilité sur l'ensemble sol-geotextile.

1.2 Précautions expérimentales

La reproductibilité des essais est conditionnée à un mode opératoire constant et le plus rigoureux possible. En particulier, il est important de noter les points suivants :
- l'arrangement des billes dans le fond de la cellule, est le plus compact possible, leur nombre est toujours le même,
- l'eprouvette de géotextile est pesée après conditionnement a 20° C et 65 % d'humidité relative,
- de la graisse au siliconne est étendue sur la paroi interne de la cellule afin de diminuer les frottements du sol sur cette même paroi au cours du compactage,
- le sol d'une teneur en eau donnée subit un "pré-compactage" qui donne l'état initial du système. Il est obtenu par une première chute de la masse,
- quand le geotextile est retiré, il est séché et les particules accumulées en surnombre a la surface de contact avec le sol sont enlevees en retournant delicatement l'échantillon. Puis il est conditionné et pesé. Par différence avec son poids initial, on obtient la masse piégee par unité de surface M_2.

1.3 Résultats

Un arrangement de billes de même diamètre ne subit pas de tassement notable ; par contre, le sol voit son epaisseur b décroître avec le nombre de coups.

Les mesures de tassement relatif Δb/b (en %) en fonction du nombre de coups N ont eté portées sur les figures 4 et 5.

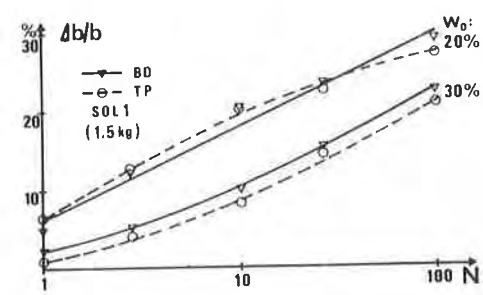

Fig. 4 - Tassement relatif du sol 1 en fonction du nombre de coups

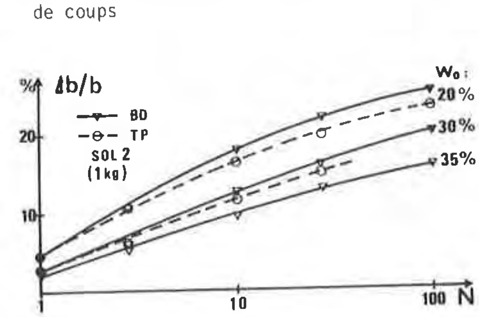

Fig. 5 - Tassement relatif du sol 2 en fonction du nombre de coups

On peut remarquer que :
- pour un sol, un géotextile et un nombre de coups donnés, le tassement est moindre aux teneurs en eau initiales W_o elevées : le sol est proche de la saturation en fin d'essai et les teneurs en eau initiales sont supérieures à celle donnant l'optimum de compacité pour l'énergie appliquee.
- pour un sol et une teneur en eau donnés, le géotextile n'a pas d'influence significative sur le tassement relatif.

Les mesures de teneur en eau effectuées en fin d'essai ont eté portées sur la figure 6. On constate que :
- les teneurs en eau finales sont inférieures à W_o : il y a donc évacuation d'eau. Ce phénomène est d'autant plus important que la teneur en eau est élevée.

Des essais complementaires, sur sol 2 ont été effectués avec un geotextile tissé, très ouvert (trous de 1 mm2). On peut observer, fig. 6, que les teneurs en eau mesurees après compactage sont plus élevées que pour les nontissés. Cela peut s'expliquer, puisque le passat est très important, par le fait qu'une partie de l'énergie de compactage a eté perdue dans l'extrusion du sol au travers du tissé. On remarque aussi une accumulation d'eau au bas du gateau. Ce type de géotextile ne pouvant être veritablement "colmaté", l'excès d'eau, au niveau de tous les geotextiles, ne peut donc pas être attribue a un éventuel colmatage mais est dû à l'action conjuguee :
- des efforts de compressions dynamiques,
- de la teneur en eau,
- des phénomènes de tension superficielle au sein du sol.

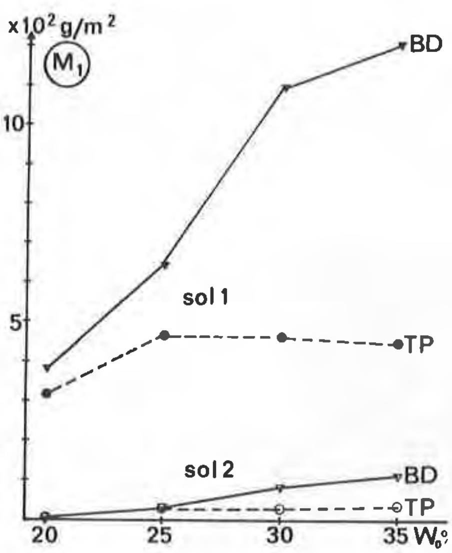

Fig. 7 - Passat en fonction de la teneur initiale w_o

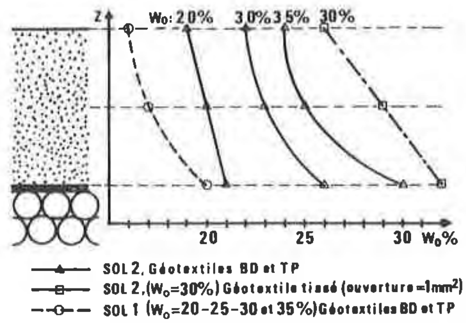

Fig. 6 - Teneur en eau du sol après 100 coups

a - Etude du passat

On a porté sur la figure 7 la masse de sol, par unité de surface, qui a traversé chacun des deux types de géotextiles nontissés en fonction de la teneur en eau initiale W_o :
- les quantités de passat augmentent avec W_o et comme il en est de même pour les quantités d'eau, on peut conclure que le passage de l'eau accroît celui du sol.
- le passat M_1 est plus important avec l'aiguilleté que le thermolié de même masse surfacique,
- des analyses granulometriques et sédimentométriques effectuées sur les passats, figures 8 et 9, montrent des granulométries plus fines que le sol d'origine : les fines passent plus facilement que les grosses particules, ce qui explique que les masses de passat du sol 1 soient plus grandes que celles du sol 2.
- Lorsque W_o est élevee, les fines du sol 1 traversent plus facilement (pour W_o = 30 % le sol 1 est saturé au premier coup), pour le sol 2, la fraction fine étant moins importante, le rôle de W_o n'est pas mis en évidence.

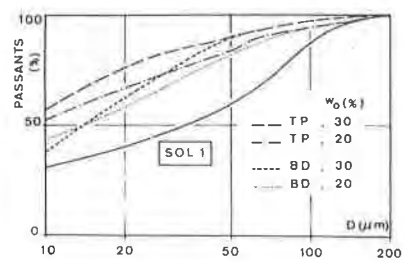

Fig. 8 - Granulométrie des passats du sol 1 sous sollicitations dynamiques

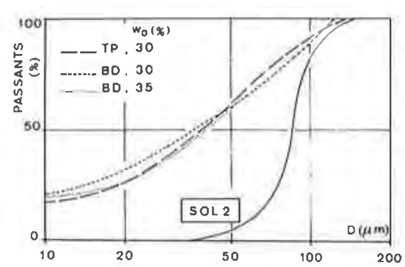

Fig. 9 - Granulométrie des passats du sol 2 sous sollicitations dynamiques

- FAYOUX (1) a déterminé un diamètre de filtration de 125 µm pour BD, quant a TP, une mesure porométrique par tamisage donne un O_{98} de 85 µm. Or on constate que les sollicitations dynamiques permettent le transfert de particules supérieures à 150 µm pour BD et TP. Des essais complémentaires realisés avec un sol de granulometrie plus grosse devraient permettre de déterminer le diametre maximum du passat en fonction de l'énergie appliquée.

b - Evaluation du risque de colmatage

Etude du sol piégé dans le géotextile : sur la figure 10, la quantité de sol piégée par unité de surface M_2 est representée en fonction de W_o :
- un géotextile donne renferme plus de sol 1 que de sol 2 ; on peut donc penser que ce sont surtout les particules fines qui sont retenues, mais malheureusement, nous ne disposons pas de courbe granulométrique nous indiquant quelle fraction du sol est plus facilement piegee.
- Le nontissé aiguilleté piège davantage le sol que le thermolie ; celà est dû à sa porosité et à son épaisseur plus grandes. En effet, si on rapporte la masse piégée à l'unité de volume des vides, on constate alors que cette grandeur est sensiblement la même pour les deux types de géotextile, figure 11.

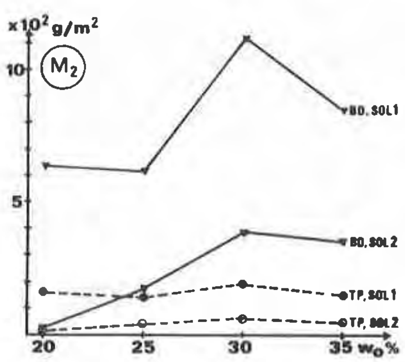

Fig. 11 - Piégé en fonction de la teneur initiale w_0

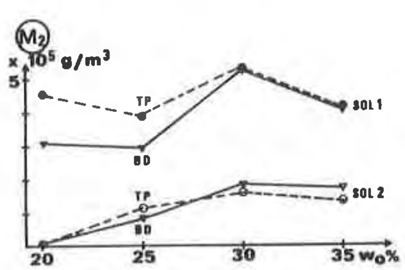

Fig. 10 - Piégé par unité de volume des vides en fonction de w_0

Mesure de la perméabilité : la cellule est retournee puis reliée à un réservoir d'eau, l'ensemble constituant un permeamètre à charge constante, figure 12. Compte tenu des faibles vitesses d'écoulement, les pertes de charge propres à l'appareillage sont négligeables devant celles que l'on peut mesurer pour le sol compacté. Les valeurs de perméabilité (tableau 1) montrent que la permeabilité globale mesurée est celle du sol seul.

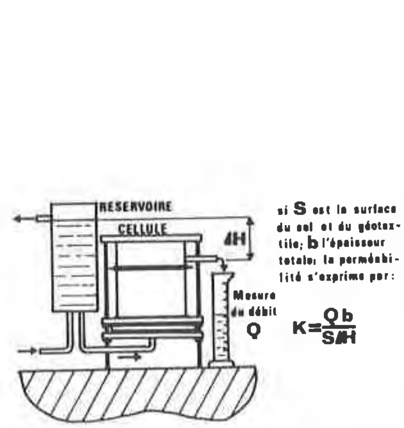

Fig. 12 - Mesure de la perméabilité

Tableau 1 - Perméabilités après essais dynamiques

Sol	W_o (%)	Sol + BD			Sol + TP		Sol	
		20	30	35	20	30	20	30
1	K (m/s) x 10^{-7}	2,0	0,8		1,5	1,7	1,8	2,1
	γ_d (kN/m3)	18,70	20,60		18,50	19,80	18,10	21,00
2	K (m/s) x 10^{-5}	0,5	0,7	1,1	0,6	1,9	1,2	1,0
	γ_d (kN/m3)	17,30	17,10	16,80	16,20	16,60	16,20	16,80

Etant donné que les valeurs de perméabilité sont comparables en presence ou non du geotextile, la permittivité des géotextiles vierges est très forte vis-à-vis de celle du sol (2) :

$$\psi_{BD} \simeq 2\ s^{-1} \qquad \psi_{TP} \simeq 0,2\ s^{-1}$$

$$\psi_{Sol\ 1} \simeq 10^{-5}\ s^{-1} \qquad \psi_{Sol\ 2} \simeq 10^{-3}\ s^{-1}$$

pour une épaisseur de 1 cm de sol.

$$\psi_{Global} = \frac{1}{\frac{1}{\psi_{Sol}} + \frac{1}{\psi_{Geotextile}}} \simeq \psi_{Sol}$$

Le résultat trouvé sur le complexe sol géotextile montre l'absence de colmatage aussi bien du TP que du BD.

Examen des géotextiles : après compactage, le géotextile presente des empreintes de silice blanche qui marquent l'emplacement des billes. C'est à ces endroits qu'on peut observer la présence la plus nette de sol inclus, figure 13.

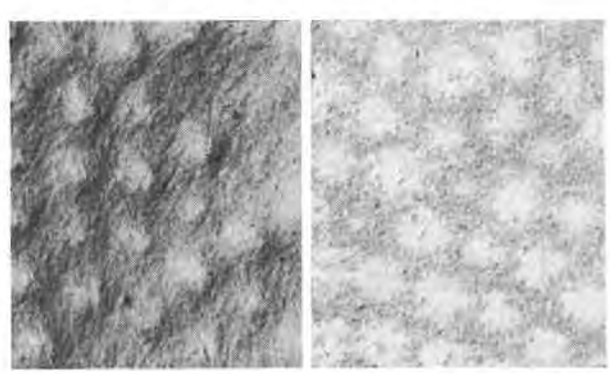

Fig. 13 - Géotextile en fin d'essai. Adroite : côté sol
A gauche : côté drain.

2 ESSAIS STATIQUES (Simulation de l'ouvrage en service)

Le dispositif expérimental est analogue à celui présenté pour les essais dynamiques : le complexe sol-géotextile-billes de verre de 150 mm de diamètre est comprimé par un piston grâce à un système de bras de levier pouvant exercer une contrainte normale au plan du géotextile allant jusqu'à 1 000 kPa, figure 14.

Fig. 14 - Cellule d'essais statiques

Après saturation de l'ensemble par montée lente de l'eau dans la cellule, depuis les billes vers le sol, un écoulement permanent est établi du sol vers les billes. Le débit est mesuré par pesée ou à l'aide d'un débitmètre.
Des capteurs de pression branchés à la base du piston, dans les billes et à différents niveaux dans le sol permettent de déterminer les pertes de charge ΔH entre plusieurs points :
- ΔH dues au sol seul,
- ΔH dues au sol + géotextile,
- ΔH dues au géotextile et à quelques mm de sol.

Ce sont des capteurs de pression à membrane, sans variation de volume ce qui permet de faire des mesures de pression avec des temps de stabilisation très courts malgré la faible perméabilité du sol.
Le but est de suivre l'évolution des permittivités respectives dans le temps. La durée des essais a été de 1 jour, 1 semaine ou 2 mois.
Sous les billes de verre, un papier filtre à forte rétention de particules permet de récupérer par lavage le sol passé au travers du geotextile. Des tests ont montré qu'aux vitesses d'écoulement utilisées, au maximum de 3.10^{-5} m/s, les pertes au travers du papier filtre étaient négligeables.
Sous compression statique, nous n'avons testé que le sol 1, en présence des géotextiles BD, TP et GSM 700, ce dernier étant un nontissé aiguilleté à fibres courtes à très forte porosité et très compressible (3).

2.1 Etude du passat

a - Influence du gradient de perte de charge dans le sol

Une série d'essais sous faible compression (10 kPa) a été réalisée sur BD avec des gradients $i = \Delta H/b$ allant de 0,5 à 30. Nous avons porté sur la figure 15 les masses de passat, par unité de surface de géotextile, en fonction de la vitesse d'écoulement mesurée.
Etant donnée la faible perméabilité du sol, $K = 7 . 10^{-7}$ m/s, des gradients élevés n'engendrent pas des vitesses d'écoulement suffisamment grandes pour obtenir des variations de masse des passats significatives. Il en est de même pour la granulométrie : il n'a pas été observé une influence particulière du gradient de perte de charge.

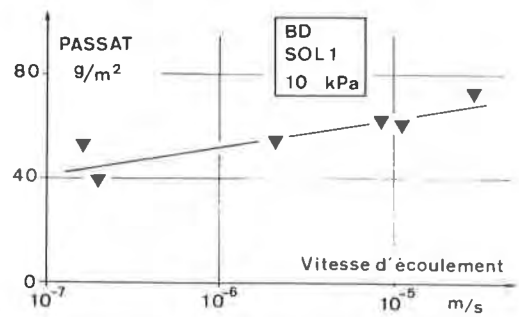

Fig. 15 - Passat en fonction de la vitesse d'écoulement

b Influence de la contrainte appliquée

La figure 16 montre les valeurs moyennes des masses de passat obtenus pour chaque contrainte. Malgré la forte dispersion, on constate que, avec l'augmentation de contrainte, la masse des passats M_1 diminue ; par ailleurs, le géotextile thermolié favorise moins le passage aux faibles compressions.

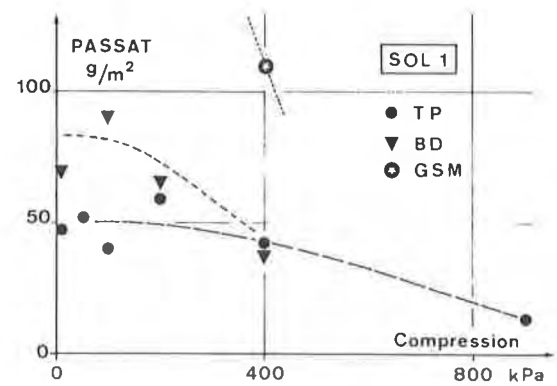

Fig. 16 - Passat en fonction de la contrainte appliquée

Les passats du GSM se trouvent réduits d'un facteur 10 pour une compression variant de 10 à 400 kPa.
Les analyses granulométriques, figures 17, 18 et 19 montrent que les passats sont différents du sol d'origine (sol 1) pour BD et TP. Il n'en est pas de même pour GSM dont le passat a même granulométrie que le sol 1.

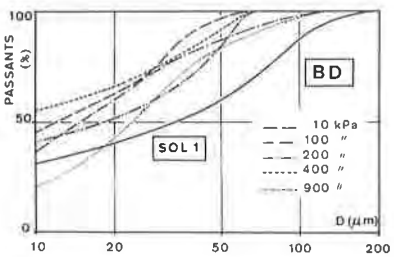

Fig. 17 - Granulométrie des passats du sol 1 avec BD

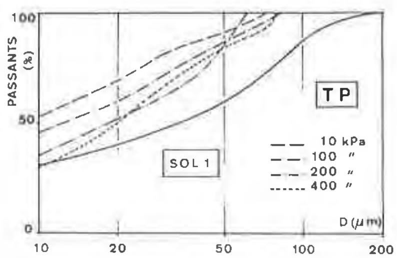

Fig. 18 - Granulométrie des passats du sol 1 avec TP

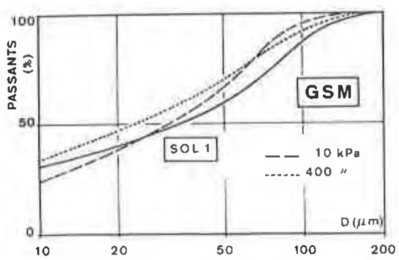

Fig. 19 - Granulométrie des passats du sol 1 avec GSM

Definissons un diamètre de filtration, comme étant égal au d_{98} du passat.
Pour l'aiguilleté BD on peut noter que pour 100 kPa ou 900 kPa, on retrouve la valeur de 125 µm annoncée par FAYOUX (1). Mais pour beaucoup d'essais, dans les conditions enoncées (compressions statiques, vitesses d'écoulement faibles) le diamètre de filtration est resté bien inferieur : les particules de 70 à 125 µm qui pourraient traverser le géotextile ont pu rester piégèes dans l'épaisseur de l'aiguilleté, ce qui a moins de chance de se produire pour le thermolié, plus mince.
Pour le TP, la compression n'influence pas le diamètre de filtration qui est d'environ 80µm, égal au O_{98}. Cependant lorsque la contrainte augmente, le passage des particules les plus grosses tend à croître (% de refus plus grand). La compression a un double effet :
- elle favorise la penétration du sol dans le textile. Si celui-ci est mince, cas du TP, le passat comprend plus de gros grains ; si il est épais, cas du BD, les chances de piegeages sont plus grandes mais aussi plus aleatoires.
- elle resserre les grains du sol entre eux, limitant la migration des particules les plus fines.

2.2 Risques de colmatage

La duree de filtration n'a pas eu, avec le sol 1 d'influence particuliere, ni sur la masse du passat, ni sur sa granulométrie.
Les mesures de permittivité effectuees, figure 20, ne montrent pas de diminution notable en fonction du temps. La légère décroissance observée au niveau du geotextile n'a pas de repercussion sur la permittivité globale du complexe sol + géotextile.

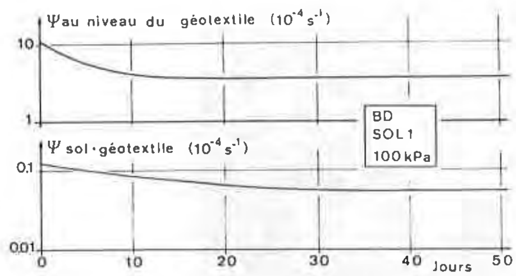

Fig. 20 - Permittivité en fonction du temps.

CONCLUSION

Les essais statiques et dynamiques se sont révélés complementaires, faisant ressortir chacun dans son domaine d'application, le rôle de certains paramètres. Nous retiendrons en particulier le rôle de la teneur en eau sur la fraction de sol passée au travers du géotextile et sur la fraction piégée lors du compactage des sols fins, et le rôle de la contrainte appliquée sur la granulometrie des passats.
Par rapport au O_{98} obtenu à l'aide d'essais classiques de tamisage, le diamètre de filtration obtenu par essais de simulation statique peut être inférieur à cette valeur, tandis que pour des essais sous sollicitations dynamiques, il peut être nettement supérieur.
Le d_{98} n'apparaît donc pas comme une caractéristique intrinsèque du geotextile.
Pour les sols fins non cohérents considérés, quelles que soient les conditions d'essais (compression, gradient hydraulique) nous n'avons jamais observé de colmatage notable. Quant au lessivage (masse de passat) il est beaucoup moins important sous sollicitations statiques que sous sollicitations dynamiques.

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Influence of the Fiber Size on the Filtration Characteristics of Needled-Punched Geotextiles

Influence de la fibrométrie sur les caractérestiques de filtration des géotextiles aiguilletés

Cut fibers manufacturing technics by means of carding and needlepunching process gives large possibilities, mainly for the fiber composition, fitted to filtration and draining works.

A global research has been made to determinate the influence of size fibers on ther hydraulic characteristics.

This investigation about filtration diameter, proves that depends of 3 following parameters :

- Fiber diameter
- "V", factor related to the total fiber length by volume unit
- "A", factor characterizing the spacial distribution of fibers and included between 2 (open structure) and 0,8 (dense structure).

Within these boundaries, "A" varies not only according to the geotextile void ratio but also to the needlepunching parameters.

Les techniques de fabrication, par cardage aiguilletage de fibres coupées, offrent de larges possibilités, principalement, l'adaptation de la composition "fibres" aux travaux de filtration et de drainage.

Une étude a été entreprise pour déterminer l'influence de la fibrométrie sur les caractéristiques hydrauliques de ces produits.

Cette communication, relative au diamètre de filtration, montre que celui-ci dépend des 3 paramètres suivants :

- Diamètre des fibres
- Facteur "V", lié à la longueur totale de fibres, par unité de volume
- Facteur "A", caractérisant la répartition spaciale des fibres.
  Ce facteur "A" est compris entre 2 (structures ouvertes) et 0,8 (structures denses).

A l'intérieur de ces limites, "A", varie non seulement en fonction de l'indice des vides du géotextile, mais aussi en fonction des paramètres d'aiguilletage.

Les techniques de fabrication de nappes nontissées, constituées de fibres coupées et basées sur le cardage et l'aiguilletage offrent de larges possibilités dans le domaine des géotextiles spécifiques du fait du nombre élevé de paramètres sur lesquels on peut intervenir.

En effet :

1°) Les fibres peuvent être variées à l'infini en jouant sur les paramètres suivants : polymère, titre de la fibre (diamètre moyen), section transversale, longueur de coupe frisure(fréquence et amplitude), thermofixation.

2°) Chaque nappe peut comporter un ou plusieurs types de fibres, d'un même polymère ou de polymères différents, tout en conservant une homogénéité excellente, du fait des cardages et nappages successifs.

3°) Il est possible d'associer, dans le cours du process, des nappes de caractéristiques très différentes, notamment sur le plan hydraulique, et d'obtenir ainsi des produits polyvalents.

4°) L'aiguilletage consolide les nappes, en réalisant une sorte de feutrage mécanique des fibres synthétiques. Il apporte à la fois la cohésion et la résistance, tout en conservant une grande souplesse et une très bonne aptitude à la déformation. Cette dernière caractéristique est particulièrement importante lors de l'emploi de nontissés comme couche filtrante enterrée, lorsqu'un contact aussi parfait que possible doit exister entre le sol et les filtres, sans risque de rupture ni d'éclatement. Les paramètres d'aiguilletage, parfaitement maitrisés et reproductibles permettent de contrôler les caractéristiques physiques du produit fini.

5°) Ces produits peuvent, en supplément, recevoir des éléments ou des traitements de renforcement tels que : structures tissées, fils continus, et/ou imprégnation, ou enduction superficielle, de résines en émulsion, tout en conservant leurs caractéristiques hydrauliques. L'ensemble des ces propriétés, permet de concevoir des produits géotextiles particulièrement bien adaptés au rôle hydraulique de filtration et de drainage, répondant aux spécifications des Ingénieurs du Génie Civil. Ces raisons ont conduit la Société Sommer, qui possède une large expérience de ces techniques de fabrication, à entreprendre une étude systématique des propriétés hydrauliques des géotextiles en fonction de la dimension des fibres constitutives. L'étude des caractéristiques de filtration, qui fait l'objet de cet exposé, a été confié au Cemagref.

I/ MODE OPERATOIRE

L'essai utilisé est l'essai de filtration hydrodynamique (présenté au colloque de PARIS 1977). Il s'agit d'un tamisage à l'eau, d'un sol à granulométrie continue et étalée. L'eau est animée d'un mouvement alternatif de haut en bas pour éviter tout colmatage. Le géotextile testé est placé sur une grille pour éviter sa déformation. On détermine ainsi le diamètre des plus grosses particules ayant traversé le géotextile. En pratique on retient comme valeur le d_{95} du sol ayant traversé le géotextile que l'on appelle le diamètre de filtration D_f.

II / PRODUITS TESTES

Les nappes testées ont été constituées à partir de quatre types de fibres :

A/ Fibres très fines
B/ Fibres fines
C/ Fibres moyennes
D/ Fibres grosses (PP)

Il s'agit de fibres polyester standard présentant une régularité de titre de ± 10 %. La gamme des titres varie de 3,3 à 70 dtex.

La composition des échantillons testés peut comprendre 1-2-3 ou 4 types de fibre, en mélange homogène. La proportion de chaque type de fibre peut varier. Pour chaque composition il a généralement été réalisé plusieurs nappes de masse surfacique différente (de 1 à 6).

Le programme d'essai a porté sur treize compositions différentes et un total de 54 échantillons. Les compositions testées sont les suivantes :

A0-100 % fibre A	A1-60 % fibre A	B1-60 % fibre B
B0-100 % fibre B	40 % fibre B	40 % fibre C
C0-100 % fibre C	A2-40 % fibre A	B2-40 % fibre B
D0-100 % fibre D	60 % fibre B	60 % fibre C
	A3-30 % fibre A	B3-28 % fibre B
	40 % fibre B	44 % fibre C
	30 % fibre C	28 % fibre D
	A4-25 % fibre A	C1-60 % fibre C
	25 % fibre B	40 % fibre D
	25 % fibre C	C2-50 % fibre C
	25 % fibre D	50 % fibre D

La masse surfacique des échantillons est comprise entre 130 et 640 g/m2, la masse minimale étant choisie, dans chaque gamme de produits, de façon à assurer une bonne homogénéité.

Les échantillons ont été réalisés en deux séries, la première série balayant largement les compositions possibles (onze) mais avec un maximum de trois nappes de masses surfaciques différentes par composition. La deuxième série n'a compris que cinq compositions, celles-ci comprenant par contre cinq ou six nappes de masse surfacique différente. La composition C2 a été réalisée ultérieurement en un seul grammage.

Les échantillons ont été réalisés en atelier pilote, sur des machines ne possédant pas toutes les possibilités de réglage et de contrôle du matériel industriel en grande largeur, ce qui peut expliquer certains points aberrants.

III/ CARACTERISTIQUES DES NAPPES EN FONCTION DE LEUR MASSE SURFACIQUE.

L'indice des vides e et l'épaisseur T_g des produits testés sont reportés sur les figures 1 et 2. Pour rendre possible leur lecture, nous n'avons tracé les courbes que pour quelques produits caractéristiques. Ces courbes présentent généralement des discontinuités, qui correspondent aux modifications des paramètres d'aiguilletage, nécessitées par l'augmentation de la masse surfacique de la nappe. (Il semble que l'on ait des familles de courbes parallèles, correspondant à des paramètres d'aiguilletage identiques). Nous verrons que cette particularité est importante pour l'interprétation des résultats. On remarque également des différences sensibles entre la première et la deuxième série, à composition fibres identique. Les masses volumiques étant différentes par suite de changement des paramètres d'aiguilletage.

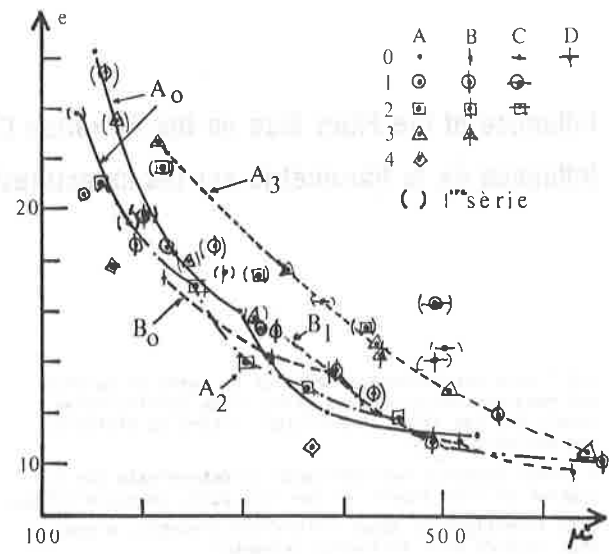


Fig. 1 Indice des vides des échantillons en fonction de la masse surfacique.

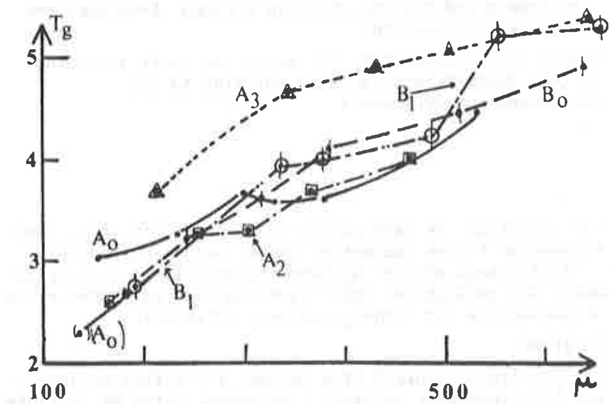


Fig. 2 Epaisseur des échantillons en fonction de la masse surfacique.

IV/ VARIATIONS DU DIAMETRE DE FILTRATION

IV1) Variation en fonction de la masse surfacique (Fig.3)

Dans cette figure, les courbes ont été lissées afin de mettre en évidence les tendances générales.

On observe les points suivants :

Pour une composition donnée, le diamètre de filtration décroit avec l'augmentation de masse surfacique. Pour les masses surfaciques faibles, cette décroissance est très forte, et d'autant plus marquée que le titre des fibres est faible.

Par contre, aux fortes masses surfaciques, il semble que l'on tende vers une limite et le diamètre de filtration est presque constant.

A masse surfacique constante, le diamètre de filtration croit avec l'augmentation du titre des fibres.

On observe des différences entre la première et la deuxième série. A composition égale et pour une même masse surfacique, le diamètre de filtration décroit avec l'indice des vides.

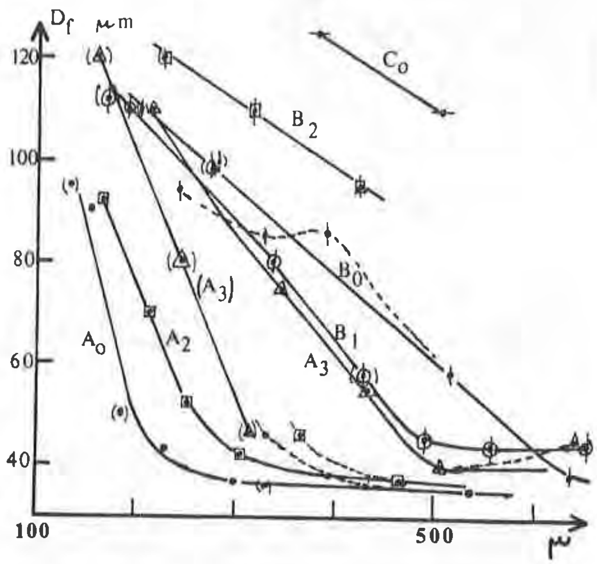


Fig. 3 Diamètre de filtration en fonction de la masse surfacique.

On observe enfin, sur la partie inférieure de la courbe, des fluctuations du diamètre de filtration, qui semblent aller en sens inverse des phénomènes précédents. (Augmentation du diamètre de filtration alors que la masse surfacique augmente et l'indice des vides diminue. Courbes Ao - A2 - A3).

IV2) Variation de D_f en fonction de l'indice des vides (Fig. 4)

Cette représentation permet de faire les constatations suivantes :

. On observe un resserrement des courbes, correspondant à une même composition, entre la première et la deuxième série. (Voir en particulier Ao - Bo (B1 A3). Les écarts s'expliquent par la différence de masse surfacique : à composition et indice des vides égaux, le matériau de masse surfacique la plus faible a un diamètre de filtration plus élevé, sauf pour deux points qui sont cerclés sur la figure (B1 et A3).

. On remarque aussi une très forte atténuation des points aberrants. (palier de la courbe Bo).

. Par ailleurs, les oscillations que l'on remarquait pour les masses surfaciques les plus élevées, sont toujours visibles.

. Pour une composition donnée, le diamètre de filtration est croissant avec l'indice des vides.

. Pour un indice des vides donné, le diamètre de filtration augmente avec la grosseur des fibres constituant le géotextile, sauf si la masse surfacique de l'échantillon le plus fin est plus faible. Il y a toutefois une anomalie entre les courbes Bo et B1 dont la position respective de certains points ne peut pas être expliquée par les remarques ci-dessus.

. La concavité des courbes se modifie avec la grosseur des fibres (diamètre). On pourrait attribuer ceci à une évolution de la structure interne du géotextile par suite d'une variation concomitante de la rigidité des fibres.

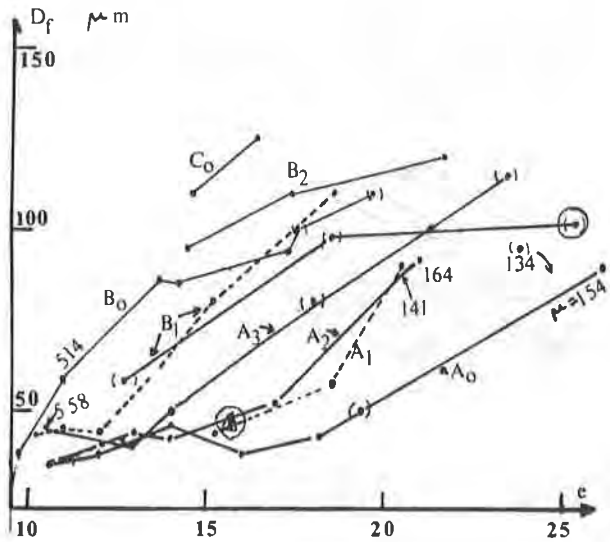
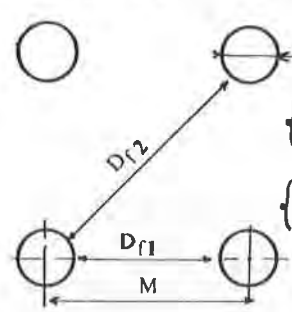


Fig. 4 Diamètre de filtration en fonction de l'indice des vides (La masse surfacique des échantillons est indiquée entre paranthèses).

V/ INTERPRETATION DES RESULTATS

Compte tenu du nombre d'essais réalisés, il est tentant d'essayer d'établir un modèle permettant de prévoir une formulation permettant d'obtenir un diamètre de filtration donné. MM. Leflaive et Puig, en 1973 proposent un modèle où l'on suppose que les fibres sont localement parallèles, suivant un réseau à mailles carrées (Fig. 5) et que la répartition des fibres est homogène dans le géotextile. On définit ainsi une maille M et un diamètre de filtration théorique en fonction de M et du diamètre de la fibre d_f.



$$\begin{cases} D_{f1} = d_f \left(\sqrt{\frac{\pi}{4}(1+e)} - 1\right) \\ A = 1 \end{cases}$$

$$\begin{cases} D_{f2} = d_f \left(\sqrt{\frac{\pi}{2}(1+e)} - 1\right) \\ A = 2 \end{cases}$$

Fig. 5 Diamètre de filtration théorique, selon Leflaive et Puig (73).

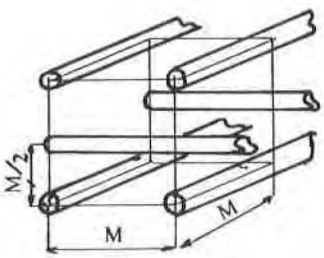
Le diamètre D_{f1} correspond au passage à travers un paquet de fibres situées dans le plan de la nappe ; D_{f2} correspond aux fibres entrainées perpendiculairement à la nappe par l'aiguilletage.

De même, Giroud a proposé une autre répartition de fibres conduisant à la relation :

$$\sqrt{\frac{\pi}{2}(1+e)}-1 < \frac{O\ moy}{d_f} < \sqrt{\frac{2}{3}\pi(1+e)} - 1$$

O moy = Diamètre moyen des pores qui peut être assimilé au diamètre de filtration.

La valeur $D_f = d_f\left(\sqrt{\frac{\pi}{2}(1+e)} - 1\right)$ correspond aussi à une structure comprenant deux lits de fibres perpendiculaires l'espace entre fibres parallèles étant M et l'espace entre deux lits étant $\frac{M}{2}$.



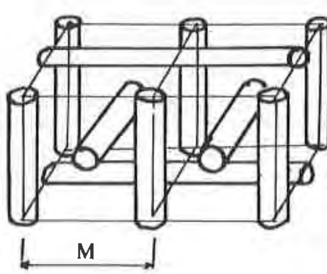
$$D_f = d_f\left(\sqrt{\frac{\pi}{2}(1+e)} - 1\right)$$

$$A = \sqrt{2}$$

Fig. 6 Répartition homogène des fibres dans le plan du géotextile.

Ces valeurs théoriques encadrent une partie de nos résultats, tout en étant un peu trop élevées pour les masses surfaciques les plus fortes.

D'autre part, les produits aiguilletés ayant des fibres verticales, nous avons étudié le modèle ci-dessous, où l'on suppose une répartition de fibres égale dans les trois directions, les fibres ne se touchant pas. (Fig. 7)



$$M = d_f\sqrt{\frac{3\pi}{4}(1+e)}$$

$$D_f = d_f\sqrt{\frac{3\pi}{8}(1+e)} - 1$$

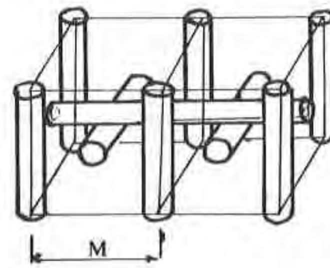
$$A = \sqrt{\frac{3}{2}}$$

Fig. 7 Répartition des fibres dans les trois directions

Dans ce schéma, le diamètre d'une sphère pouvant traverser le système est $\frac{M}{2} - d_f$ soit $D_f = d_f\left(\sqrt{\frac{3\pi(1+e)}{8}} - 1\right)$

Dans le cas où les lits de fibres horizontaux se rapprochent (Fig. 8), ce qui est une éventualité probable lorsque l'on diminue l'indice des vides, le diamètre de filtration devient égal à $\frac{M}{2} - d_f$ soit :

$$D_f = d_f\left(\sqrt{\frac{3\pi}{16}(1+e)} - 1\right).$$



$$D_f = d_f\sqrt{\frac{3\pi}{16}(1+e)} - 1$$

$$A = \sqrt{\frac{3}{2}}$$

Fig. 8 Répartition des fibres dans les trois directions avec nappes horizontales rapprochées.

Pour des géotextiles composés d'un mélange fibres de diamètres différents, pour lequel ces formules ne sont plus valables, il faut reprendre les mêmes schémas en calculant la maille M à partir de la longueur de fibre L par unité de volume :

$$L = \frac{\mu}{T_g}\ 10^4 \sum \frac{p_i}{\lambda i}$$

p_i étant la proportion en poids (p_i compris entre 0 et 1 de fibres de titre λ_i en dtex, la masse surfacique μ étant exprimée en g/m2.

Le diamètre de filtration est alors fonction de la maille et d'un diamètre équivalent des fibres, pondéré en fonction de la longueur respective de chacun des composants.

$$de = \frac{\sum \frac{d_{fi}\ p_i}{\lambda i}}{\sum \frac{p_i}{\lambda i}}$$

Dans le cas de la figure 5, on a $M = \sqrt{\frac{1}{L}}$; pour la figure 6, $M = \sqrt{\frac{2}{L}}$; pour les figures 7 et 8, $M = \sqrt{\frac{3}{L}}$.

Ceci nous conduit à une expression théorique du diamètre de filtration de la forme suivante :

$$D_f = A\sqrt{\frac{10^5\ T_g}{\mu \sum \frac{p_i}{i}}} - d_f = A \times V - d_f$$

avec μ en g/m2.

T_g = épaisseur du géotextile en mm.

λ_i = Titre de la fibre en dtex.

D_f et d_f en μ m.

d_f étant, soit le diamètre de la fibre si le géotextile n'en contient qu'un seul type, soit le diamètre équivalent d_e s'il en contient plusieurs.

Si les fibres sont fabriquées à partir du même polymère V peut également s'écrire sous la forme suivante :

$$V = 100\sqrt{\frac{10\ (1+e)}{\rho_f \sum \frac{p_i}{\lambda i}}} \quad \text{avec } \rho_f \text{ en Kg/m}^3$$

A prend les valeurs suivantes, en fonction de la structure :

Fig. 5	$A = 1$
Fig. 6	$A = \sqrt{2}$
Fig. 7	$A = \sqrt{\frac{3}{2}} = 1{,}22$

Fig. 8 $A = \frac{\sqrt{3}}{2} = 0{,}87$

V est donc un paramètre caractérisant la longueur de fibre par unité de volume et A un paramètre caractérisant la structure du géotextile.

Les diamètres de filtration mesurés sont donc reportés dans la figure 9 sous la forme $D_f + d_f$ en fonction de V. Les droites correspondant aux différentes valeurs de A ci-dessus, sont également reportées sur la figure.

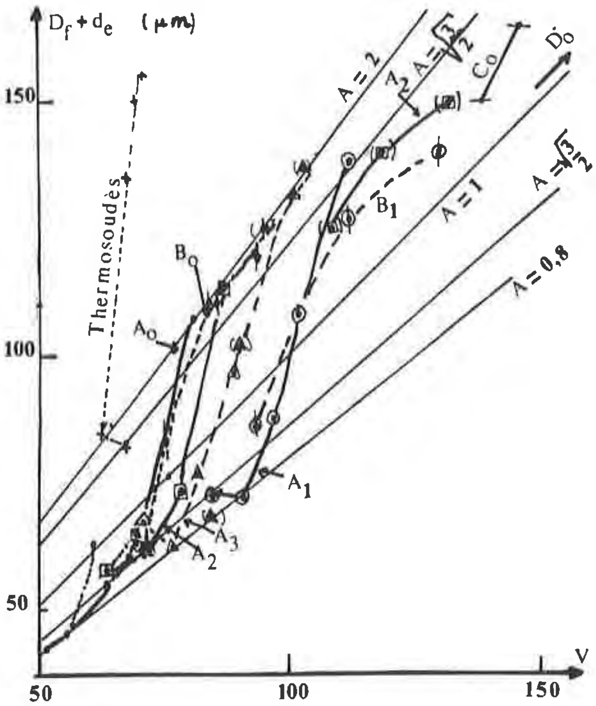


Fig. 9 Représentation de $D_f + d_f$ en fonction de V.

Les points correspondant, dans chaque gamme de mélange, aux produits les plus légers, sont compris entre les droites $D_f + d_f = \sqrt{2}$ V et $D_f + d_f = \frac{\sqrt{3}}{2}$ V. Par contre, les points correspondant aux produits les plus lourds et l'indice de vide le plus faible, sont centrés autour de la droite $D_f + d_f = \frac{\sqrt{3}}{2}$ V et, en ne descendant pas au dessous de la droite $D_f + d_f = 0{,}8$ V.

Entre ces deux zones, les points correspondant à une composition donnée, se déplacent sur des droites à peu près parallèles, de pente comprise entre trois et quatre. Ces droites semblent correspondre à une évolution de la struc ture entre un état lache et un état dense, provoquée par l'augmentation de masse surfacique et la diminution de l'indice des vides et la modification des paramètres.

Considérons le produit Ao. La courbe représentative se compose de deux tronçons décalés, alors que les deuxième troisième, cinquième et sixième points sont centrés sur la droite V $\frac{\sqrt{3}}{2}$, le quatrième est nettement plus haut.

Or, ceci correspond à une modification des paramètres d'aiguilletage, bien visible sur la courbe indice des vides en fonction de la masse surfacique. Le schéma de répartition des fibres dans les trois directions permet d'expliquer ce phénomène, en remarquant qu'à un indice des vides donné, l'augmentation du nombre de fibres verticales, (qui sont en fait essentiellement concentrées autour des points d'aiguilletage) diminue le nombre de fibres horizontales et donc augmente l'espace entre celles-ci, et par conséquent, augmente le diamètre de filtration. La structure étant modifiée on a donc un décallage de la courbe. Ce schéma permet bien de prendre en compte toutes les discontinuités et remontées que l'on peut observer sur les courbes A2, A3, B1.

On remarque aussi, sans qu'il puisse y être donné d'explication, que courbes Ao et Bo, qui sont très différentes dans les autres graphiques, se confondent en partie dans celui-ci, la zone correspondant aux échantillons lourds de Bo correspondant aux échantillons légers de Ao

La position de ces tronçons de droite, de pente 4, qui semblent caractériser le passage d'un état lache à un état dense, parait être fonction de la dimension des fibres, de la masse surfacique, de l'indice des vides et aussi des caractéristiques d'aiguilletage. Des essais complémentaires vont être entrepris pour préciser cet aspect.

On voit donc apparaitre un nouveau facteur qui est la structure du géotextile.

A titre indicatif, nous avons rapporté, sur la figure 9, les points représentatifs d'une gamme de géotextiles thermosoudés (T). Les points sont nettement en dehors des autres, mais pour les produits les plus lourds (226 et 267 g/m2), ils se situent aux alentours de la droite $\sqrt{2}$ V, correspondant à la structure de la figure 6. (nappes des fibres parallèles croisées à 90°), ce qui est assez satisfaisant pour l'esprit.

VI/ CONCLUSION

Le diamètre de filtration d'un géotextile dépend de sa fibrométrie, de sa masse surfacique, de son épaisseur et de sa structure.

A la production, les paramètres d'aiguilletage devront être strictement fixés et respectés si l'on veut garantir un diamètre de filtration donné.

Pour des produits aiguilletés dont la masse surfacique minimale est suffisante, en fonction de sa fibrométrie, pour que l'effet d'hétérogénéité ne joue pas, le diamètre de filtration est compris entre les bornes suivantes :

$0{,}8\ V - d_e < D_f < \sqrt{2}\ V - d_e$

V et d_e étant définis ci-dessus. Pour la plupart des produits épais, la valeur $0{,}88\ V - d_e$ est correcte à 10 % près.

La valeur $0{,}8\ V - d_e$ semblant être un minimum lié à la structure et en particulier à la répartition des fibres horizontales , il serait intéressant de vérifier si cette relation est appliquable pour déterminer le diamètre de filtration d'un géotextile sous contrainte.

Un panier spécial pour faire le test de filtration sous contrainte, doit être réalisé prochainement et permettra d'aborder ce problème.

Compte tenu des résultats acquis, une étude complémentaire va être entreprise pour préciser l'effet de la structure, ce qui permettra de définir, avec encore plus de précision, une formulation permettant d'obtenir un diamètre de filtration donné pour une masse surfacique donnée.

La superposition de ces résultats avec ceux de l'étude de la perméabilité en fonction des mêmes paramètres permet alors de définir, à priori, un géotextile répondant parfaitement aux exigences des prescripteurs.

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Inplane Permeability of Compressed Geotextiles
La perméabilité dans le plan des géotextiles comprimés

An apparatus was constructed to measure the transmissivity and thickness of geotextiles under a range of normal compressions between 2.5 kPa and 330 kPa. Water flowed radially in the plane of a ring-shaped specimen of the geotextile. Results are presented for fifteen different fabrics for the transmissivity and the derived permeability constants. The transmissivity values ranged from $5X10^{-4} m^2/s$ for a thick needled fabric under low compression to $2X10^{-9} m^2/s$ for a thin, compressed woven fabric. The corresponding derived permeability constants were 10^{-2}m/s to 10^{-5}m/s

Calculation of permeability constants using the channel theory developed by Fowler and Hertel gave excellent agreement for the experimentally observed values for thick, needled geotextiles.

On a construit un appareil pour mesurer la perméabilité en plan des géotextiles sous des crompressions normales entre 2.5 kPa et 330 kPa. L'eau s' écoule radialement dans le plan de l'échantillon du textile en forme d'anneau. On mesure le débit de l'eau et l'épaisseur du textile à diverses charges hydrauliques et compressions normales. On a étudié divers géotextiles y inclus des feutres aiguilletés de polyester et polypropylène, des non-tissés de liaison thermique ou chimique et des tissés de monofilaments de polypropylène.

La valeur de la transmissivité s'étend de $5X10^{-4} m^2/s$ pour un non-tissé, aiguilleté et épais sous une compression faible, à $2X10^{-9} m^2/s$ pour un tissé bien comprimé. Les coefficients de perméabilité correspondants sont 10^{-2}m/s et 10^{-5}m/s. De plusieurs théories pour calculer les coefficients de perméabilité pour les arrangement de fibres au hasard et d'une porosité déterminée, la théorie de canaux de Fowler et Hertel a donné des coefficients en trés bon accord avec les résultats des expériences avec les non-tissés aiguilletés.

1 INTRODUCTION

Geotextiles can be considered as having two important hydraulic characteristics: The ability to transport water across the fabric, i.e. the permittivity and the ability to conduct water along the plane of the fabric, i.e. the transmissivity. The latter has received relatively little attention particularly as a function of the compression normal to the plane of the fabric. This contribution is intended to supply more information on this subject.

The transmissivity of geotextiles is of importance in many applications, not only where the transport of water by the geotextile is its primary function such as in drains in earth dams; the ability of a geotextile to transport water in its plane can also affect the performance and endurance of structures where the fabric acts primarily as a reinforcement or as a filter.

The dependence of transmissivity on the normal (compressive) forces on a fabric has been measured in the past (1) but a more comprehensive investigation seemed indicated. The present tests were designed to cover a pressure range from 2.5 kPa to 332 kPa (equivalent to 0.1m to 15m of overburden, respectively).

2 DEFINITION AND FORMULAE

2.1 Transmissibility and Inplane Permeability

The transmissivity, θ, of a fabric is a measure of the ability to transport water in the plane of the fabric and is expressed as a volume rate of flow q_{20} (referred to a standard temperature of 20°C), per unit fabric width B_g, and unit hydraulic gradient, i. Hence:

$$\theta = q_{20}/B_g \ i \ m^2/s \tag{1}$$

The transmissivity can be converted to the inplane permeability coefficient K_p if the thickness of the geotextile, H_g, is known. This results in the well known Darcy's law (2):

$$K_p = \theta/H_g = q_{20}/B_g \ H_g \ i \ m/s \tag{2}$$

If experiments are carried out at another temperature T°C then the values of q_T observed must be converted to q_{20} to allow for variations of water viscosity, η_w, which are implicit in Darcy's law.

$$q_{20}/q_T = \eta_{20}/\eta_T = 1.78/(1 + 0.0352T + .002T^2)$$

2.2 Linear and Radial Flow

The form of Darcy's law given in equation 2 describes linear flow. The present apparatus uses radial flow. It consists of a central water reservoir, a ring-shaped specimen with inside diameter d_i and outside diameter d_o. After suitable mathematical manipulation the equation for laminar, radial flow can be given as:

$$\theta = \frac{q_{20}\ (\ln d_o - \ln d_i)}{2\pi(d_o - d_i)\ i} = \frac{q_{20}\ \ln(d_o/d_i)}{2\pi\ h} \ m^2/s$$

where h = hydraulic head (m of water).

2.3 Porosity

The definition of the porosity, n, of a geotextile is analogous to that of soil: It is the volume fraction of air in the fabric and can be related to the area density, μ_g, the fabric thickness, H_g, and the volume density of the constituent fibers, ρ_f, by

$$n = 1 - \mu_g/H_g \rho_f$$

This quantity can also be expressed as a percentage.

3 THEORETICAL

Several theories have been proposed to calculate the permeability of fiber assemblies. They all assume a minimum fabric thickness in excess of 20 fiber diameters. Assuming in addition random distribution of fibers, the theories can be generalized as

$$K_p = (g\rho_w \lambda_f / 24\pi n_w \alpha_f \rho_f)\ f(n) \qquad (3)$$

where α_f is a fiber shape factor and equal unity for circular fibers and f(n) is a function of the porosity, n, the form of which depends on the theory used.

For the channel theory developed by Fowler and Hertel (3)

$$f(n)_C = n^3/(1-n)^2.$$

For the drag theories of Happel (4) and Kuwabara (5) the functions are, respectively

$$f(n)_H = [4\ \ln(1-n)^{-1} - n(8-4n+n^2)/(2-2n+n^2)]/(1-n)$$

and

$$f(n)_K = [2\ \ln(1-n)^{-1} - n(2-n)]/(1-n).$$

Fig. 1 shows the shape of these three functions from n = 0.1 to n = 0.96 covering a predicted range of permeabilities of over 10^5.

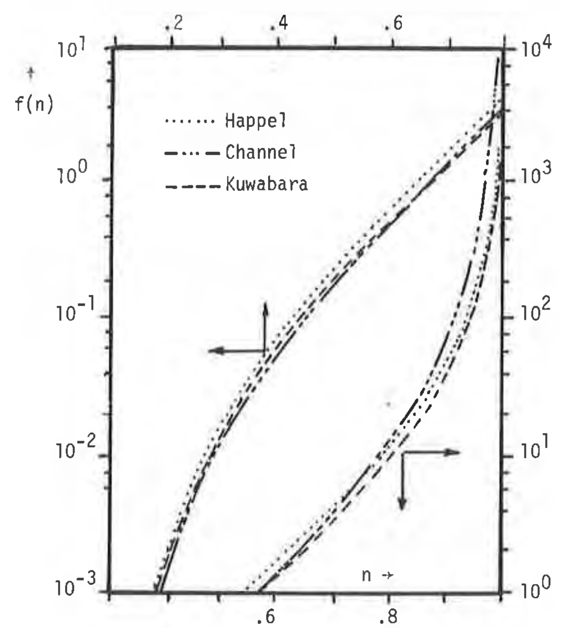

Fig. 1 Log Porosity permeability function, f(n), vs. Porosity, n, for channel and drag theories.

The region of greatest interest for geotextiles lies between n = 0.7 to 0.95 and thus includes a portion (above n = 0.85) where there are significant differences between the three theories. As will be seen the experimental data favored the mathematically simpler, channel theory.

4 EXPERIMENTAL

4.1 Apparatus

As already stated the test specimens were in the shape of a ring and a section of the specimen support is shown in Fig. 2. The geotextile, A, is sandwiched between thin rubber gaskets, B, of a similar shape and which act to form a seal against the base plate C and the upper part, shaped like a dome D. The base plate has a central access tube E containing a thermometer T and leading to manometer M (in Fig. 3).

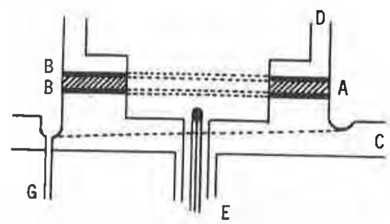

Fig. 2 Diagram of sample support and water collection groove.

Surrounding the specimen support area, the base plate has a sloping circular groove, F, which collects the water passed through the geotextile and leads it to the discharge tube G to be collected over measured time intervals.

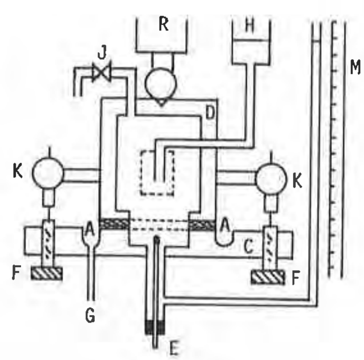

Fig. 3 Schematic of permeameter for inplane water flow.

Fig. 3 shows the complete apparatus with the upper dome having been lowered onto the specimen assembly using locating pins for better alignment. The knurled screws, F, act as adjustable anvils to the micrometers, K, which are rigidly connected to D. The micrometers are set to read zero when no fabric is between the rubber gaskets and subsequently read the fabric thickness under various compressions. They are also used as a check to assure uniform pressure application to the specimen.

Deaired water is admitted via a constant head reservoir, H, to the cavity of the dome. The water inlet pipe ends inside a diffuser to assure uniform distribution of water to the specimen. A bleed pipe and valve, J, are used to fill the cavity at the start of the run.

The compression to the geotextile is applied by means of a pneumatic ram, R, ending in a hemispherical shape which fits into a conical depression in the top of the dome. The pressure on the ram, displayed on a gauge, together with the weight of the dome and the hydraulic upthrust due to the water in the cavity, are used to calculate the compressive force on the geotextile.

The compressive stress varied from 23 kPa (when only the weight of the dome acted on the fabric) to 332 kPa when the pneumatic plunger was pressurized to 1380 kPa.

Flow readings were taken 5 minutes after any change in pressure. The water was collected over a convenient time interval (usually 60 seconds) and repeated twice. At the same time the temperature, pressure and fabric thickness were also recorded.

4.2 Geotextiles Tested

Fifteen geotextiles, obtained between 1976 and 1978 from commercial suppliers, were tested. They are listed with some of their characteristics in Table 1.

TABLE I FABRIC CONSTRUCTION AND PROPERTIES

Fabric Code †	Bond ‡	Polymer †	ρ_f kg/m^3	λ_f mg/m	μ kg/m^2	H_g *mm	n *	θX10^6 *m^2/s	K_pX10^4 * m/s
B1	N	PET	1380	9.3	.167	.68	.82	123	18.1
B2	"	"	"	9.4	.225	.93	.825	160	17.2
B3	"	"	"	9.0	.276	1.10	.825	152	13.8
B4	"	"	"	8.9	.331	1.38	.83	215	15.6
B5	"	"	"	9.4	.523	2.26	.83	410	18.1
F2	N	PP	910	7.0	.351	1.86	.79	223	12.0
C0	"	"	"	10.3	.302	1.29	.74	114	8.8
S4	"	"	"	3.5	.121	.64	.79	54	8.5
P4	B,N	PP,N	"	4.6	.143	.52	.70	11	2.0
M4	B,N	PP	"	11.8	.132	.47	.69	27	5.5
T4	B	"	"	12.5	.134	.40	.63	9	2.2
T6	B	"	"	--	.202	.51	.565	6	1.2
L8	W	"	"	--	.241	.45	.41	27	6.0
P8	W	"	"	--	.235	.43	.35	8	1.9
A1	W	PAN	980	--	.104	.30	.65	8	2.8

‡ Bonding Code: N Needled, B Heat or chemical bonding
W Woven

† Polymer Code: PET Polyester, PP Polypropylene
PAN Acrylic fiber, N Nylon

* Values given under a compression of 100 kPa

† B1 through B5 are Bidim® (registered trade mark of Monsanto Company) nonwoven polyester fabrics

The specimens were selected at random from 1 m^2 samples and cut in a die press. The outer diameter was 108mm and the inner diameter 57mm, hence the test length in the direction of flow was 25.5mm. Most tests were carried out in quintuplicate. Reproducibility for individual specimens was less than 5% but variability between specimens was typically ±15% although this varied with the geotextile tested.

4.3 Applicability of Darcy's Law

Preliminary calculations had indicated that laminar flow was expected in the tests. This was verified experimentally by a test series at 23 kPa compression. The flow rates were varied by changing the height of the water reservoir over the range of 20mm to 1 m. The 5 polyester fabrics and 6 polypropylene fabrics of various thicknesses and constructions were tested. In all cases the results showed good linear behavior. The correlation coefficients were all 99.9% or greater and the flow axis intercepts lay within ±0.2X10^{-6}m^3/s thus confirming within experimental error that Darcy's law was applicable.

Subsequent tests were carried out at a constant hydraulic head of 300mm of water, so that the transmissivity was related to the volume flow (corrected to 20°C) by:

$$\theta = 3.374 \times 10^{-7} \; q_{20} \; m^2/s.$$

4.4 Supplemental Tests Under Very Low Compression

Because of the large weight of the apparatus described earlier it was decided to build an alternate lightweight head to do additional runs at much lower fabric compressions. The compressive stress in this tester was 2.52 kPa and the tests were carried out under a falling head mode with time and volume readings being taken at intervals of 25mm hydraulic head from 356mm to 105mm. These readings were temperature corrected and appropriately transformed for the different geometrical arrangement and the transmissibility calculated. These readings were well in line with the trends observed in the constant head apparatus and are included in the following plots.

The fabric thickness was not measured during these low compression permeability tests but calculated from independent thickness measurements under various compressions.

4.5 Transmissivity Measurements

The experimental results of 5 different test specimens are plotted in Fig. 4 and show the decrease in transmissivity of the family of needled polyester geotextiles. They were produced by the same manufacturing process and only differed in their area density (mass per unit area). Under lower compressions the transmissivities were in the order of the area densities, but at higher compressions some divergencies were observed. These differences lay within specimen to specimen variations and are therefore not unexpected.

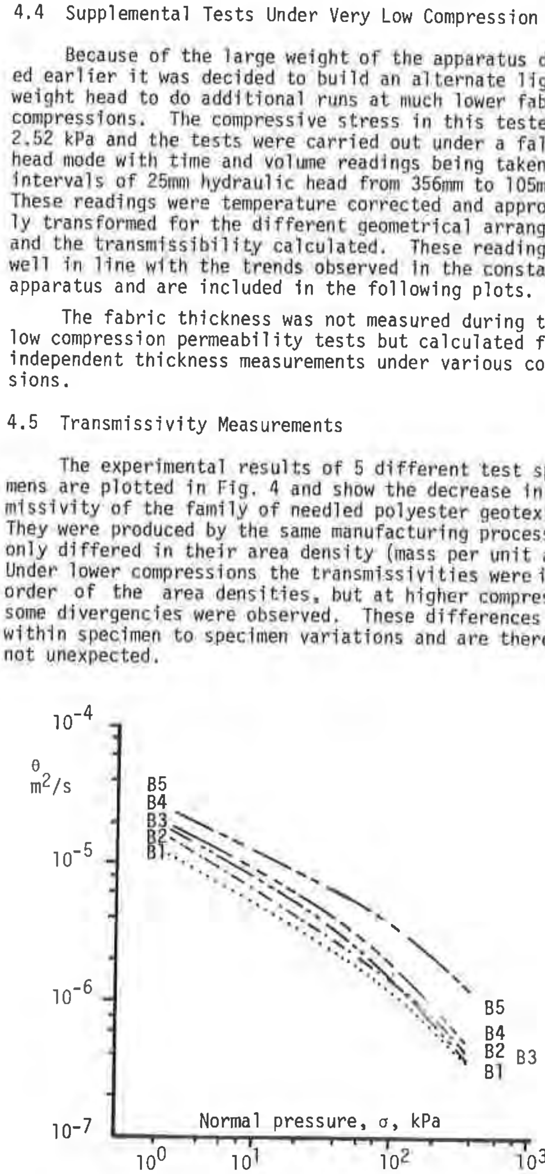

Fig. 4 Log Transmissivity, θ, vs. log normal pressure, σ, for needled polyester geotextiles.

Fig. 5 shows the corresponding plot for the group of needled and bonded polypropylene fabrics and the woven geotextiles. The needled polypropylene fabrics are very similar to the needled polyester fabrics but the transmissivities tend to decrease somewhat more rapidly at higher compressions. Some of the other fabrics showed some surprising trends. The woven fabrics P8 and L8 were of similar structure, construction, appearance and transmissibility at low compression. However at higher compressions they differed by a factor of 30. Neither of these materials nor the geotextiles designated A1, P4, T4, T6 can be considered suitable as drainage geotextiles. M4 and S4 are of intermediate transmissivity at low compressions but must be included in this list of unsuitables, especially at higher compressions.

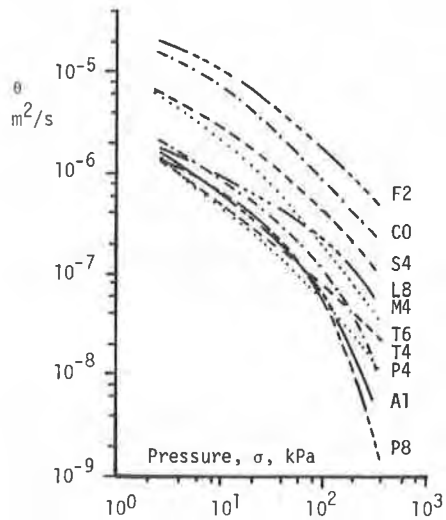

Fig. 5 Log Transmissivity, θ, vs. log normal pressure, σ, for polypropylene and woven geotextiles.

The fabrics best suited for transporting water in their plane are thick, needled fabrics with area densities of .250 kg/m^2 or higher.

As Figs. 4 and 5 also show, the decrease in transmissivity tends to be more gradual for lower compressions but accelerates as the pressure are increased. It is therefore advisable when estimating transmissivities for pressures beyond those for which experimental data exist to allow for this factor.

Over the range of normal compressions tested the transmissivity decreased to less than 10% of the low compression value for all fabrics. In one case it dropped to less than 0.2%.

4.6 Fabric Thickness under Pressure

When using geotextiles as a drain the value of the transmissivity of the fabric under the anticipated compressions is the most important parameter for the geotechnical engineer. However, the value of the permeability of the geotextile can be used to gain insight into the behavior of fabrics. It has been possible to link it to theories using fabric properties easily measured and which may make extensive hydraulic testing of new fabrics unnecessary. A few key tests can then be extrapolated beyond the range of experiments and yield reasonably assured estimates.

The thickness of fabrics was measured under two different conditions: One series was measured wet in the permeability test as already indicated. In another series the polyester fabrics were tested dry in a compression tester. Fig. 6 shows the two series combined as there was no significant difference between the two when the slightly different area densities of the samples were taken into account.

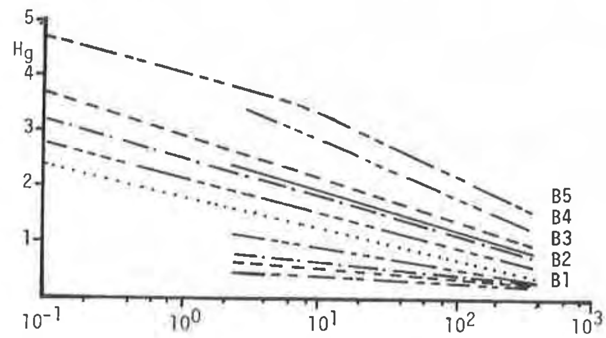

Fig. 6 Thickness of geotextiles, H_g vs. log normal pressure, σ.

On the basis of the thickness tests the porosity of the geotextiles was calculated and as Fig. 7 shows it decreased as the pressure was increased. The B family of fabrics showed a range of porosities from .906 to .946 at 500 Pa (the pressure recommended by ASTM for thickness measurements of geotextiles). At this pressure the thinnest fabric was also the most porous. At the highest pressure measured the porosity had decreased to .748 to .772 with the thickest fabric showing the highest porosity.

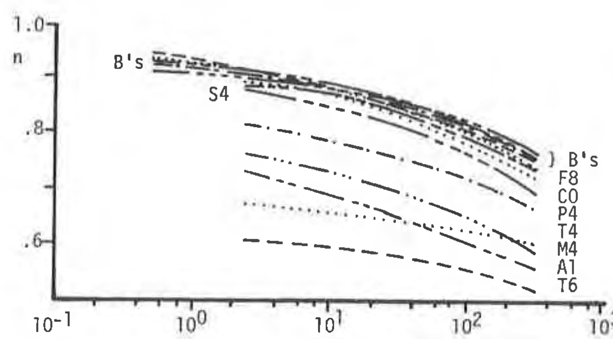

Fig. 7 Porosity of geotextiles, n, vs. log normal pressure, σ.

The concept of porosity for woven or very thin, compressed nonwoven fabrics is not really appropriate but included here to show how the properties of geotextiles are quite gradual and any statement of cut-off between porous and nonporous fabrics is to some extent arbitrary.

4.7 Inplane Permeability Constants

The values of transmissivity shown in Figs. 4 and 5 divided by the thickness of the fabric at the same normal pressure yield the value of the inplane permeability constants. They are shown in Fig. 8. They can be seen to cover a narrower range than the transmissivity values since both the thickness as well as the porosity of the fabrics decrease with pressures.

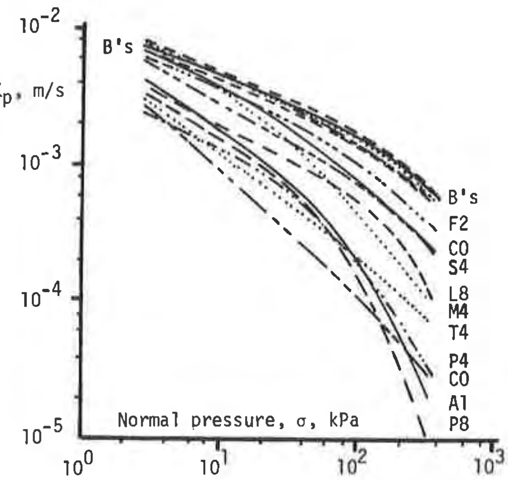

Fig. 8 Log permeability constant K_p vs. log normal pressure, σ for geotextiles.

The needled polyester fabrics are essentially independent of the area density of the fabric, which merely indicated that the structure of these fabrics was very similar and not affected by the thickness. This fabric group showed the highest permeability constants. The values of the needled polypropylene fabrics F8, CO and S4 were similar at low compressions but decreased more rapidly with pressure. Most of other fabrics were significantly less permeable at all pressures, and the differences increased with higher compressions.

Typical values for permeability constants for normal pressures of 100 kPa are listed in Table I, others will be found in the next section.

4.8 Comparison with Theory

The extensive data available on the needled polyester geotextiles were used to test the relative fit of experimental data with the three theories discussed in section 3. To this end the derived values of f(n) were compared to the values of $f(n)_C$, $f(n)_H$ and $f(n)_K$. From equation (3)

$$f(n) = (24\pi\eta_w\lambda_f/g\rho_w\lambda_f)K_p = A\ K_p$$

the factor A can be calculated from the data contained in Table I. Fig. 9 shows the points for the thick needled fabrics: the needled polyesters and polypropylenes F8 and CO. The fit with the channel theory is excellent for the polyesters and F8 and not unsatisfactory for CO. The latter shows a significant deviation from the theory but the fit with the channel theory is better than for the drag theories.

From this evidence the channel theory was used to calculate permeability constants for several of the nonwoven geotextiles tested. They are shown in Table II and illustrate that the fabrics M4, T4 and T6 are too thin for the theory to be appropriate. Fabrics S4 and P4 also show significant deviations which may be due to their non-uniformity of structure.

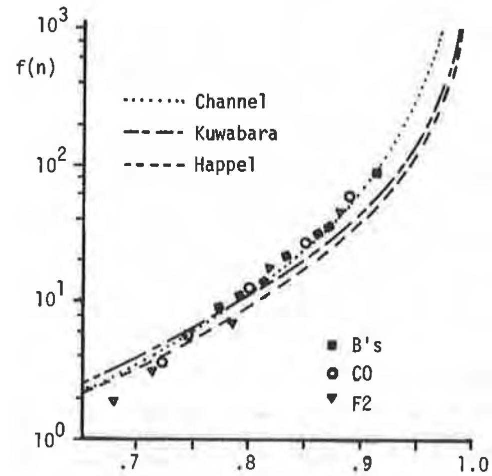

Fig. 9 Log porosity permeability function, f(n) vs. porosity, n. Theory and from experiments.

TABLE II EXPERIMENTAL VS. CALCULATED PERMEABILITY CONSTANTS

Fabric	Experimental/Calculated, K_p, 10^{-4}m/s at: σ = 2.52 kPa	σ = 23 kPa	σ = 137 kPa	σ = 332 kPa
B1	75.5 / 103.5	32.1 / 27.1	13.2 / 10.9	6.8 / 5.7
B2	83.8 / 90.7	30.4 / 27.1	12.3 / 10.9	6.2 / 6.8
B3	76.9 / 78.9	33.8 / 33.0	14.3 / 11.9	7.3 / 6.8
B4	68.2 / 90.7	31.2 / 33.0	12.6 / 11.9	6.4 / 8.0
B5	64.4 / 86.0	32.7 / 33.0	12.9 / 11.9	6.6 / 8.0
F8	59.2 / 58.3	25.5 / 27.3	8.1 / 10.0	3.9 / 5.0
CO	64.2 / 77.4	24.5 / 36.3	5.7 / 10.6	2.7 / 4.7
S4	54.7 / 28.8	18.1 / 15.8	6.1 / 4.9	2.5 / 1.8
P4	24.7 / 15.8	8.7 / 5.3	1.4 / 3.3	0.3 / 2.0
M4	83.2*/ 13.4	20.0*/ 7.0	3.2 / 3.8	1.2 / 2.3
T4	31.1*/ 5.7	6.4 / 4.7	1.7 / 3.4	0.8 / 2.7
T6	26.8*/ 2.9	4.3 / 2.5	0.8 / 1.8	0.3 / 1.4

* Suspected inadequate seal between rubber gasket and specimen

5 SUMMARY AND CONLUSIONS

These tests on the transmissivity and inplane permeability constants showed that for thick, bulky (needled) fabrics the observed values agreed remarkably well with those calculated from the channel theory developed by Fowler and Hertel. The theory, being based on the assumption of complete (3 dimensional) istropy of the fiber arrangement will therefore predict the same permeability constant for water moving normal to the fabric plane. Within 20 per cent that appears to be the case for values reported for B-3 (1).

The theoretical development shows that if the fabric porosity and the fiber shape, linear and bulk densities are known, then the permeability and hence the transmissivity and permittivity can be calculated.

For bulky fabrics that use binders or have significantly nonisotropic or non-uniform construction the theory may still yield a useful approximation but must be used with great caution. It may give an upper or lower limit to actual permeabilities depending on the nature of deviation from assumed structure.

The transmissivity of thin fabrics is so low that the drainage function of such geotextiles is essentially negligible and correspondence with this or any other theory is of no interest to the design engineer.

ACKNOWLEDGEMENTS

The author wishes to recognize the contribution of Dr. C. C. Cheng who was responsible for much of the work here reported and for the design of the apparatus. The assistance of W. E. Artz is hereby also gratefully acknowledged.

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Filtration and Drainage with Geotextiles—Tests and Requirements

Filtration et drainage au moyen de géotextiles-essais et specifications

Hydraulic properties of geotextiles are important, specially when they are used for hydraulic works such as dams, Five different tests are presented.

- The hydrodynamic filtration test toe determine the filtration diameter ; the way of operation is proposed and justified through many different tests.

- The static filtration test, which concernes the behaviour of the soil-geotextile combination under the action of flowing water.

- The measure of the permittivity, developed with the help of the I.T.F.

- The measure of the transmissivity, the apparatius are conceived to appreciate the influence of the normal strain and of the contiguous soil.

In the last part of the paper, some aspects of design of geotextiles in hydraulic works are presented.

Les propriétés hydrauliques des géotextiles sont évidemment essentielles lorsqu'il s'agit de leurs utilisations dans les ouvrages hydrauliques. Cinq types d'essais sont proposés :

- La mesure du diamètre de filtration au moyen d'un essai de filtration hydrodynamique, le mode opératoire proposé est justifié par des essais comparatifs

- L'essai de filtre statique qui permet d'évaluer le comportement sous l'action de l'eau du complexe sol-géotextile.

- La mesure de permittivité développée en collaboration avec l'I.T.F.

- La mesure de transmissivité : les appareils présentés permettent de prendre en compte l'effet de contact avec le sol et l'influence de la contrainte normale.

Dans la deuxième partie de la communication, certains aspects du dimensionnement des géotextiles sont abordés tant en ce qui concerne le drainage que la filtration.

Dès l'apparition des géotextiles en France, le CEMAGREF s'est intéressé de très près à leur utilisation, en particulier pour les possibilités qu'ils offraient dans les barrages en terre. Dans ce domaine d'application, ces matériaux remplissent le rôle de filtre et/ou de drain, en général au contact avec un sol compacté. Nous avons donc été amenés à développer d'une part une série d'essais permettant de caractériser les caractéristiques hydrauliques des géotextiles en filtration (caractérisation des pores d'un géotextile, comportement sol-textile, permittivité, et drainage), et d'autre part une méthodologie d'emploi de ces produits.

I - DETERMINATION DU DIAMETRE DE FILTRATION - ESSAI HYDRODYNAMIQUE

Le terme de porométrie nous paraît être généralement employé de façon abusive. Il désigne en effet une répartition de la taille des pores, c'est-à-dire des cheminements possibles à travers le géotextile, en fonction de leur fréquence. Or, pour un non-tissé, à partir d'un point d'entrée dans le textile, il y a un très grand nombre de cheminements possibles, le grain pouvant se déplacer dans le plan de la nappe. Avant d'étudier la répartition des chemins possibles, comment les définit-on ? A partir d'un point d'entrée, sont-ils limités par un cône à 20°, 35, 90 ou 135° ? Si la signification physique de O n, tels qu'il est défini dans les recommandations pour les unités et les symboles de ce colloque est évidente dans le cas des matériaux tissés, elle ne l'est donc pas du tout dans le cas des non-tissés. L'approche de ce terme par les différents auteurs recouvre d'ailleurs souvent des réalités physiques très diverses. D'autre part, la courbe granulométrique de sols ayant traversé le géotextile ne peut nous donner que la dimension des plus gros éléments qui peuvent le traverser. Le reste de la courbe est fonction du ou des sols testés et des modalités de l'essai. Une telle courbe ne peut donc absolument pas donner une répartition des pores. Ces raisons nous ont conduit à garder comme seule caractéristique de filtration le diamètre des plus gros éléments pouvant traverser le géotextile, appelé "diamètre de filtration D_f ".

L'essai que nous avons retenu est l'essai de filtration hydrodynamique, déjà présenté au colloque de PARIS en 1977. Un sol fin à granulométrie continue et étalée est introduit dans des paniers dont le fond est constitué par le géotextile à étudier. Les paniers sont alternativement plongés et ressortis de l'eau (fig. 1). A l'émersion du panier, l'eau contenue dans le tamis entraine par percolation les particules du sol à travers le filtre. A l'immersion, il se produit un décolmatage qui brise le filtre naturel tendant à se former, en cours d'essai, à la surface du géotextile et chasse les grosses particules qui peuvent obstruer des pores. Le sol qui traverse le géotextile est décanté et recueilli pour analyse granulométrique. Nous avons essayé de mesurer l'influence des différents paramètres sur le résultat des essais.

- Influence du sol : nous avons testé 3 sols différents dont la granulométrie est donnée sur la figure 2.

- Quantité de sol au dessus du géotextile : nous avons testé une charge de sol dans les paniers de 2,2 kg, 1,1 kg et 0,7 kg de sol sec.

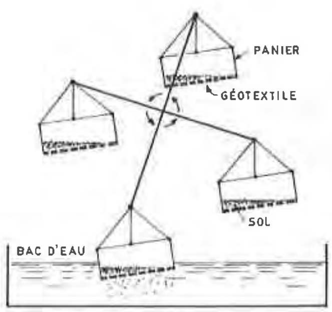

Fig. 1 : dispositif d'essai de filtration hydrodynamique

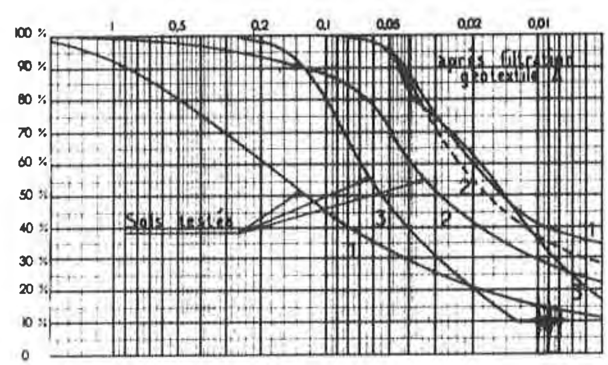

Fig. 2 : filtration hydrodynamique - sols testés

- Durée de l'essai : pour la vitesse de rotation normale de l'appareil, on a procédé à des essais pendant 7, 16 et 24 h.
- Variation du niveau d'immersion : par rapport au niveau normal d'immersion, on a procédé à des essais avec des variations de - 2, - 4 et + 2 cm.
- Vitesse : des essais ont été faits à vitesse de rotation double pendant 24 h.

Le résultat d'une partie de ces essais est porté dans le tableau I. Ces résultats montrent une faible variation de la partie la plus grossière du sol ayant traversé le textile. Celle-ci est d'ailleurs principalement due aux variations du géotextile plutôt qu'aux variations de mode opératoire ; ce qui n'est pas le cas par contre pour la partie fine de la courbe granulométrique.

. Pour des produits déformables, on a constaté une augmentation significative de la partie grossière du tamisat, lorsqu'on laisse le produit libre de se déformer sous le poids du sol.
. La quantité de sol ayant traversé le géotextile met en évidence des tendances, mais ce n'est pas un paramètre reproductible. Elle dépend en effet de l'arrangement du sol sur le géotextile et du temps d'essai .

Compte tenu de ces remarques, le mode opératoire a été fixé de la façon suivante :

- Le géotextile est placé sur une grille pour éviter sa déformation
- Sol utilisé : dimension maximale supérieure à 2 fois le diamètre de filtration : coefficient d'uniformité $d_{60}/d_{10} > 6$.
-Quantité de sol: 2 kg ± 1 par panier, la quantité dans chaque panier devant être la même.

Tableau I

Géotextile	durée essai	sol	d_{60}/d_{10}	poids de sol par panier d_{60}	immersion maximale des paniers	d_{97}	d_{95}	d_{90}
A	24h	1	60	2,2	10	50	47	41
	24	2	13,3	2,2	10	52	48	43
	24	3	5,8	2,2	10	53	50	45
	7	1		2,2	10	47	43	38
	16	1		2,2	10	45	43	38
	24	1		1,1	10	55	49	44
	24	1		0,7	10	53	48	38
B	24	1		2,2	10	53	48	37
	17	1		2,2	10	47	44	37
	24	1	vitesse double	2,2				
	24	1		0,7	10	55	50	44
	24	2		2,2	10	55	42	36
effet du niveau C	24	1			14 cm	96	92	60
	24	1			12 cm	94	90	60
	24	1			8 cm	100	95	60

- Profondeur de pénétration du panier dans l'eau : 10 cm ± 1
- Nombre de cycles immersion - émersion : 2.500
- Rapport entre le temps de ressuyage t_r et le temps d'immersion t_i

tel que $\frac{t_r}{t_i} = 4,5 \pm 0,5$

- Temps d'essai compris entre 12 et 36 h, le temps d'essai étant tel qu'en fin de ressuyage, lors des derniers cycles, il n'y ait plus d'eau au dessus du géotextile.
- Le dispositif d'essais doit être tel que l'eau d'égouttage d'un panier ne tombe pas dans le panier suivant (projection d'éléments grossiers hors des paniers)
- Le diamètre de filtration retenu est $D_f = d_{95}$ du sol ayant traversé le géotextile. Ce diamètre est plus facile à appréhender que le d_{97} et moins sensible que le d_{90} aux conditions expérimentales.
- Par ailleurs, nous avons montré que le diamètre de filtration d'un géotextile dépend de sa masse surfacique, de sa structure et de son indice des vides. La masse surfacique et l'épaisseur du géotextile doivent donc être notées.

II - ESSAIS DE FILTRATION

Les essais de filtration statiques en laboratoire constituent un intermédiaire indispensable entre des essais relatifs à la mesure de caractéristiques intrinsèques des géotextiles (mesures de diamètre de filtration par exemple) et l'observation in situ des ouvrages. Ces essais sont effectués soit à l'occasion d'études particulières, soit dans le cadre d'études systématiques. C'est surtout à ce deuxième titre que le CEMAGREF a effectué des essais de filtration statique.

Deux appareils ont été utilisés. L'appareil (A) est en aluminium et ne permettait l'observation qu'à postériori ; l'appareil (B) est en plexiglas et permet l'observation directe des entrainements en particulier lorsque les couleurs sont suffisamment contrastées.

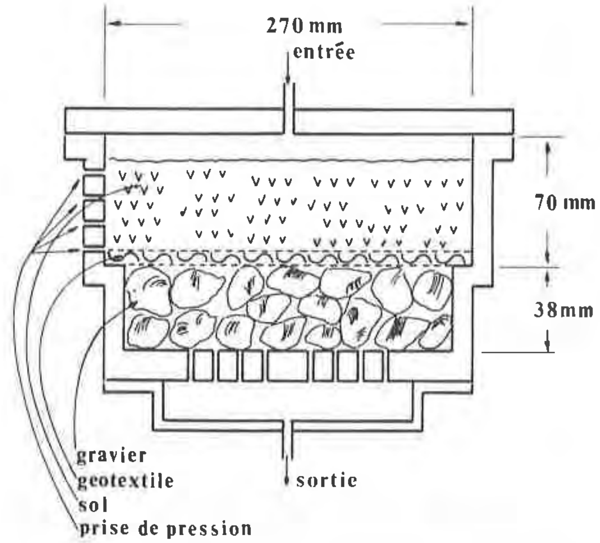

Fig. 3 : Appareil A d'essai de filtre

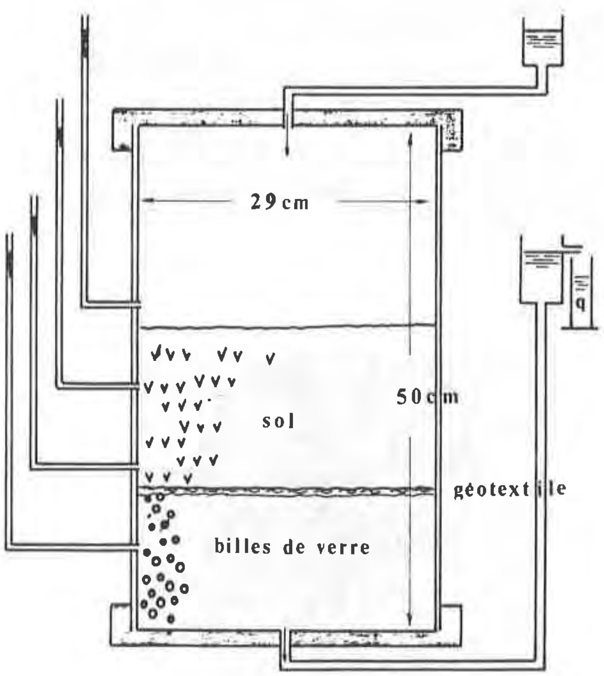

Fig. 4 : Appareil B d'essai de filtre

Trois sols différents ont été testés :

- un limon d'Orly $d_{95}=60\mu, d_{50}=16\mu$, 26% des éléments < 2 µ
- un"sable"de Modane $d_{85}=36\mu, d_{50}=16\mu$ $d_{60}/d_{10}=7$, 7% des éléments < 2 µ
- un sable de Champlan $d_{85}=110\mu, d_{50}=100\mu$ $d_{60}/d_{10}=1,1$, 7% des éléments < 80µ , 2% des éléments < 20µ

Pour tous les essais, la charge étant appliquée par paliers, afin d'obtenir des gradients : le débit, la charge à l'entrée, à la sortie et en des points intermédiaires du sol, étaient régulièrement mesurés.

II.1. Essais sur l'appareil A

Pour les 3 essais effectués, le géotextile utilisé est un Bidim U34 (diamètre de filtration de 125 µ)

Le premier essai a été effectué sur le sable de Modane (6,2 cm d'épaisseur), il a duré 9 mois et a comporté 9 gradients hydrauliques (1,2,3,4,5,6,7,8 et 10). Jusqu'au gradient 8, les variations de débit et de charge aux points intermédiaires marquent une dispersion acceptable à partir de la loi de Darcy. Au gradient 10 (charge de 60cm), deux phénomènes sont apparus : d'abord la charge aux points intermédiaires était, dès l'application de cette charge, voisine de 60 cm et elle a peu varié ; le débit, normal en début d'essai, a diminué lentement pour être divisé par 2 après 15 jours : compte tenu des variations de débit, il y a donc eu transfert de la perte de charge à l'entrée du géotextile (le reste du sable voyant sa perméabilité augmenter), puis un colmatage progressif de cette zone de contact. Il faut souligner que ce phénomène de colmatage immédiat s'est produit à perméabilité globale constante.

Le deuxième essai a porté sur du sable de Champlan (même géotextile) d'épaisseur 7 cm. Pour des gradients de même valeur maintenus 24 h, on peut noter une variation régulière de débit avec le gradient.; par contre, aux points de mesure de charge intermédiaires, celle-ci a à peine augmenté (8 à 10 cm, alors que la charge totale variait de 10 à 70 cm). Dans ce cas, la zone en contact avec le géotextile a perdu ses fines et sa perméabilité a augmenté d'un coefficient 10, alors que la tranche supérieure de l'échantillon gardait une perméabilité constante qui déterminait les variations du débit global.

Le troisième essai a porté sur un limon d'Orly, et les observations faites sur le débit et les pertes de charge sont analogues au cas précédent.

Dans ces trois essais effectués avec une faible couche de sol dans un appareil de diamètre relativement important, les règles de filtre que nous proposons n'étaient pas respectées. Les observations ont montré d'abord qu'aux faibles gradients, il était difficile d'observer la moindre anomalie de comportement ; par contre pour les gradients élevés, on a observé deux phénomènes opposés : un blocage des particules à l'entrée du géotextile (1er essai), un lessivage du sol à proximité immédiate du géotextile (2ème et 3ème essai).

II.2. Essais sur l'appareil B

A partir de sable de Champlan, compacté à l'optimum Proctor sur 20 cm, 4 géotextiles ont été étudiés :

- essai n° 1 : géotextile aiguilleté, réf. N1B-FAP, masse surfacique 433 g/m2, diamètre de filtration 50 µ
- essai n° 2 : géotextile aiguilleté, réf. Bidim U24, masse surfacique 203 g/m2, diamètre de filtration 120µ
- essai n° 3 : géotextile aiguilleté, réf. Sommer n° 13, masse surfacique 254 g/m2, diamètre de filtration 170µ
- essai n° 4 : géotextile aiguilleté, réf. Sommer 2A 17, masse surfacique 484 g/m2, diamètre de filtration 250µ

Les gradients globaux appliqués (0,5, 1, 2, 4 et 8) étaient maintenus selon les cas, entre 3 et 5 jours. L'eau utilisée est au préalable désaérée.

Comme précédemment, les mesures ont porté sur le débit et les pertes de charge. De plus, l'état du géotextile a été examiné non seulement en fin d'essai, mais encore les entrainements éventuels étaient visibles à travers le plexiglas.

Par rapport à la série d'essais précédents, il faut noter une épaisseur de sol 3 à 4 fois plus forte et un véritable compactage de ce sol (95 à 98 % de l'optimum Proctor normal).

La figure n° 5 représente la répartition de la charge le long de l'échantillon de sol, la charge ayant été prise nulle au niveau du géotextile. Les valeurs portées sont celles obtenues en fin de palier après écoulement sous gradient constant pendant 3 à 5 jours. On peut noter que l'incurvation des courbes H (z) s'est accentuée progressivement ; il y a donc eu réduction relative de perméabilité dans la partie supérieure du sol.

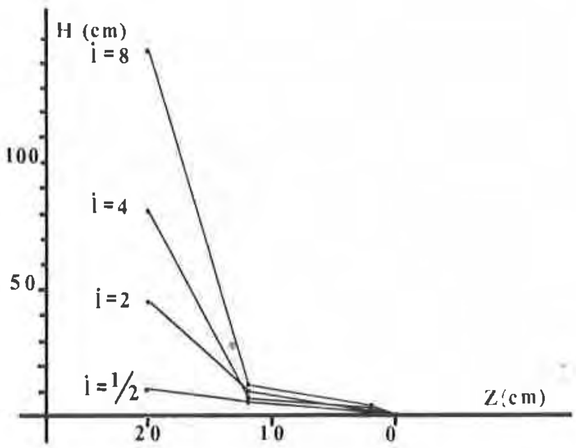

Fig. 5 : Essai n° 1 Variation de la charge dans le sol à protéger.

Les figures 6 et 7 représentent l'évolution du débit en fonction du gradient mesuré localement à l'amont immédiat du géotextile. Chaque courbe correspond à un palier de gradient global.

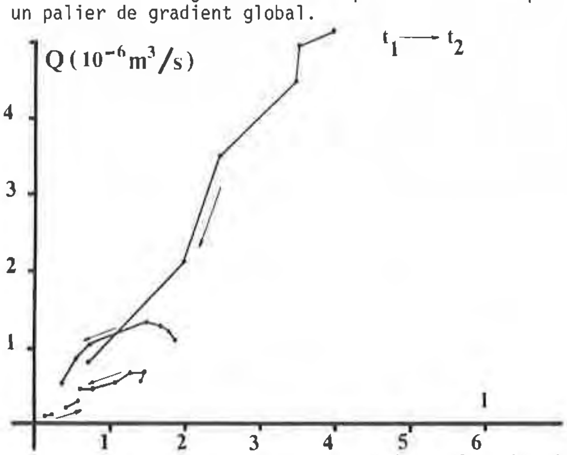

Fig. 6 : Essai n° 1 Evolution du débit en fonction du gradient dans le sol à proximité immédiate du géotextile

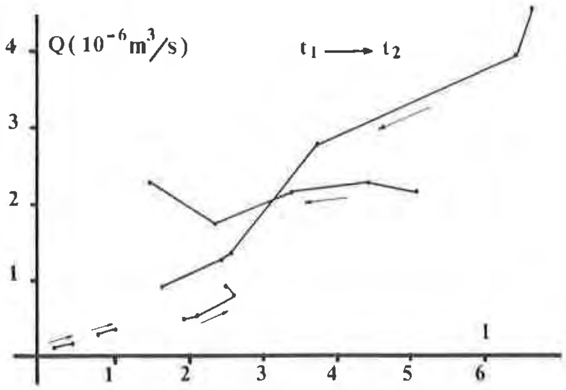

Fig. 7 : Essai n° 3 Evolution du débit en fonction du gradient dans le sol à proximité immédiate du géotextile

De façon générale, aux gradients globaux 1/2 et 1, la relation débit gradient local est linéaire, sauf pour l'essai N° 4 pour lequel un débourrage est constaté au gradient global 0,5. Aux gradients globaux 2 et 4, on constate un certain débourrage en particulier pour le gradient 4 de l'essai n° 3. Systématiquement au gradient global 8, il faut souligner la proportionnalité entre débit et gradient local.

Aucun dépôt solide n'a été constaté, tant au niveau des billes de verre que dans le bac de réception. La figure 8 montre une lame mince effectuée sur géotextile en fin d'essai n° 2, on peut constater la très faible proportion de particules retenues; pour les 3 autres essais, les observations sont analogues.

De ces essais, il faut retenir 3 éléments essentiels : les variations locales importantes de gradient en cours d'essai, l'influence du gradient sur le comportement d'ensemble, la tolérance probablement très grande que l'on peut admettre vis-à-vis des règles de filtre énoncés en § IV.2.

Fig. 8 : Coupe du géotextile en fin d'essai n° 2

III - PERMEABILITE

III.1.Permittivité

Celle-ci variant peu sous contrainte (réduction de l'ordre de 1/2 à 1/3 pour une augmentation de contrainte de 2 KPa à 250 KPa, d'après Gourc) il est suffisant,pour les applications courantes, de mesurer la permittivité sous contrainte nulle.

L'appareil que nous avons développé en commun avec l'I.T.F. permet de mesurer la permittivité, sans contrainte sur un échantillon de 70 mm de diamètre. L'alimentation se fait en eau désaérée et la mesure est faite sous perte de charge constante comprise entre 0,01 et 0,05 m. La difficulté principale pour l'exécution de ces essais réside dans le maillage des échantillons. En particulier, lorsque ceux-ci présentent une certaine hydrophobie, les résultats peuvent varier de façon considérable : par exemple, augmentation dans un rapport de 20 lorsque l'on passe d'un trempage de 10 mn à 48 h. pour un aiguilleté présentant une hydrophobie assez marquée. L'échantillon n'était d'ailleurs pas encore pleinement saturé après 48 h de trempage.

III.2. Transmissivité

Deux types d'appareil ont été développés, en commun avec SOMMER.

- mesure de la transmissivité du géotextile seul (fig.9)

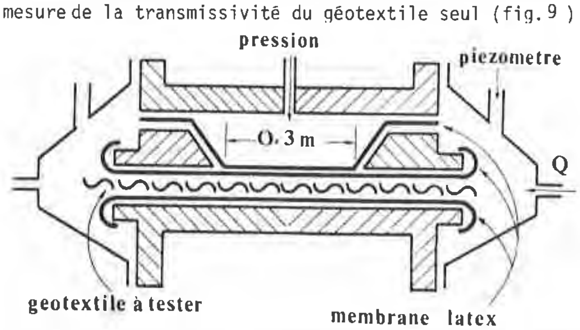

Fig. 9 : mesure de la transmissivité du géotextile seul

L'échantillon de géotextile, de 20 cm de large est placé dans une manchette en latex de 0,3 mm d'épaisseur. La contrainte est appliquée sur une longueur de 0,30 m par une deuxième membrane, gonflée à l'air. L'alimentation de l'appareil est faite en eau désaérée et on procède à la mesure du débit, au débitmètre pour des gradients de 1/3 et de 1, sous des contraintes croissantes. Le débit à vide de l'appareil est pratiquement négligeable et est soustrait de la mesure si nécessaire. Les mesures sont faites après un temps d'application de la contrainte de 1 h minimum.

- mesure de la transmissivité au contact du sol (fig.10)

Le principe de l'appareil est analogue. Le géotextile est placé entre deux couches de sol compactés. Les dimensions utiles de l'échantillon sont 0,16 x 0,30 m. Un intervalle de 0,02 m est laissé entre l'échantillon et les parois latérales pour éviter tout effet de bord. Le sol est comprimé par une membrane gonflée à l'air, les parois latérales du moule sont graissées Le sol utilisé est, soit un sol de référence (limon d'Orly = limon argileux compacte à O.P.N. + 2), soit un échantillon du sol provenant du site pour lequel le géotextile est étudié.

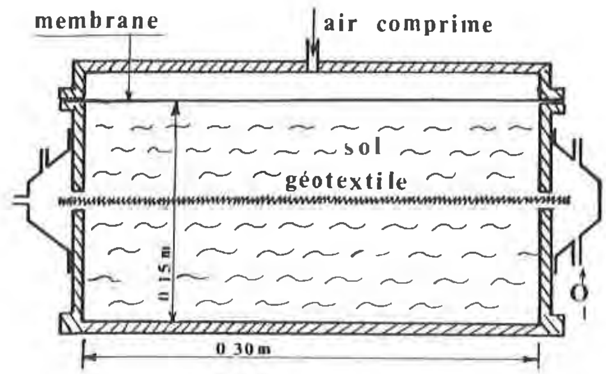

Fig.10: mesure de la transmissivité du géotextile au contact du sol

Les mesures sont faites suivant le même principe que pour l'appareil précédent.

- comparaison entre les 2 types de mesure

Les comparaisons diffèrent notablement en fonction de la structure du géotextile : tissé ou non-tissé

produits non tissés

L'accord est assez bon entre les deux types de mesure si l'on tient compte des remarques suivantes.

Le compactage exercé sur le sol et le géotextile une pression de préconsolidation, de l'ordre de 100 KPa. La transmissivité du géotextile dans le sol est donc, à contrainte faible, sensiblement égale à celle que l'on trouve sous 100 KPa pour la transmissivité mesurée sous géotextile entre membrane. Vers 200 KPa, les valeurs sont sensiblement identiques.

Pour certains sols, comme la craie, on peut observer un lessivage au contact avec le géotextile. Ceci entraine une augmentation apparente de la transmissivité (supérieure à celle mesurée sans sol) et se traduit par un écoulement d'eau très chargée dans le géotextile. Ce phénomène est d'autant plus marqué que le diamètre de filtration du textile est élevé. On peut craindre, dans la pratique, un colmatage du géotextile drainant. L'emploi d'un composite avec couche filtrante fine paraît recommandé dans ce cas.

produits tissés

Les valeurs obtenues entre membranes sont supérieures de plusieurs puissances de 10 à celles obtenues pour un géotextile noyé dans le sol. Ceci peut être attribué à une pénétration de la membrane latex dans la structure du tissé très inférieure à celle du sol.

Il semble donc que les mesures de transmissivité entre membranes et à fortiori entre deux plaques rigides, ne sont pas représentatives de la transmissivité des produits tissés dans le sol. Il semble qu'il en soit de même pour les produits composites composés d'une grille en plastique et d'une couche filtrante fine.

IV - DIMENSIONNEMENT

Remarque générale - mouillabilité des géotextiles

Pour l'emploi dans un rôle hydraulique, il est fondamental que le géotextile présente une bonne mouillabilité, et que celle-ci soit pérenne.

Le non fonctionnement de certains systèmes drainants (drains PVC enrobés de géotextiles) observé en France est attribué de façon certaine, à la mauvaise mouillabilité de l'enrobage.

IV.1. Drainage

Le dimensionnement d'un système de drainage par géotextile se fait par l'application de la loi de Darcy ; $q = \theta i$. Les principes suivants doivent être respectés :

- La transmissivité θ à prendre en compte est celle mesurée sous la contrainte subie dans l'ouvrage : poids des terres et surcharges pour un géotextile horizontal ou incliné (car en pratique, dans ce cas, il y a de fréquents paliers horizontaux), poussée butée ou action des terres au repos pour des géotextiles verticaux. L'action due au compactage du sol doit être prise en compte si elle est supérieure aux contraintes ci-dessus.

- Pour la détermination du gradient i , deux cas sont à considérer

. si l'on désire un drainage parfait (potentiel hydraulique égal à la cote du point), le géotextile doit avoir une pente (angle β avec l'horizontale) et une collecte à la pression atmosphérique en partie basse. Le débit maximal que peut évacuer le géotextile, pour respecter les conditions ci-dessus, est alors
$q_u = \theta \sin \beta$

. Dans les autres cas et en particulier si $\beta \simeq 0$, il apparaît obligatoirement une pression interstitielle u dans le géotextile. A partir de cette valeur, il est possible de calculer la transmissivité, compte tenu de la distance entre le point où u sera maximum et celui où le géotextile sera à la pression atmosphérique, et du débit à évacuer.

Pour le drainage de grande surface (drainage horizontal, drainage sous remblai ...), il peut être intéressant d'intercaler des collecteurs fonctionnant à surface libre constitués par de simples drains plastiques perforés, la transmissivité nécessaire variant généralement comme le carré de la distance entre collecteurs.
Exemples :
Drainage horizontal : (2)

Ce type de drainage peut être intéressant pour les terrains de sports, jardins sur dalles etc... Pour une pluie d'intensité I (en m3/s) et une cote maximale de la surface libre Z_0 au dessus d'un géotextile horizontal, l'écartement L entre 2 collecteurs est :

$$L = 2 \sqrt{\frac{2\,\theta}{I} \left(1 - \frac{I}{K_s}\right) Z_0}$$

- géotextile sous remblai sur sol compressible, GIROUD propose (3) la relation :

$$\theta = \frac{L^2 K_s}{\sqrt{c_v . t}} \quad \text{d'où un écartement } L = \sqrt{\frac{\theta \sqrt{c_v\, t}}{K_s}}$$

t étant le temps de construction du remblai

La comparaison de cette relation avec les simulations faites par différences finies, par Bourdillon (4) montre que cette relation conduit a une augmentation du temps de consolidation inférieure à 20 % par rapport à un drainage parfait. Si les collecteurs sont placés perpendiculairement au remblai, leur section peut généralement être faible et cette solution sera beaucoup plus économique que l'utilisation d'un géotextile de très forte transmissivité.

Drainage vertical dans les barrages

Pour des ouvrages de moins de 20 m et compte tenu de la faible perméabilité des massifs, un seul collecteur est généralement suffisant, en partie basse. Il est recommandé de mettre ce collecteur dans une tranchée drainante. Le CEMAGREF a étudié 3 ouvrages de ce type de hauteur comprise entre 10 et 17 m. Le drainage est assuré par des géotextiles aiguilletés de masse surfacique comprise entre 550 et 700 g/m2.

IV.2. Règles de filtre

Rappelons d'abord que les tests relatifs aux caractéristiques de filtration ne fournissent en fait qu'une caractéristique intrinsèque que nous avons intitulé : diamètre de filtration D_f et que les liens entre granulométrie des vides, porométrie et filtration en sont à leurs premières ébauches.

La plupart des propositions relatives au choix des filtres en géotextile, sont basées soit sur une extension des règles appliquées aux filtres en sols pulvérulents, soit sur un nombre limité d'essais réalisés dans des conditions particulières. Il nous a paru préférable de recommander les règles les plus contraignantes.

- sol pulvérulent
  . sol à granulométrie étalée ayant tendance à être autofiltrant $D_f \leqslant d_{85}$
  . sol à granulométrie étroite $(d_{60}/d_{10} \leqslant 4)$ $D_f \leqslant 0{,}8\ d_{50}$
- sol cohérent
  . mêmes règles sans descendre en dessous de 50 μ

De nombreux auteurs ont proposé ou proposeront des règles moins strictes en cas de faible gradient, de forte plasticité, de coefficient d'uniformité élevé, ou d'écoulement alterné.

Dans le cas des barrages où la sécurité et la pérennité sont des caractéristiques essentielles, nous considérons que la satisfaction des règles proposées ne pose pas, en général, de difficultés particulières, n'implique pas de surcoût notable et apporte les garanties indispensables de maintien de la fonction attendue.

L'attention du projeteur doit être attirée sur le fait que ces règles s'appliquent à des matériaux homogènes (attention à la ségrégation en cas de granulométrie très étalée) et normalement compactés, avec un bon contact sol-géotextile (maintien d'une contrainte normale permanente). Elles ne sauraient être appliquées sans études particulières à des sols difficiles de granulométrie discontinue, évolutifs ou dispersifs.

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De l'approche classique présentée, on peut s'éloigner: un géotextile placé sous un noyau mince fortement incliné vers l'amont peut très bien se colmater entièrement sans provoquer de désordres de l'ouvrage, à condition qu'il n'y ait aucune particule qui traverse le géotextile. Ce mécanisme peut d'ailleurs constituer un dispositif de prévention des renards extrêmement efficace.

Les réflexions devront donc à l'avenir s'orienter d'abord vers l'étude de la porométrie sous contrainte normale effective, du complexe sol-géotextile et vers l'observation d'ouvrages, mais ensuite nous devrions pour les géotextiles imaginer des fonctions nouvelles et spécifiques, en particulier dans les travaux hydrauliques

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Geotextile Filtration Performance and Current Filter Criteria

Les performances de filtration des géotextiles et les critères de filtre courants

The results of measurements of flow rates carried out at ICI Fibres soils laboratory in the UK through systems composed of soils and geotextiles are discussed. The results suggest that the presence of a fabric has little effect upon the permeability of the system. Preliminary results from work at Southampton University lend support to this observation. The concept of wrapping pipes directly with a fabric filter is investigated and advocated only with the use of corrugated pipes. The general trend towards specifying coarser particle sizes for graded aggregate filters in land drainage situations is discussed in relation to current criteria for fabrics.

Les résultats des débits mesurés dans le laboratoire de mécanique des sols d'ICI Fibres, RU, à travers des systèmes sol/géotextile sont discutés. Ces résultats indiquent que le géotextile n'a que peu d'influence sur la perméabileté du système. Des résultats préliminaires de l'université de Southampton renforcent cette observation. L'idée d'enrober directement des tuyeaux de drainage d'un géotextile est considéré et recommandé uniquement pour les tuyeaux ondulés. La tendance générale de prescrire une granulométrie plus grosse pour des matériaux de filtration calibrés utilisés en drainage de terrain est discuté vis-à-vis des critères courants pour les géotextiles.

1 INTRODUCTION

The wide range of materials manufactured as permeable membranes - or "Geotextiles" has been well documented (1). Their uses are wide and varied, and have been classified (2) by virtue of their primary functions - Separation, Filtration, Drainage in the Plane, and Reinforcement.

Geotextiles have been used in a variety of geotechnical 'filter' situations for over fifteen years during which time various design criteria have been developed.

Confidence has grown considerably in the use of these materials although it is often desirable to evaluate their suitability in specific situations.

The application that generated some of the work to be discussed was concerned with the use of geotextiles in place of graded gravel filters as a surround or envelope to pipes used for land drainage. Graded gravel envelopes placed around pipe drains are expensive and difficult to obtain. One example of this was on a large reclamation scheme in the Middle East (3) where an 80 mm diameter corrugated plastics drain pipe was enveloped with a minimum 50 mm thickness of gravel within (although on the coarse side of) an approved grading envelope. A layer of thin non-woven geotextile filter (Terram 140) was then placed on top of the envelope to separate the gravel from the loose trench fill.

The success of the system lent support to the view that the pipes might work satisfactorily if the gravel were omitted, and the pipes were simply wrapped with the geotextile.

The purpose of the paper is to collate results from tests of filter performance in typical land drainage models, to compare various filter criteria and comment on the concept of wrapping pipes directly with a geotextile.

2i MEASUREMENT OF SOIL PERMEABILITY

Permeameter cells are used for studying the passage of water through various soil and filter combinations under different static flow conditions; they are designed basically for measuring flows but also permit visual examination.

Usually in the ICI laboratory, permeameters are set up in batches of 4, using apparatus similar to that illustrated in Fig: 1.

A weighed 200 mm dia disc of filter fabric is bedded down on to aggregate placed in the base, and the glass top firmly clamped down. The soil is placed in layers while the cell is filled with water from below keeping the soil wet all the time to minimise air entrapment and eliminate surface tension effects.

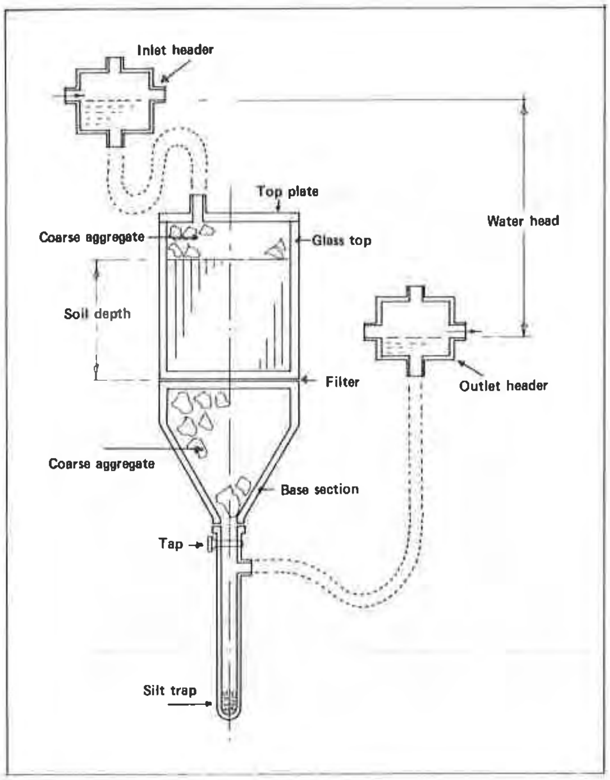

FIG. 1. SECTION THROUGH PERMEAMETER CELL.

After placing the soil, the upper part of the cell is filled with aggregate, the top plate clamped in place, and constant flow maintained with some overflow in the top water header. The flow through the system is measured at intervals and is converted to hydraulic conductivity (m/s) on the basis of Darcy's law. Upon dismantling, all elements are dried and weighed in order to establish particle movement.

2ii FLOW RESULTS FROM PERMEAMETER CELLS

The tests described in this paper were carried out to evaluate the changes in the flow rate which were liable to occur in field situations due to saturated ground-water flow from soil through a geotextile to a highly permeable drainage blanket. Most of the tests were carried out on various combinations of fabric filter and soil types. However, in some situations a 2 mm mesh replaced the fabric in an attempt to create an extreme 'no filter' situation.

In general the flow rate declined rapidly in the initial stages and more slowly in later stages as is shown in Fig: 2, following a pattern noted as being typical (4, 5). Many results deviated from this norm due to compaction and experimental variations but reductions by a factor of 10 in the soils' initial hydraulic conductivities were common.

The experiments were undertaken using normal (as opposed to de-aired)water and the evidence of many researchers' (6) would suggest this was a root cause of the sporadic nature of the results.

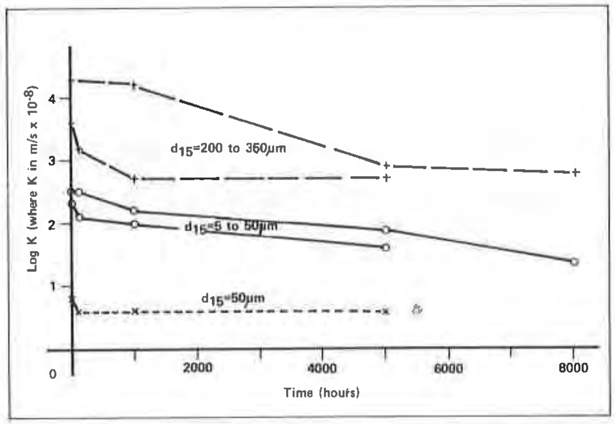

FIG. 2. GRAPH OF (LOG) HYDRAULIC CONDUCTIVITY V. TIME.

Relationships were sought between values of hydraulic conductivity and potential soil size parameters. Fig: 3 however, typifies the rather erratic nature of this relationship between, in this case, hydraulic conductivity and the soils' d_{15} size which other researchers (7) have found to be indicative of hydraulic conductivity.

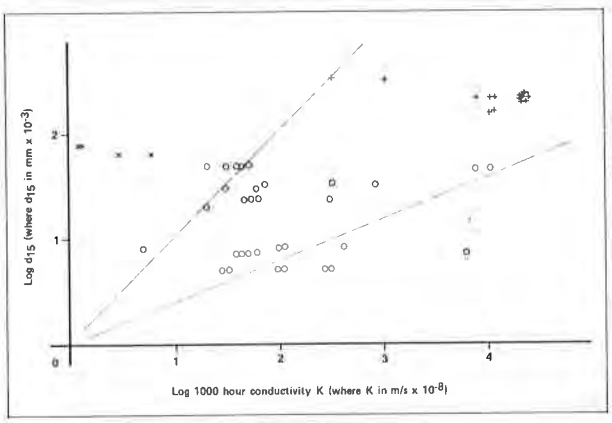

FIG. 3. GRAPH OF (LOG) PARTICLE SIZE V. (LOG) HYDRAULIC CONDUCTIVITY.

In general there was no visual or recorded evidence that the fabrics had in any way become blocked to a degree that had any measurable effect upon the system's overall hydraulic conductivity. However, situations were tested in which fines appeared able to suffose through gap graded soils and settle on the filter to form a layer of reduced hydraulic conductivity.

The conclusions drawn from the many hours of tests on soils drawn from different areas of the world are that:

i Simple tests carried out on disturbed soils can vary in an inconsistent manner.

ii The hydraulic conductivities of the fabric and soil system appear to be controlled almost wholly by the hydraulic conductivity of the soil.

iii In certain soils liable to suffosion the fabric filter can retard the onward migration of fines and lead to a local build-up of material of limiting hydraulic conductivity.

2iii PIPING OBSERVATIONS IN PERMEAMETER CELLS

Piping results from the permeameter cells are limited due to the difficulty of the measurement of particle movement, which is calculated by dry weight changes between assembly and dismantling and therefore depends upon difference measurement.

The results give the impression that significant piping of soils was only evident when a gauze with a nominal 2000 µm opening size (representing the no filter situation) was used. Unfortunately there is a dearth of results between these and the 50 to 350 µm pore size fabrics, and it is difficult to relate the degree of piping to the fabric type and soil type.

Consideration should be given to the degree of particle loss in the model necessary to represent piping (8). Some piping occurred initially in several tests but this was often observed to cease after several hours. The evidence suggested that this initial phase of piping might involve the movement of up to 2-3% of the original soil mass. Quantities of transported soil in excess of 3% were associated with soils in which continuous piping occurred.

On this basis it can be said that only one of the soils tested significantly piped through a filter cloth. This was a particularly mobile silt with d_{15} = 10 µm and C_u (coefficient of uniformity) = 4. Conversely several of the soils tested appeared to suffose through the gauze, although to varying degrees, as may be expected (9).

3i THE FEASIBILITY OF WRAPPING PIPE DRAINS WITH THIN GEOTEXTILE FILTERS

Seepage towards pipe drains constitutes a very testing hydraulic condition in the sense that all the excess water from a relatively large area of land (typically 50-100 m^2/m length of pipe) has to converge and flow through a very small area of soil adjacent to the pipe in order to gain access to the pipe (typically $10^{-3} m^2$/m length).

Most of the available head generating flow to the pipe is lost in the region of convergence near the pipe giving rise to high hydraulic gradients (up to 8) (10). It follows that the conditions at, and near to, the pipe are of particular importance since they influence this headloss.

The total headloss, as flow occurs from a water table to a pipe, may be schematically subdivided into losses due to vertical, horizontal and radial flow (11). Each of these flow zones may be described in terms of a resistance, though the only ones that may be influenced by the design of the pipe and its surrounding envelope are the components of the radial flow, namely: convergence/radial resistance, Wr, and especially the entrance resistance, We (12) where:

We = entrance head loss due to the inflow to the pipe of a unit discharge/unit length of pipe. Units $T\ L^{-1}$.

The entry resistance can also be expressed in a dimensionless form: α = We K

where K = hydraulic conductivity of the surrounding soil/envelope through which water passes to gain entry to the pipe.

Typical values of α are presented in Table 1:

Nature of Pipe	Dimensionless Entry Resistance α	Source
Clay tile drain	1.6 - 2.3	(13)
Smooth plastics	0.4 - 2.6	(14) (15) (16)
Corrugated plastics	0.02 - 0.04	(17)

Table: 1 Typical measured values of the dimensionless resistance factor.

The values reflect the different entry characteristics of pipes. Clay pipes have a small entry area poorly distributed in the form of gaps between the pipes. Smooth plastics pipes, at least in Europe, have a minimum entry area of 800 mm^2/m length whilst corrugated pipes tend to have a larger area (typically 2000 mm^2/m length) which is favourably distributed in the form of small slots.

The entry characteristics of pipes tend to be ignored in the design process since the water table midway between the drains is not affected to a significant degree by the head loss at entry (until this exceeds 0.1-0.2 m which corresponds to values of α of 0.25-0.36 respectively) (12).

In general it may be concluded that clay and smooth plastics pipes have a somewhat higher resistance than the implied minimum limit of 0.25.

The effect of wrapping pipes with the sort of thin geotextiles which only permit flow across the plane of the fabric would in the case of the clay tile drain and the smooth plastics pipe lead to a further reduction in the entry area leading to increased resistances. This in itself would be inadvisable especially as subsequent partial clogging of the filter above the entry areas might lead to even further increases in resistance.

The effect on the corrugated pipe could be different, for two reasons. Firstly, the basic resistance of this type of pipe is lower than the suggested limit. Secondly, the effect of bridging the grooves between the corrugations of a pipe slotted on the inner grooves would increase the area of the interface bewteen the soil and the pipe.

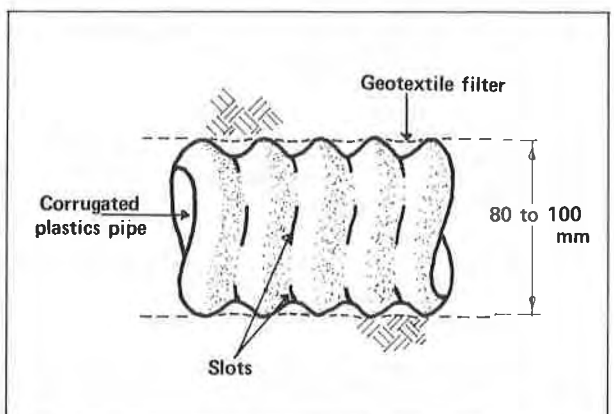

FIG. 4. SCHEMATIC OF WRAPPED CORRUGATED PIPE.

The few published values of the resistance α for pipes wrapped in this matter, suggest it to be in the range of 0.02-0.09 (18, 19).

3ii EXPERIMENTS TO DETERMINE THE EFFECT OF HYDRAULIC CONDUCTIVITY OF A GEOTEXTILE WRAPPED PIPE

Experiments were carried out in a sand tank, similar to that described by Knops (18), and illustrated in Fig: 5.

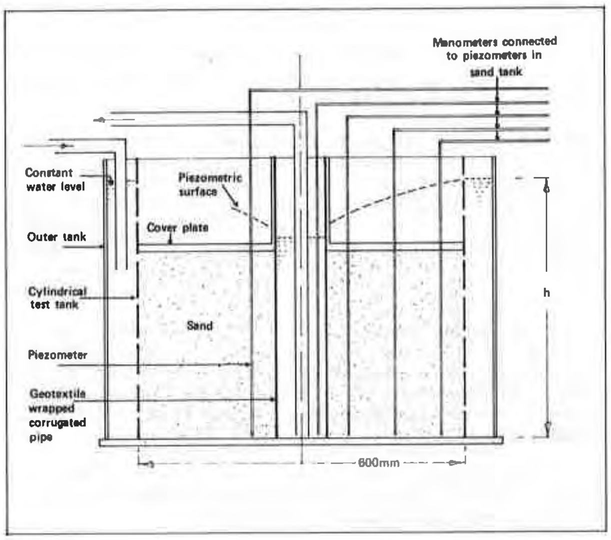

FIG. 5. SECTION THROUGH SAND TANK APPARATUS.

The objective of these tests was to identify the reduction in the hydraulic conductivity of the geotextile that would be necessary to reduce the resistance to the arbitrary limit of 0.25. The tests were carried out using a highly permeable coarse sand, while the geotextile's hydraulic conductivity was reduced by covering the wrapped pipe with porous paper, to simulate clogging. Measurements of discharge, headloss at entry, and piezometric levels enabled the entrance resistance 'We', the hydraulic conductivity of the sand K and the dimensionless resistance to be determined.

The hydraulic conductivities of the combinations of fabric and paper were determined at the conclusion of these tests by transferring a 100 mm ø dual wafer to a separate permeameter illustrated in Fig: 6. This apparatus enabled the conductivity to be determined under low hydraulic gradients (<6) theoretically preserving Laminar Darcian flow.

The test programme has not yet been completed and the results presented in Fig: 7 must therefore be regarded as being provisional, and the conclusions of a preliminary nature. (The data has been presented in this dimensionless form since theoretically α is functionally dependent upon the ratio K-fabric/K-sand.)

The data indicates that the limiting resistance of 0.25 corresponds to a conductivity ratio of 0.15.

3iii DISCUSSION

This result may be extended to drained lands where, typically, the hydraulic conductivity of the soil around the pipe might be of the order of 1.m. day $^{-1}$. In this situation the limiting value of α would occur when the hydraulic conductivities were reduced to 0.15 m.day^{-1}.

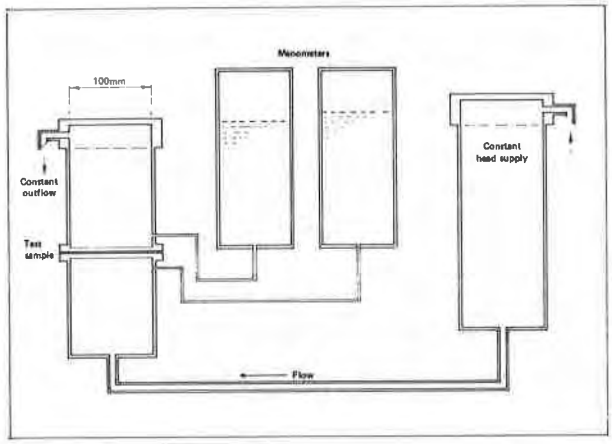

FIG. 6. APPARATUS TO MEASURE FABRIC HYDRAULIC CONDUCTIVITY UNDER HYDRAULIC GRADIENTS.

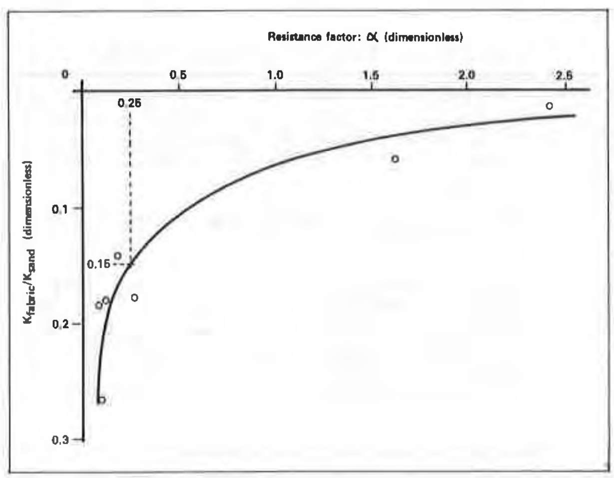

FIG. 7. RELATIONSHIP BETWEEN RESISTANCE AND FABRIC/SAND PROPERTIES.

As initial fabric hydraulic conductivities in this instance were approximately 100 m.day^{-1} we may conclude that a 500 fold reduction of the initial conductivity would be required in the field situation to bring about the limiting resistance (α).

It is clear that the thin geotextile used under the conditions described can suffer considerable clogging without affecting the performance of the drainage system.

These results, however, may also be considered as showing that very little change would be needed to the hydraulic conductivity of the soil immediately surrounding the pipe to reduce the overall resistance to the suggested limit of 0.25.

It seems fair to conclude that it is highly unlikely that the fabric would clog to a significant degree without the soil adjacent to the fabric also clogging to a similar degree; since the soil's initial hydraulic conductivity is much lower than the fabric's, the effects of soil clogging would be that much more serious. The results suggest that our attention should be directed much more towards the hydraulic properties of the soil adjacent to the filter and less at the permeability of the fabric. However, the choice of fabric indirectly influences the soils properties in the sense that fines moving in the soil may either pass through the filter and improve soil hydraulic conductivity or they may be retained in the soil near the fabric, possibly reducing soil hydraulic conductivity. The choice of fabric could therefore be vital to the conditions which will develop in the soil.

4i FILTER DESIGN METHODS.

The design criteria for granular filters for land drainage have evolved in recent years towards coarser, more permeable, materials. Fig: 8 illustrates the potential filter envelopes which could be advocated for protecting the silty soil that gave rise to this initial interest in the applications of fabrics to land drains. These represent a chronological progression of four widely used criteria: Cedergren, 1967 (20); USDA 1973 (21); USBR 1978 (22); FAO 1980 (23).

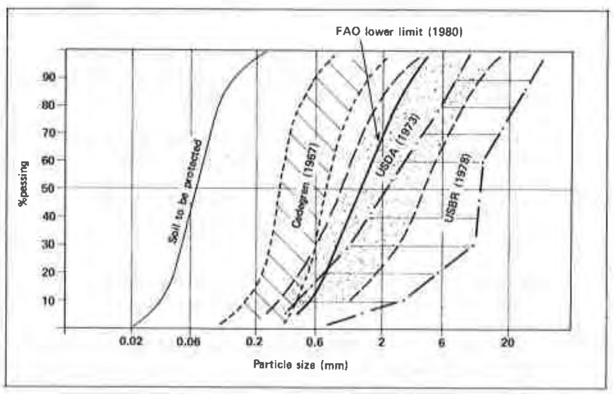

FIG. 8. PARTICLE SIZE RANGES OF VARIOUS GRANULAR FILTER CRITERIA.

The figure clearly shows the modern trend to specifying coarser filters thereby emphasising the importance of the hydraulic performance of the soil filter system and lessening the importance of the strict filtering ability.

Similarly four potential fabric filter criteria have been related to the same soil, namely those proposed by Ogink 1975 (24); Designing with Terram 1977 (25); US Corps of Engineers 1977 (26) and Schober and Teindl 1979 (27). These criteria relate to pore size and their resulting designs are illustrated in Fig: 9 where they are contrasted with the range of pore sizes of potential gravel envelope designs.

4ii PORE SIZE DETERMINATION

The measurement of aggregate and fabric pore sizes is difficult, and comparing the results obtained from different test methods a somewhat uncertain procedure. The pore sizes of the gravel filters were estimated by dewatering the aggregates with increasing tensions. The capillaric relationship between tension μ and pore diameter ($d = 0.3/\mu$ cm) was then used to estimate pore sizes. In view of the limitations of this indirect method of determining pore sizes, the lower end of the range can at best be considered indicative. Similar

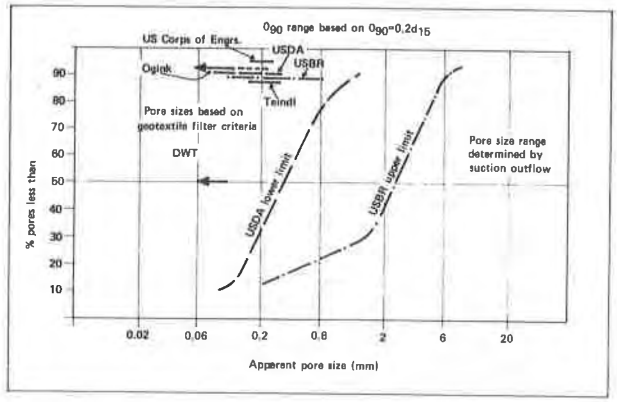

FIG. 9. PORE SIZES OF VARIOUS FILTER CRITERIA.

problems are encountered measuring the pore sizes of fabrics, since direct measuring optical methods can only be employed with woven fabrics with regular mesh size openings.

Various indirect measuring techniques have been devised, the most commonly used being the "dry sieving" method reported by several researchers (8 24 25 28). Such methods are usually specified to relate to a particular design method, the apparent opening sizes again being indicative and relating only to the larger pore sizes. Moreover, other factors may need to be considered when estimating effective fabric pore size - such as the compression of thick fabrics, or the possible opening up of loose weaves under stress.

4iii COMPARISON OF FILTER CRITERIA

The measured pore sizes of the two gravel filters appear considerably larger than the equivalent fabric filter pore sizes, though they should not be compared directly. In a gravel filter it seems logical to suppose that some fine particles are likely to encounter some of the finer pores, though this will not prevent water from moving around a blockage within the thickness (usually $>$ 75 mm) of the filter. In a thin fabric the fines are liable either to pass or to be entrapped thus limiting or even preventing the passage of water through the blocked pore. In view of this it may be that the critical pore sizes for filtration should be based upon the larger pore sizes of fabrics and the smaller pore sizes of gravels.

5 CONCLUSIONS

In conclusion the following comments appear relevant to the use of fabrics in land drainage applications:

i the performance of soil is a complex phenomenon and neither hydraulic conductivity nor piping potential can be accurately recognised from particle size alone.

ii bands of low hydraulic conductivity can occur in gap graded soils due to suffosion.

iii in hydraulic conditions where initial filter K $\gg$ Soil K, fabric filters themselves are unlikely to clog significantly.

iv the hydraulic conductivity of disturbed soils is likely to reduce in time and this may be accelerated by the choice of filters which are "too tight".

v permeameter tests indicate that the available filter fabrics ($O_{90} < 350$ um) only rarely permit soil piping.

vi by taking the coarest option from the design method chosen, soil blocking will be minimised, and fabrics can be used with greater confidence in non homogeneous soils.

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Evaluation of Geotextiles as Liquid Filter

Evaluation de géotextiles en tant que filtres pour liquides

Laboratory filtration tests were conducted on seven filter media (two non-woven and four woven geotextiles and a wire screen). Two commerically available clay products and water of low and high ionic strength were used to prepare suspensions with reproducible properties. The total discharge, the head loss across the filter medium, and the solids removal efficiency were monitored during each test. Values of a filterability index were computed. Although this index was originally developed to rate the performance of granular media, its application to geotextiles yielded values consistent with the performance of each medium as described by observations of discharge velocity, head loss, clogging, and solids removal efficiency. The best overall performance was observed for the non-woven geotextiles. The performance of the 5 μm wire mesh screen appeared to be similar to that of non-woven geotextiles. In general, all four woven geotextiles had the poorest performance, in relation to the other media, under all conditions of suspension, concentration and water chemistry.

Des tests de filtration en laboratoire ont ete entrpris sur sept moyens de filtration (deux non-tisses, quatre tisses et un crible metallique). Ont ete utilises deux produits d'argile disponibles au marche, aussi bien que de l'eau possedant une action ionique basse et elevee, afin de preparer des suspensions avec des qualities reproductibles. La decharge totale, la perte de pression au long du moyen de filtration, aussi bien que l'efficience d'eloignement des solides, ont ete controlles pendant chaque test. Les valeurs d'un indice de filtrabilite ont ete computes. Malgre le fait que cet indice a ete developpe a l'origine pour evaluer la performance du moyen grannleux, son application aux geoxtiles a apportu des valeurs conformes avec la performance de chaque moyen, tel qu'il est decrit par des observations de velocite de decharge baisse de pression, encrassement, et efficience de eloignement des solides. La meilleure performance de tous les moyens a ete observee aux geotextiles non-tisses la performance de la crible de 5 μm est apparue similaire de celle des geotextiles non-tisses. En general, tous les quatre geotextiles tisses ont en la performance le plus faible.

INTRODUCTION

Filtration is one of the main functions of geotextiles. It is the process of allowing the fluid to flow through the fabric while retaining suspended soil particles. Filter cloths have been in use for a long time (1) as components of mechanized filtration systems which may require relatively large pressure gradients. Such systems may be used for dewatering thick slurries (vacuum filters, filter presses, and belt filters) or for clarifying waters with very low concentrations of suspended solids (wedge wire screens, microscreens, and precoat filters). For these cases, very few quantitative design criteria are available for selecting the correct or best filter cloth and designers rely mostly on empiricism, experience, and qualitative charts or criteria (2). For non-mechanized, low maintenance filtration systems, even less information exists on the capabilities of filter cloths to perform as components of the system.

In recent years, the state-of-the-art in designing with geotextiles has substantially advanced (3, 4, 5, 6). In order to evaluate the performance of a geotextile with respect to some of its functions, such as drainage and filtration, it is necessary to obtain its hydraulic properties. The most important hydraulic properties are: permeability, porosity, pore size and distribution, soil retention, level of clogging, and cake formation (7). However, standard methods for testing these properties or even interpreting data from tests are really not available. It has been proposed (7) that the hydraulic properties be determined either relative to the intended use of the geotextile or exclusively on the basis of fabric structure.

Geotextiles are considered herein as shallow or surface filters with a limited capacity for storage of solids. For a single layer woven geotextile the filtration, if effective, results in a cake build-up on the surface. A nonwoven geotextile is a filter with a very shallow depth which is on the order of 10 to 100 times the pore or fiber diameter. Tests on both types of geotextiles are reported herein.

Investigators in the area of filtration have attempted to bridge the gap between theory and empiricism by developing rating techniques and proposing indices for describing the suspension filterability of an arbitrary filter medium. Most of these attempts have been made for application to granular filters. Methodologies proposed for filter cloths (for example, 8, 9) are severely restricted principally because commercially available products come in a wide variety of fabric structures and geometries.

Geotextiles have numerous applications as liquid filters or solid-liquid separators, including settling ponds, groundwater recharge, hydraulic filling, silt fences, silt curtains, and clarifiers for dredging containment area effluents. This work is intended to describe retention and clogging characteristics of several geotextiles challenged with clay-silt suspensions with the results pertinent to one or more of the above applications.

EXPERIMENTAL PROGRAM

The experimental program presented herein was conducted as part of an extensive investigation of filtering systems for dredged materials containment area effluents. Accordingly, the selection of test variables and ranges of these variables was influenced by field performance requirements and the expected range of operating conditions. It is believed, however, that the results obtained provide insight into the performance of the geotextiles tested as liquid filters for dilute suspensions and allow their comparison to other solid-liquid separation devices.

The equipment used for conducting the laboratory tests consisted of a suspension storage tank with continuously operating mixers, a variable speed pump, upstream and downstream overflow tanks for constant head operation, a specially designed filter chamber, a manometer board, and a flow meter. The overall arrangement, as well as some of the structural details of the equipment, is shown in Figures 1 and 2.

The suspended solids in the influent suspensions consisted of commercially available clay soils of two different types: kaolinite and illite (Grundite). The kaolinite was a water-processed hydrated aluminum silicate clay known as Hydrite-R marketed by the Georgia Kaolin Company of Elizabeth, New Jersey; the illite was the principal clay mineral in the clay soil marketed under the name Grundite by the Illinois Clay Products of Joliet, Illinois. The grain-size distributions of these two clays, obtained by hydrometer analysis without the use of a dispersing agent, are shown in Figure 3. Both of these clays possess low levels of soluble salts and certainly do not solubilize significantly. The clays may undergo flocculation/deflocculation depending on solution chemistry.

Suspensions were prepared using 1 g/L and 10 g/L dry solids suspended in either fresh or salt water. Fresh water tests were conducted by using Evanston, IL tapwater (ionic strength about 2×10^{-3}). The salt water used for testing (ionic strength about 0.5) had the same ionic composition of the tap water except for the addition of 30 g/L of a commercial-grade granulated sodium chloride (NaCl). Since varying periods of time may be required for the properties of clays to equilibrate with their aquatic environment, the pH and electrophoretic mobility of the clay suspensions were measured as a function of time. Neither mobility nor pH showed much variation in a typical 8 hour test. The mobilities were negative and generally small (1.5 and 1.9 μm/s/volt/cm for kaolinite and Grundite, respectively) in tap water and approached zero in salt water.

The characteristics of the filter media used are summarized in Table 1. The media were selected in order to obtain variation of media material, pore size, and weave pattern. The average pore size given for media "A" and "B" was obtained from available literature (5) and represents the equivalent opening size (EOS) of the media; pore sizes given for media "C" and "F" are the average minimum dimension obtained by a calibrated laboratory microscope; the specifications for material "E" were supplied by the manufacturer.

Reproducible "standard" procedures were used to prepare the artificial suspensions, conduct each test, and analyze the samples taken. Each test was conducted for a specific combination of filter medium, type of water, and type and concentration of suspended solids. All tests were conducted under constant head rather than constant flow rate since this may be more typical of practice. Each test was started with a flow rate of 16.7×10^{-6} m^3/s but the permeability of the specific filter medium dictated the initial head.

After the equipment was prepared and inspected, the suspension was pumped into the upstream constant head tank and allowed to flow by gravity through the filter cell to the downstream constant head tank. Samples of the influent were taken at the beginning, the end, and periodically during each test. Effluent samples were taken at frequent intervals during the first hour of testing and every hour thereafter. The flow rate and water levels in the piezometer tubes were recorded whenever filtrate samples were collected. Each test was terminated either after eight hours of operation or when the flow rate decreased by approximately one order of magnitude from the initial rate.

For each filter medium, influent and effluent were compared to obtain the size and mass removal efficiency. The number of suspended particles of various sizes was obtained by use of a Model A Coulter counter interfaced with a Nuclear Data multichannel analyzer. The mass removal efficiency was evaluated on the basis of gravimetric determinations and turbidity readings; for this purpose, a correlation between turbidity (NTU) and suspended solids concentration was developed for both the suspensions used (10). Both size and mass removal efficiency measurements required substantial dilution of samples. This resulted in added uncertainty in case of very high or very low removal efficiency as noted later.

RESULTS AND DISCUSSION

The results obtained from this experimental investigation are presented in summary in Table 2; this gives the suspension type, concentration of suspended solids, water chemistry, discharge rate, head loss, flow time, and integrated removal efficiency for each test. Basically, three parameters were used to evaluate the clogging and/or blinding tendency of a given filter medium: (a) the run duration, (b) the reduction in discharge velocity, and (c) the rate of head loss increase across the filter medium. The value for mass removal reported in Table 2 is the mean for the duration of each test. Size removal efficiency is the mean number removal for particle sizes between 1.0 μm and 5.5 μm.

In general, mass and size removal efficiencies were quite low and run times long. The apparent contradictory observation of low size removal yet eventual filter clogging suggests that the coarsest sizes in the suspension are responsible for clogging of the filter.

Increasing influent concentrations from 1 g/L to 10 g/L had varied effects on filter performance. In every test conducted with a high concentration of Grundite, immediate clogging of the filter due to sedimentation and blocking of the face was observed. Similarly, high concentrations of kaolinite caused rapid clogging (run lengths less than 1000 s) in all but the two non-woven fabrics. The run times, in many cases, became so short that it was impossible to obtain samples for determining mass removal efficiencies. In other tests, the mass removal was such that the measured removal efficiency was beyond the accuracy of the experimental method. As expected, high salt concentration suspensions, in general, had increased removal efficiencies compared to tap water suspensions as a result of flocculation of the clays in the salt water.

Comparison of results obtained during this investigation with known data using similar suspensions was possible only for the non-woven media A and B and is presented in Figure 4. According to the information provided (11), a suspension of loess with a concentration of 7 g/L was used to challenge the filter cloths. It can be observed that the results of this

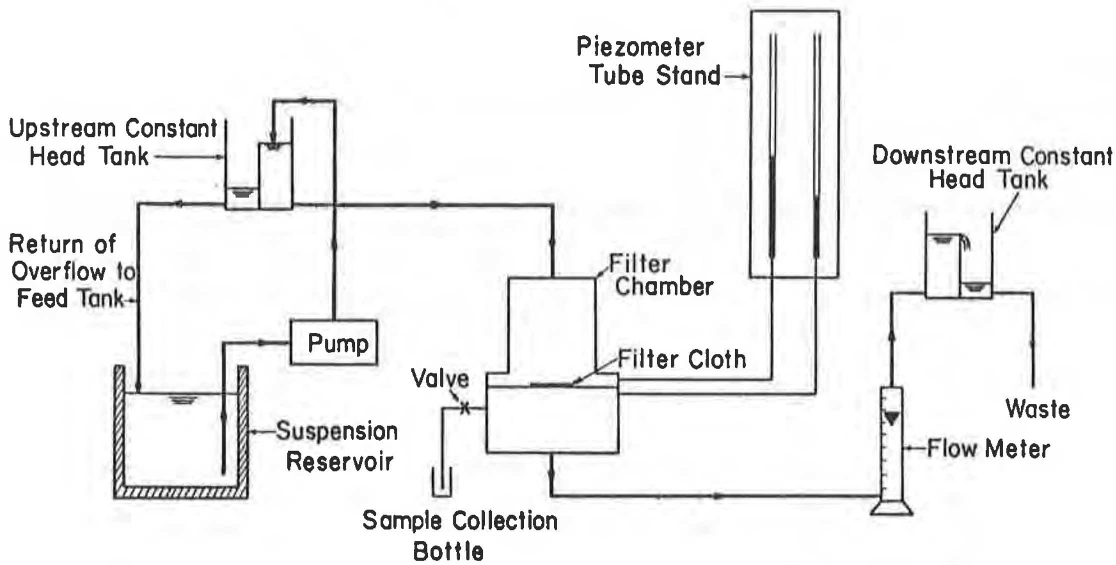

Figure 1. Arrangment of Equipment for Laboratory Testing.

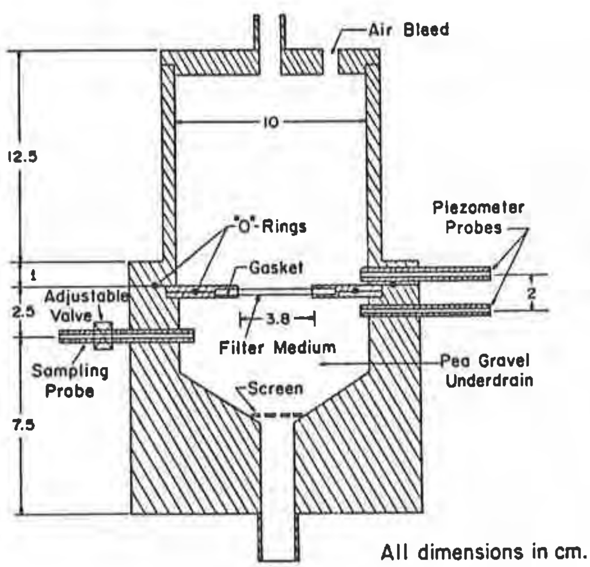

Figure 2. Structural Details of Testing Apparatus.

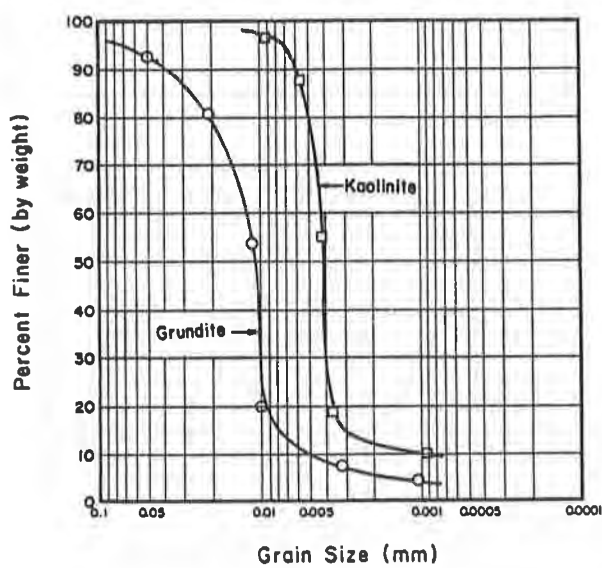

Figure 3. Grain Size Distributions of Suspended Solids

investigation would suggest higher removal efficiencies in the 15-20 μm size range than observed by Monsanto (11); however, results presented herein are not inconsistent.

Based on the results summarized in Table 2, the following observations can be made with respect to the different types of filter media tested:

Non-woven--These materials generally showed the best overall behavior; for run durations ranging from two hours to seven hours, the mass removal efficiency was found to range from 2% to 29% and the size removal efficiency for 1.0 μm to 5.5 μm diameter particles ranged from nil to 56%. Medium A had longer runs before clogging, smaller head loss build-up, and slightly lower removal efficiencies than medium B. Neither medium performed well with suspensions having high influent solids concentrations; immediate clogging was experienced for Grundite suspensions and the performance was even poorer for kaolinite suspensions. For low concentration suspensions the performance of both media was better for either suspension type of water chemistry.

Wire Screen--The performance of the wire screen tested (5 μm opening size) appears to be similar to that of the non-woven media tested under similar conditions.

Table 1

Characteristics of Filter Media

Medium Identification	Average Pore Size (microns)	Weave Pattern	Basic Material	Manufacturer
A	180	Random Fiber	Polyester Homofilament	Monsanto Company
B	150	Random Fiber	Polypropylene Homofilament and Nylon Heterofilament	Celanese Corporation
C	5		Stainless Steel	Cambridge Wire Cloth Company
D	-	2-2 Twill	Multifilament	Lamports Company
E	29	1-1 Plain	Monofilament	Tetko Inc.
F	50	2-2 Twill	Monofilament Polypropylene	National Filter Media Company
G	-	2-2 Twill	Multifilament Polypropylene	National Filter Media Company

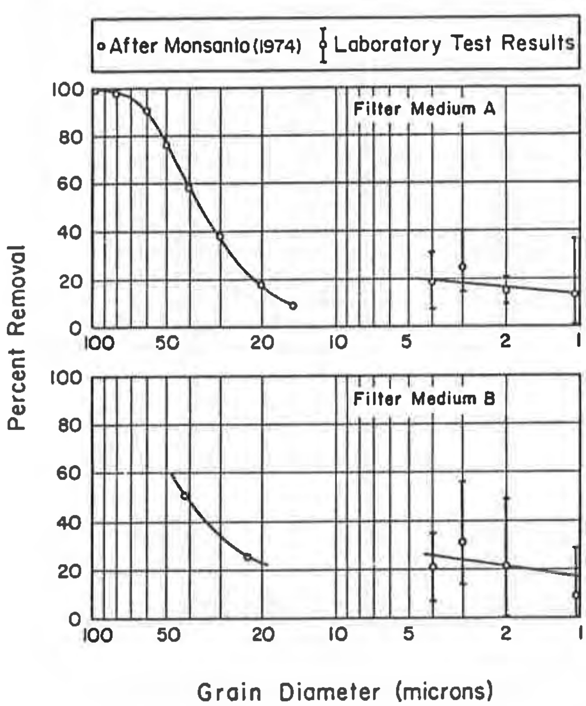

Figure 4. Comparison of Results with Known Data for Media A and B.

Immediate clogging was experienced for high concentration suspensions. At low concentrations the mass removal efficiency was about 15% and size removal efficiency ranged from 1% to 26%; the rate of change of flow rate with time was much smaller than for non-woven media and there was no appreciable increase in head loss with time.

Woven--In general, all four media tested had poor performance (immediate clogging or very short run time) under all conditions of suspension concentration and water chemistry. Medium F (pore size of 50 μm) can be considered an exception for the case of kaolinite suspensions; it had a mass removal efficiency of 8% to 15% and sustained a run time of two hours to three hours.

Over the last 50 years a number of efforts to develop filter indices have focused primarily on application to deep bed rather than surface filters. Sought is an index to facilitate prediction of the behavior of an arbitrary fibrous filter medium. Early attempts to develop such an index resulted in the formulation of expressions which incorporated only one or two of the many variables encountered in cloth filtration. In most cases the dominant variable used was the head loss across the filter. Efforts to incorporate certain cloth characteristics such as pore size and pore geometry in the index (8, 9), were unsuccessful because they required standard pore configurations and could not account for multifilament fabrics or randomly oriented fibers (non-woven media). Furthermore, it was recently concluded (7) that the hydraulic properties of a geotextile are mainly affected by the method of construction and not by the type of material (polymer) used.

Filterability indices have been proposed for a long time (12) to describe granular filter performance; none have found wide applicability or acceptance. However, an index originally proposed by Hudson (13) was developed to the point where a commercial device mar-

Table 2

Results of Laboratory Filtration Tests

Filter Medium	Suspension	Type of Water	Concentration (g/l)	Duration of Test (hours)	Discharge Velocity (cm/sec) Initial	Discharge Velocity (cm/sec) % Change	Head Loss (cm) Initial	Head Loss (cm) Final	Removal Efficiency (%) Mass	Removal Efficiency (%) Size	Filterability Index (10^4)
A	Kaolinite	Fresh	1	4.5	1.46	40	3.56	--	2	ND	7
			10	6.0	1.46	80	3.30	4.83	ND	17-40	14
		Salt	1	4.0	1.46	45	1.02	2.54	4	10-17	5
			10	3.0	1.46	93	1.52	4.06	ND	11-20	32
	Grundite	Fresh	1	7.0	1.46	59	2.03	8.89	9	0-8	8
			10	IC	—	—	—	—	—	—	—
		Salt	1	5.5	1.46	88	2.54	5.84	14	21-31	21
			10	IC	—	—	—	—	—	—	—
B	Kaolinite	Fresh	1	2.0	1.46	95	3.05	7.87	20	29-51	82
			10	2.0	1.46	93	2.29	10.16	ND	0-56	150
		Salt	1	6.5	1.46	87	4.57	8.38	ND	0-16	27
			10	5.0	1.46	90	4.83	7.87	19	9-28	37
	Grundite	Fresh	1	6.0	1.46	87	7.87	11.43	5	12-35	44
			10	IC	—	—	—	—	—	—	—
		Salt	1	3.0	1.46	91	12.45	16.00	29	0-14	161
			10	IC	—	—	—	—	—	—	—
C	Kaolinite	Fresh	1	3.0	2.05	37	23.88	26.67	16	2-20	64
			10	4.0	1.46	32	14.48	14.48	2	1-26	32
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
	Grundite	Fresh	1	4.0	1.46	42	13.72	—	10	1-8	27
			10	NT							
		Salt	1	4.0	1.46	46	11.18	—	15	ND	22
			10	IC	—	—	—	—	—		—
D	Kaolinite	Fresh	1	0.5	1.46	92	16.76	24.64	6	0-9	1926
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
	Grundite	Fresh	1	IC	—	—	—	—	—	—	—
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
E	Kaolinite	Fresh	1	IC	—	—	—	—	—	—	—
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
	Grundite	Fresh	1	IC	—	—	—	—	—	—	—
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
F	Kaolinite	Fresh	1	NT							
			10	3.0	1.46	90	1.52	2.79	8	7-30	14
		Salt	1	NT							
			10	2.0	1.46	91	4.57	6.10	15	10-27	62
	Grundite	Fresh	1	2.0	1.46	92	4.57	7.37	12	8-16	107
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—
G	Kaolinite	Fresh	1	2.0	1.46	91	4.57	8.38	4	0-25	95
			10	NT							
		Salt	1	NT							
			10	0.67	1.46	90	42.16	43.69	25	4-37	1674
	Grundite	Fresh	1	NT							
			10	IC	—	—	—	—	—	—	—
		Salt	1	NT							
			10	IC	—	—	—	—	—	—	—

Notes: IC = Immediate Clogging ND = Not Determinable NT = Not Tested keted in the United Kingdom was designed for its determination. This filterability index (12), FI, is computed as

$$FI = \frac{hC}{vtC_o} \qquad (1)$$

where h is the head loss across a granular filter layer, v is the discharge velocity, C and C_o are the effluent and influent suspended solids concentrations, respectively, and t is a certain time period over which the measurements are made. It can be seen that FI is dimensionless and independent of filter depth. High (poor) values are obtained for high head loss or poor filtrate, and low (good) values result for high flow rates, long run times, or low effluent concentrations.

Equation (1) is employed to the results of this study and numerical values of FI are given in Table 2. Note that each variable needed in the determination of the index is time dependent; thus, a weighted averaging was used for each test to develop a single representative value for each parameter over the duration of the test. The averaging technique is expressed mathematically as

$$B_{avg} = \frac{\Sigma\ B(t)\ \Delta t}{\Sigma\ \ \Delta t} \qquad (2)$$

where B(t) is any one of the time dependent variables at the end of a time increment Δt.

Most recent efforts to utilize indices for depth filters require analysis for behavior in a time element and thus the application of Equation (1) to surface or fibrous filters (geotextiles) is not farfetched. In fact, the index (a) was developed on the basis of "single pass" tests, and (b) incorporates the effects of head loss, flow quantity, and mass removal efficiency. Relatively shallow sand layers (30mm thick) were recommended for evaluation of the index. All in all, the index when applied to fibrous media (Table 2) yielded values that were consistent with the performance of each medium, as described by any of the other parameters listed in Table 2. However, these latter values should be considered only as a guide to the performance of a given filter medium; they do not yield information on the time variation of efficiency, blinding vérsus cake build-up on the medium, or the nature of the breakthrough (if it occurs). Nevertheless, the filterability index can constitute an effective basis for rapidly comparing various filter media or operating conditions.

The results obtained from a filtration test and the performance of a fibrous filter medium are a very sensitive function of the geometric characteristics of the medium. These characteristics include pore size, pore size distribution, porosity, weave pattern, and individual fiber geometry. Obtaining accurate measurements of the effects of these parameters is not possible at this time since standard methods for fibrous filter media rating have not been advanced yet. However, it can be observed that the effects of some, if not all, of these parameters are incorporated into the filterability index presented above, even if only implicitly. For example, a very tightly woven fabric may have small pore sizes and low porosity; neither of these parameters appear in the filterability index but they should be expected to have a significant effect on the removal efficiency, head loss, and/or run time of a test; all of these effects are apparent and incorporated in the filterabilty index. It can be seen that tightly woven fabrics have higher FI than the non-woven varieties on Table 2 independent of suspension and water type.

CONCLUSIONS

Based on the limited experimental investigations reported herein, the following is concluded:

1. In general, geotextiles appear to be poor candidates for use as components of non-mechanized, low maintenance filter systems.
2. Non-woven geotextiles appear to have a better overall performance than woven geotextiles.
3. An indexing technique based on measurements of head loss, flow rate, and mass removal efficiency shows promise of at least effectively screening or rating different geotextiles as liquid filters.
4. Because of the high degree of complexity which exists in the behavior of geotextiles as liquid filters, it is suggested that any new rating technique be employed with caution and that additional tests to those described herein be conducted before sizing a system using geotextile filters as one component.

ACKNOWLEDGEMENTS

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Soil Filtration Phenomena of Geotextiles

Phénomènes de filtration des sols par des géotextiles

The mechanism of soil-filtration in porous media is rather complicated. Granular and fabric filters seem to be different in their behaviour. Based on an objective comparison, which takes into account the real geometrical parameters as pore-openings of soils and opening-sizes of fabric filters, respectively their maximum values, it is demonstrated, that in relation to the elements of the different media (grains, fibres) the filter characteristics are similar. Except woven filter fabric, which shows a real sieve-filtering, all other filter media (non-wovens and granular filters) are influenced in their particle-retaining ability by the so-called filtration-length (filter-thickness).

Le mécanisme de filtration des sols en milieu poreux est très compliqué. Le milieu granulaire et les textiles semblent se comporter differemment. En se basant sur une comparaison objective, qui prend en compte les paramètres géometriques veritables, tels que l´ouverture des pores du sol et les ouvertures du textile, en particulier leurs valeurs maximales, on montre que les caractéristiques du filtre sont comparables à celles du milieu granulaire à l´échelle du grain et de la fibre. A l´exception des filtres tissés qui filtrent comme à travers un tamis, tout les autres filtres (non-tissés et granulaires) sont influencés, dans leur capacité à retenir des particules, par la longueur de filtration (épaisseur du filtre).

1 INTRODUCTION

The mechanism of soil-filtration in porous media (soils and fabrics) is rather complicated and thus, the approaches in engineering practice are mainly based on empricism. Only few and sometimes very simplified models have been established, to get a better understanding of the fundamental mechanism. Due to sometimes rather complicated hydraulic boundary conditions (stationary and instationary loading of filter interfaces), which in most cases cannot be quantified precisely, the safest approach is based on geometry of the filter media. Geometrical parameters are pore-openings for mineral filters (sand, gravel, etc.), mesh-sizes for woven geotextiles and opening-sizes for non-woven geotextiles. Filter criteria can then be established by comparing grain-sizes of the soil to be filtered (base-material) with the former described geometrical characteristics of the filter media.

Latest research work on mineral filters (Wittmann (1)) has pointed out the influence of filter-thickness, the so-called filtration-length, which was not taken into account so far in existing criteria. Based on pore-measurements of granular media (sand and gravel) these mineral filters are now directly comparable to fabric filters. This comparison of different filter media is exemplarily done in this paper, mainly dealing with the pore-size and opening-size characteristics and the filtration-length. A complete review of existing filter criteria is not given in this paper. For this it is referred to Heerten (2), Wittmann (3), etc.

2 CHARACTERIZATION OF POROUS MEDIA

Filter media (soils, geotextiles) are geometrically characterized in regard to their particle-retention ability (soil-filtration) by opening-sizes between the grains or fibres. Only for woven geotextiles with simple structures, optical methods can be used to determine the "mesh-openings". All other structures (non-woven geotextiles and soils) are rather complicated and thus direct measurements have to be replaced as for example by sieving-tests for non-wovens. These sieving-tests with glass-ballotini or natural soils identify the range of existing opening-sizes and especially the maximum opening-size, if sieving time, amount of fines, amplitude and frequency of sieving are being optimized, to give the extreme values. For granular filters these sieving tests were not applied so far. Filter tests under hydraulic loading (percolated samples) were those tools, being used to establish the well-known grain-size-ratios (filter criteria) for granular filters (Examples: Terzaghi, US Bureau of Reclamation, etc.).

So far, pore-sizes of granular media are only calculated on model assumptions of regular sphere-packings, as for example minimum pore-size $d_{p,min}$ in dense packing

$$d_{p,min} = 0.155 \cdot D_{min} \qquad (1)$$

or on Atterberg's assumption, that the average pore-diameter $d_{p,a}$ is equivalent to

$$d_{p,a} = 0.2 \cdot D_{10} \qquad (2)$$

Further information on the pore characteristic of soils was not available.

3 NEW PORE MEASUREMENTS OF GRANULAR MEDIA

Because of the complexity of the granular media, spatial pore measurements are not successful. This was the main reason, why new measurements were done at slices, being produced by cutting resin bonded samples of granular media. Photographs were taken from these slices (see figure 1) and pore-size-distributions were determined at these plan reproductions of the porous medium by a digital curve integrator. The results of these measurements showed a good reproducibility. Figure 2 represents a typical pore-size-distribution for a rather uniform soil in a dimensionless graph. The minimum pore-size of these measurements was in good accordance with the value of equation (1).

Fig. 1 Slice for pore-measurements of soils, taken from a resin bonded gravel

4 FILTRATION-LENGTH OF GRANULAR MEDIA

The measured pore-size-distribution is valid for each slice being taken from the soil sample, if the soil is homogeneous and isotropic. The spatial pore-size-distribution can be evaluated from the measured pore-size-distribution by "putting these slices in series, with distances of the grain-diameter respectively the average grain-diameter between the

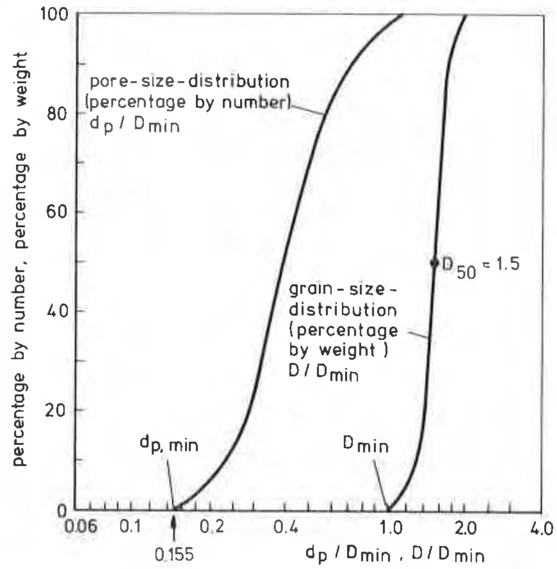

Fig. 2 Measured pore-sizes and grain-sizes divided by the minimum grain-size of the filter

slices". This model simulates the narrowing and widening of pores along the so-called pore-channel in the direction of water-flow. Big pores are thus isolated by meeting smaller pores. With increasing thickness of the filter - the so-called filtration-length, which can be expressed as filter-thickness L_F divided by the grain-diameter D of the filter (dimensionless parameter $m = L_F/D$) - the amount of bigger pores decreases, being visible in the shifting of the pore-size-distribution around the minimum pore-size $d_{p,min}$ (see figure 3). For a filter of infinite length the pore-size-distribution yields to a straight vertical line passing $d_{p,min}$.

Different steps between for finite filtration-lengths can be calculated by superimposing the pore-size-distribution for m = 1 probabilistically with itself. The result can be seen in figure 3 for m = 2 to m = 100. A detailed description of this method is given by the author in reference (1).

4 COMMENT ON EXISTING FILTER CRITERIA

This model teaches the influence of filter-thickness, which was not taken into account so far. It's validity was proved in the former mentioned sieving-tests. Granular filters of different length were used as "sieves" with fine particles being applied to identify the pore-openings. In these tests a confidence level of 99.99 % was found to be a safe approach, which means that for a certain filtration-length, the pore-diameter $d_{p,99.99\%}$ is a safe prediction of the maximum pore-opening $d_{p,max}$.

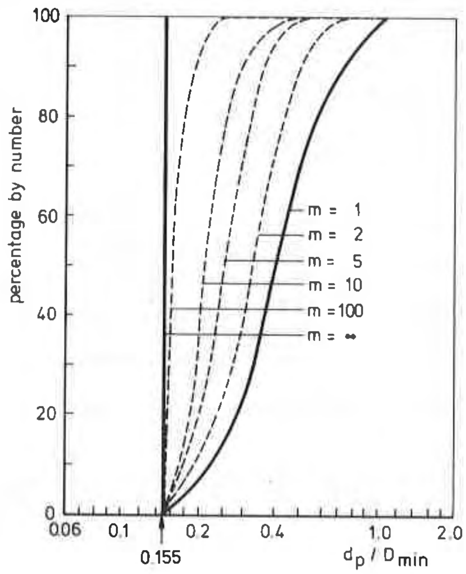

Fig. 3 Changing of pore-size-distribution with increasing filtration-length m

Using this model, it can be demonstrated that filter criteria based on grain-size-ratios as for example $A_{50} = D_{50}/d_{50}$ (D - grain-diameter of filter; d - grain-diameter of base-material) are geometrical safe, only if a certain length of the filter is given. The aspect of geometrical safety means, that a base material grain cannot pass the pores of the filter even under extreme hydraulic loading. For rather uniform base and filter material the grain-sizes d and D are well represented by d_{50} and D_{50} and with $d_{50} > d_{p,max} = d_{p,99.99\%}$ the following ratios $A_{50} = D_{50}/d_{50}$ are geometrically safe for the presented filtration-lengths $m = L_F/D_{50}$.

$A_{50} = D_{50}/d_{50} = 1.5$	$m = 2.6$
$A_{50} = D_{50}/d_{50} = 3.0$	$m = 7.6$
$A_{50} = D_{50}/d_{50} = 5.0$	$m = 35.2$
$A_{50} = D_{50}/d_{50} = 8.0$	$m = 179.6$

With the definition of filtration-length it is pointed out, that a grain-size-ratio of $A_{50} = 5$, which is the lower boundary of the filter criterion by the US Bureau of Reclamation, needs a filtration-length of about 35 grains, to be geometrical stable. A gravel filter with $D_{50} = 10$ mm thus needs a thickness of about 350 mm to retain base material with $d_{50} = 2$ mm. A sand filter with $D_{50} = 1$ mm shows the same particle retention for 0.2 mm base material at a filter thickness of 35 mm, which is almost below those thicknesses used in earth constructions. For these rather uniform soils a ratio $A_{50} = 10$, which is the upper limit of the filter criterion of the US Bureau of Reclamation, does not yield to a geometrically safe filter design. Differential analysation of this concept demonstrates that particle filtration at the interface base-material/filter (cake-filtration) is only given for very low grain-size-ratios ($A_{50} < 3$). For bigger values, filtration takes place at this interface and inside the filter.

This concept can also be applied to non-uniform soils, which is analysed in reference (4).

6 COMPARISON OF GRANULAR FILTERS AND FABRIC FILTERS (GEOTEXTILES)

Geometrical characterizations of textile filters in practice are based on sieving-tests, to gain information on the maximum opening-sizes and on the opening-size spectrum. Direct measurements of opening-size characteristics up to now were only done by Rollin et al. (5) at cross-sections of geotextiles. A similar technique of sample preparation was used as described in chapter 3. Pore-size histogramms were determined using an image-analyser. The according theory is only valid for thick geotextiles (h > 1.5 mm) and the measurements being done at cross-sections cannot be directly compared to the results from granular media in perpendicular planes.

Therefore as a first approach to this problem data being available from producer's references in literature and prospectus is analysed in regard to filtration characteristics.

The maximum pore-size $d_{p,max}$ of granular filters is comparable to opening-sizes of geotextiles. These openings are often taken near to the maximum opening, with according geometrical safety being considered in filter criteria, asking for a certain percentage of base material being bigger than the used opening value. Opening-values being used so far are O_{90} or O_{95} (French geotextile committee (6)) or O_{98} (Ogink (7)) or effective opening-sizes D_w (Heerten (2)).

As these measurements of opening-sizes are being done on nearly unloaded geotextiles, according thickness values at low loads (0.005 to 0.02 bar) have to be used in calculation. To establish a filtration-length m* for geotextiles, the thickness h of the textile is divided by the diameter d_f of the fiber or filament. This characterization is valid for mechanically bonded geotextiles without further treatments of the fiber structure (thermal or chemical bonding).

O_{95}-values of six products (needle-punched geotextiles of polyester and polypropylene) are related to the filtration-length $m^* = h/d_f$ and are plotted in figure 4. As being expected, the thickness and therefore the filtration-length m* are of influence on the opening-size. This influence of filtration-length, as being demonstrated in the graph, is

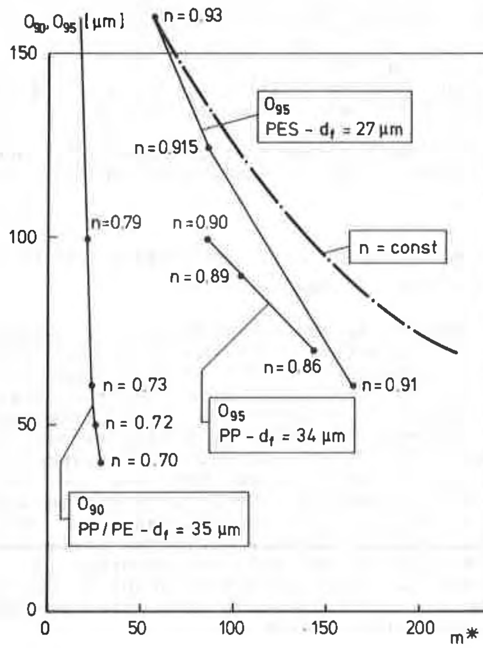

Fig. 4 Opening-sizes of geotextiles as function of filtration-length m*

not objective, because the compressibility of geotextiles is not considered up to here. As can be seen in figure 4, the according porosities are not constant but decreasing with increasing filtration-length. Thus the compressibility of geotextiles leads to a reduction of filtration-length - factors of 1.5 to 2 can be observed between light-weight and heavy geotextiles - and to a reduction in opening-size.

For constant porosity (n = const.) the estimated phenomenological curve is plotted in figure 4, because a precise quantification is not yet possible. It can be seen, that without compressibility the increasing filtration-length leads to a slighter decrease in opening-size und thus can be called uneconomical for large filter thicknesses. The influence of compressibility is again demonstrated for a thermally bonded geotextile in figure 4 by plotting O_{90} as a function of m*. But this influence is overstressed by the different manufacturing process. Further research work could thus reveal the influence of fiber diameters and different types of bonding on the opening-size characteristics.

The validity of the estimated curve for constant porosity in figure 4 is confirmed by experiments of Partenscky and Grabe (8). They showed, that several layers of non-wovens do not change the effective opening-size and the filtration characteristic significantly in regard to the value of a single layer, although retained fines are increasing to a certain amount. This result is presented in figure 5, being a good proof for the constant porosity assumption.

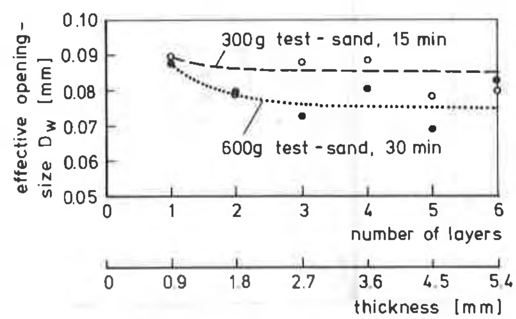

Fig. 5 Influence of filtration-length on effective opening-size D_w (reference (8))

The reduction of opening-sizes by compression of geotextiles is accompanied by a reduction in permeability, whereas increasing filtration-length without compression would not influence the permeability. The influence of filtration-length on particle-retention signifies, that particles migrate into the geotextile to a certain extent, which was also observed by microscopic observation (Rollin et al. (5)). Further calculations of these phenomena, being also described in reference (8) can contribute to clogging effects and permeability changes in relation to the virgin fabrics.

The comparisons of granular filters with maximum pore-sizes $d_{p,99.99\%}$ to fabric filters with opening-sizes O_{95} lead to the result, that both structures show similar filtration characteristics, if the element is taken into account. This is finally demonstrated in figure 6, which shows maximum pore-sizes of mineral filters in the same range as those of geotextiles in figure 4. The filtration characteristics in regard to m respectively m* are similar, especially if that filter with D_{50} = 0.75 mm (fig. 6) is compared with the estimated line for geotextiles at n = const. (fig. 4). This comparison is also valid for the hydraulic parameters. Mineral filters of figure 6 show permeabilities of $4 \cdot 10^{-1}$ to $5 \cdot 10^{-2}$ cm/s, which are equal to those of mechanical bonded nonwovens ($k \sim 10^{-1}$ cm/s) and similar to those bonded thermically or chemically ($k \sim 10^{-2}$ cm/s).

The dimensionless parameters m and m* have to be considered if real filter thickness is calculated, because in these examples demonstrated fiber diameters are about 8 to 25 times smaller than the according equivalent grain-

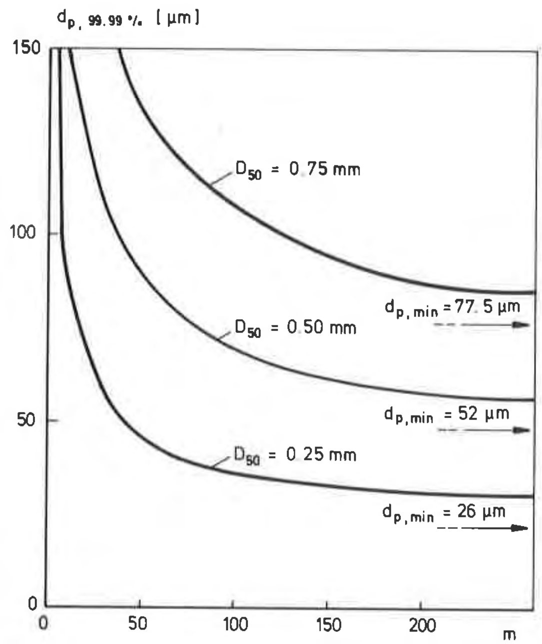

Fig. 6 Maximum pore-sizes of granular filters as function of filtration-length m

sizes and thus for the same dimensionless filtration-length, the granular filter has to be 8 to 25 times thicker than the equivalent filter fabric.

7 SUMMARY

As a first approach an objective comparison of different filter media has been tried in this paper. Based on new pore-measurements at granular filters and on the evaluation of the influence of filtration-length, the similarity of granular and fibrous filters in regard to their elements (grains and fibres) was demonstrated. Further research work has to be done, to complete the knowledge for example on the influence of compressibility, the influence of different manufacturing processes (type of fibres, type of bonding, influence of needle-punching, surface treatments, calendering etc.) and the influence of deformed geotextiles. All these influences have to be considered in filtration works. Geotextiles, being industrial products, are strongly influenced by these manufacturing processes, which in regard to a safe and economical filter design should be revealed in further studies.

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Some Aspects Concerning Retaining Capacity of Geotextiles

Quelques aspects concernant la capacité de rétention des géotextiles

The paper deals with several aspects concerning the use of fabrics as filtering elements. A laboratory procedure by which the fabric capacity of retaining solid discharge may be determined is presented. The obtained results for the examined phenomenon allow the establishment of characteristic curves and indices. It is also emphasized the fact that defining this phenomenon on the basis of dimensional elements only is unsatisfactory,many of the geotextiles manifesting a susceptibility to retain fines due to the action of electrochemical forces. The intensity of these forces is strongly correlated to the chemical nature of the polymer of which the fabric is made and the mineralogical composition of the soil. For evaluating the geotextiles filters operation, criteria have been formulated establishing the domain or utilisation conditions, taking into account all the revealed aspects concerning the geotextiles capacity to retain the solid discharge. The stated criteria validity had been confirmed by tests on physical models.

L'article traite quelques aspects concernant l'utilisation des géotextiles comme éléments filtrants. On présente une méthodologie de laboratoire à l'aide de laquelle on peut déterminer la capacité des géotextiles de retenir le débit solide. Les données obtenues permettent d'établir des courbes et des indices caractéristiques au phénomène analysé. On remarque aussi qu'il ne suffit pas à définir ce phénomène exclusivement à base des éléments dimensionnels, de nombreuses géotextiles manifestant une susceptibilité à retenir les fines particules sous l'action des forces de nature électrochimique. L'intensité de ces forces est dans une corrélation étroite avec la nature chimique du polymère des géotextiles et la nature minéralogique du sol. Pour l'évaluation de la fonctionnalité des filtres géotextiles on a formulé des critères qui établissent le domaine ou les conditions d'utilisation, en tenant compte de tous les aspects relevés concernant la capacité des géotextiles de retenir le débit solide. La validité des critères énoncés a été confirmée par des vérifications sur des modèles physiques.

The use of geotextiles as filtrant or filtrant-drainant elements requires the knowledge of the characteristics to be considered in estimating their hydraulic and protective efficiency expressed by the fabrics permeability and their capacity to retain the solid discharge. It is of extreme importance to formulate certain judgements concerning the conditions of optimal operation and the evolution in time of geotextiles capacity, as filtrant or filtrant-drainant elements.

DETERMINATION OF GEOTEXTILES CAPACITY TO RETAIN THE SOLID DISCHARGE

In the Hydraulic Engineering Research Institute from Bucharest a special programme to determine geotextiles capacity of retaining the solid discharge was initiated. The test consists in the filtering of a soil-water suspension through a geotextile under a constant high of 10 cm of suspension column (fig.1). For the performance of tests a device of the dead-level permeameter type was used. The grained material from the permeameter as well as from the water supply tank is permanently maintained in suspension by rotating agitators.

Preliminary tests led to the following optimal parameters of testing:

- suspension concentration: 5%;
- grained material having a continuous grain size distribution from 0.002 mm to 2.0 mm
- filtering duration: 15 minutes.

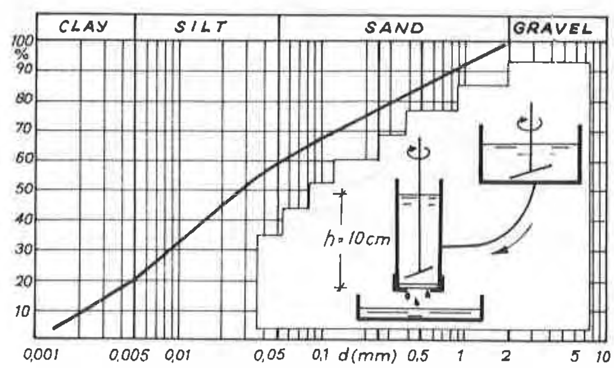

Fig.1. Testing conditions for the geotextiles capacity to retain the solid discharge

The values obtained by this test are: the grain size distribution and the solid mass quantity passed through the geotextile (P) remained on it (R) and remained in the geotextile (R_{in}). For the quantity remained on the geotextile and the one passed through it the methodology needs the complete collecting of the corresponding part of material, sedimentation and filtering of solid part, drying it to constant weight. The weights of each of these parts are then determined by weighing and the grain size distribution by usual sieving and hydrometer method. For the mass remained in the geotextile the use of indirect methods is accepted: the mass quantity is obtained by the difference between the weight of fabric sample dried at constant weight, before and after the test, and the grain-size distribution by the difference between the initial grain-size distribution of the material and the cumulated size distribution of the grains retained on and passed through. In this manner, the characteristic curves of retaining capacity for different fabrics may be represented in a semilogarithmic plot (fig.2).

OBTAINED RESULTS AND CHARACTERISTIC ELEMENTS

The tests performed on various types of woven and non-woven geotextiles: MADRIL, TERASIN, BIDIM, TERRAM, ALFA, made evident the fact that as far as fabric retaining capacity is concerned there are two different kinds of behaviour well characterized by their retaining capacity curves (fig.2).

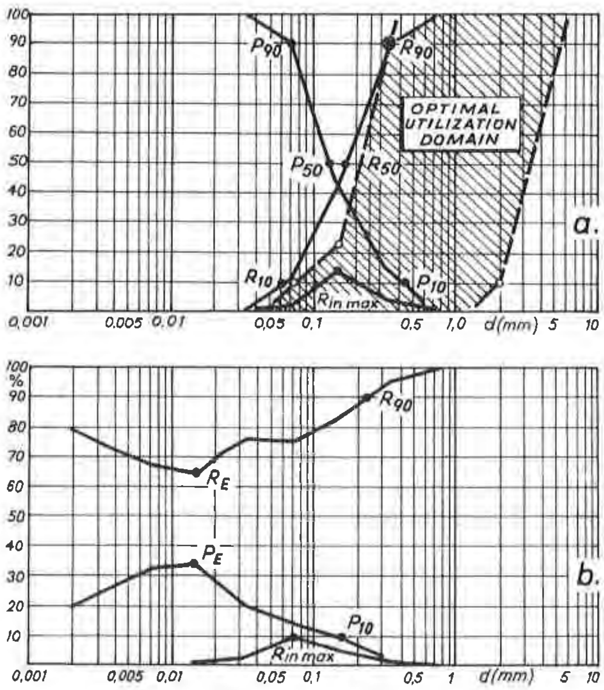

Fig.2. Retaining capacity characteristic curves.

Thus, for a certain type of fabric, the quantity of mass passed through increases as the particle size diminishes (fig.2.a). It is the case of geotextiles with large pores (non-woven) or openings (woven) which manifest a retaining capacity of mechanical nature towards the solid particles. For another group of fabrics the retaining capacity acts on the silt and clay particles, intensifying as the particles dimensions decrease (fig.2,b). As in 99% of the cases the pores dimensions are larger than that of the fractions for which the mechanical retention becomes possible, this phenomenon could be attributed to the electro-chemical forces action. This is the case for fabrics with small pores or openings that dimensionally create the conditions for the electrochemical forces action in the filtering of a soil suspension through the fabric. The tests performed up to now by the authors situate almost all examined non-woven geotextiles mechanically consolidated by needle punching in the last group.

The retaining capacity graphs may be used to define some specific values which by their nature could be considered retaining capacity dimensional indices for geotextiles.

Expressing the illustrative limits of the analysed phenomenon the following retaining capacity dimensional indices are proposed:

- the extreme dimensions of the soil particles that 90%, 50%, 10% remain on the geotextile (R_{90}, R_{50}, R_{10}) or pass through it (P_{90}, P_{50}, P_{10});
- the dimension of solid particles that in the greatest proportion are retained in the geotextile ($R_{in\ max}$);
- the dimension of solid particles from which the retention by electro-chemical forces probably begin to act (R_E).

Objective causes connected to errors that could be introduced by the manner and technique of testing, imposed as maximum and minimum dimensions of the soil particles passing through or remaining on the geotextiles those corresponding to 90% and 10% respectively.

The retaining capacity dimensional indices can equally characterize the woven and non-woven fabrics. Obviously the solid particles retention for woven fabrics being incompatible with their structure, the $R_{in\ max}$ index is missing and the remained on and passed ones are equal:

$$R_{90} = P_{10} \text{ and } R_{10} = P_{90}$$

The authors consider that the retaining capacity dimensional indices also provide informations concerning:

- the maximum dimensions of the non-woven geotextile pores and of the woven geotextile openings (R_{90});
- the evaluation of the domain of soil particles dimensions over which the geotextile performs its retaining capacity ($R_{90} - R_{10}$);
- the dimension of the solid particles susceptible to be retained in the fabric ($R_{in\ max}$);
- the geotextile susceptibility for clogging by retaining the solid particles on and in it due to electro-chemical forces (R_E);
- the degree of non-uniformity for non-woven fabrics pores or for woven fabrics openings considering the following proposed intervals:

$$\text{uniform} \quad 5 > R_{90}/R_{50} > 5 \quad \text{nonuniform}$$
$$15 > R_{90}/R_{10} > 15$$

REMARKS CONCERNING THE GEOTEXTILES SUSCEPTIBILITY TO RETAIN THE SOLID DISCHARGE

An orientation of studies concerning the manner in which fabrics perform their function of retaining the solid discharge is based on defining this function in relation with the particles dimensions and the fabric porosity characteristics (3, 6, 7, 9).

The results obtained from the testing on the geotextiles capacity to retain the solid discharge emphasized the fact that this phenomenon is far too complex to be exclusively defined by some dimensional elements: soil grain size and fabric porosity and pore dimensions (1).

For several reasons the characterization of fabric retaining capacity for solid particles by dimensional elements only is questionable:

- the dimensional values illustrating the soil grain size and the fabric porosity are obtained by indirect tests (the grain size hydrometer analysis and the method based on suction respectively) fact that give them a high degree of conventionality;

- the differing principles on which these values are determined make their comparison doubtful;

- the restrictive effect of a porous medium with wide specific surface is obviously more complex than that attributed to a mechanical process.

The above mentioned reasons suggested to the authors the examination of the geotextile and soil nature influence (chemical or mineralogical composition) over the intensity of the fine particles retention phenomenon.

The attention has been oriented towards the unexpected process of retaining particles finer than the fabric pores dimensions.

The experimental programme operated with four types of fabric produced in Romania: MADRIL M, MADRIL V, MADRIL P and TERASIN and four types of soil: caolin, loess, quartz sand and micaceous sand.

The characteristics of the experimentated geotextiles made of various polymers and having different structures are mentioned in table 1.

Table 1. Definition charcteristics of geotextiles used for the experimental programme.

geotextile	nature of the polymer	fibre charac-teristics (tex/mm)	techno-logy	mass μ (g/m^2)	thick-ness T (cm)	poro-sity n (%)	percent distribution of pores: 0 < 0.03 mm	percent distribution of pores: 0 > 0.03 mm
MADRIL M	pp	0.66/60	needle-punched	520	0.57	89.9	0.3	99.7
MADRIL V	pp	1.99/100	needle-punched	560	0.59	89.6	0.2	99.8
MADRIL P	pes	0.44/60	needle-punched	470	0.42	91.6	0.3	99.7
TERASIN	mixt. pes,pna pp (waste)	—	needle-punched and chemically bonded	660	0.75	89.0	0.5	99.5

The soils, loess excepted, are unminerals their grain-size distribution being specified in figure 3. For the tests they have been used individually and in mixtures of two. The tests have been performed in conformity with the above expressed methodology for the determination of fabric capacity to retain the solid discharge.

In order to emphasize the intensity of the observed phenomenon the retaining capacity has been analysed for the material having particle dimensions smaller than 99% of geotextile pore (d < 0.03 mm).

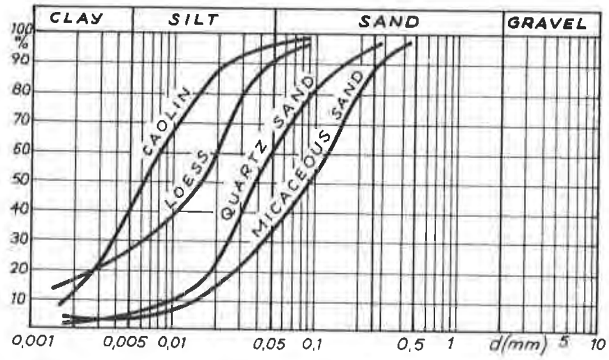

Fig.3. Grain-size distribution of the soil used in tests for the determination of the geotextiles susceptibility to retain fines.

The fabric susceptibility to retain particles smaller than the pore dimensions - Sg (0.03) - is expressed by the ratio between the quantity of particles smaller than 0.03 mm remained on the fabric - R (0.03) - and that having d < 0.03 mm from the initial soil - G (0.03)

$$Sg\ (0.03) = \frac{R\ (0.03)}{G\ (0.03)}\ 100 \quad (\%)$$

The results obtained for the four types of soil and the four tested fabrics are mentioned in table 2.

Table 2. Geotextiles susceptibility to retain particles smaller than the pores dimensions.

geotextile \ soil	quartz-sand	micaceous-sand	caolin	loess
MADRIL M	91	53	72	94
MADRIL V	78	80	92	77
MADRIL D	87	92	88	92
TERASIN	74	79	87	84

The presented results allow the following findings:

- in all the cases the fabric susceptibility to retain particles smaller than the pore dimensions is obvious;

- the differences are present both in the behaviour of each of the soil types towards the fabric and in the behaviour of each fabric towards the corresponding soils, the difference between the maximum and minimum percentage of the retained material being for the same fabric between 5% and 41% and for the same soil between 17% and 39%.

- generally speaking the analysed susceptibility is greater for caolin and loess than for quartz and micaceous sands, the former ones being retained on an average of 85% and 87% respectively and the latter ones on an average of 76 % and 83% respectively.

These facts argue for the supposition that during the filtering process the fabric restrictive effect on the solid discharge cannot be expressed only by a retention of mechanical nature, but also by the influence of some electrochemical forces action.

The intensity of this action is connected to the nature of the two media: soil and fabric.

The specific behaviour of each unimineral soil-fabric ensemble imply a more complex manifestation than in the case of mineral mixtures of two or more than two components.

The tests accomplished to point out this aspect have been performed on mixtures of two components from the four soils: quartz sand, siliceous sand, caolin, loess, the combinations in all cases being made for a weight ratio of 4/1; 1/1; 1/4.

The influence of the mixture on the fabric susceptibility to retain fines was expressed by the ratio between the susceptibility estimated by calculus and the one ascertained by tests.

Considering that the difference of ± 0.1 is included in the domain of possible errors that can be introduced by the determination technique it was accepted that the ratio from 0.9 to 1.1 emphasize that the behaviour is corresponding to the values computed in conformity with the dosage.

The ratio smaller than 0.9 indicate an accentuation of the analysed susceptibility and the ones higher than 1.1 its diminution.

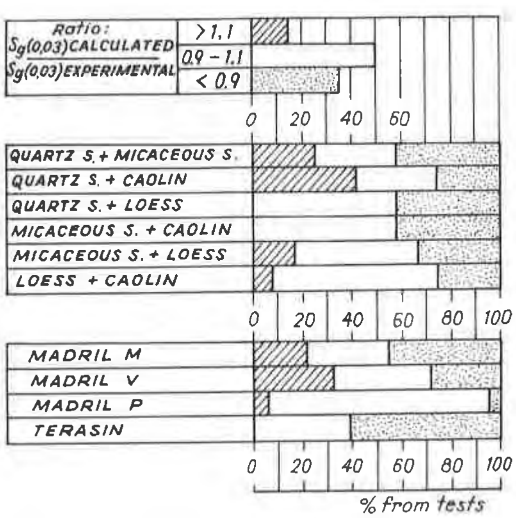

Fig.4. Soil mixtures influence on the geotextile susceptibility to retain solid discharge.

The obtained results (fig.4) leaded to the following:

- a general estimation resulting from the analysis of all the tests performed shows that the mineral mixture is able to influence the fabric susceptibility to retain the solid discharge; only 50% of the tests corresponds to the calculated values, the rest showing changes on both sides;

- the obtained results for each mineral mixture show in all cases an influence on the fabric retention of particles but with no important differentiations due to the dosage;

- irrespective of the mixture dosage and components each fabric intoduces a specific behaviour; the ratio and the way in which the modifications act on the retaining capacity are seriously different depending on the polymer nature. An almost similar behaviour is to be noticed for the fabrics made of the same polymer (MADRIL M and MADRIL V).

The presented results concerning the fabric susceptibility to retain particles smaller than the pores dimensions underline the fact that during the filtering process the fabric capacity to retain the solid discharge is influenced by the soil mineralogical composition and the polymer chemical nature. Therefore the estimation of fabric behaviour as filtrant or filtrant-drainant element exclusively based on dimensional values of the two media is unsufficient.

CRITERIA FOR ESTABLISHING THE FABRICS OPTIMAL OPERATION CONDITIONS

The efficient operation of fabric filters is conditioned by a correct selection of the geotextiles in relation with the soil characteristics and the knowledge of the soil-fabric ensemble behaviour during the filtering process.

By analogy with the criteria for the grained filters several empirical rules (2, 4, 7) were proposed which define the hydraulic efficiency and hydrodynamic stability conditions depending on certain characteristic values of the two media: soil and fabric.

On the basis of the results obtained by tests concerning the fabric capacity to retain solid discharge, the authors establish the fabric optimal operation conditions using criteria that correlate the soil characteristic value (d) to the retaining capacity dimensional indices (5).

These criteria define the hydraulic efficiency and hydrodynamic stability condition, consenting that during the filtering process the fabric must allow the entire water amount to pass selectively restricting the solid particles transport as it is consented for the granular filters too.

For fabrics manifesting only a restrictive effect of mechanical nature the formulated criteria are as follows:

- for the stability condition:

  • the protected soil must contain maximum 10% fractions that pass through the fabric in a 90% ratio ($d_{10} \geqslant T_{90}$);

  • protected soil must contain mininimum 10% particles larger than the pores or openings maximum dimension ($d_{90} \geqslant R_{90}$);

· protected soil must contain maximum 25% fractions corresponding to the maximum ones retained in the fabric ($d_{25} \geqslant R_{T\ max}$).

- for the permeability condition:

$$k_{fabric} \geqslant 2\ k_{soil}$$

The stated criteria define a "size distribution domain of optimal operation" specific for each fabric (fig.2.b).

For fabrics susceptible to retain fine particles by means of electrochemical forces the following criteria are formulated:

- concerning the condition of stability:
  · fractions smaller than 0.05 mm pass through the fabric with a ratio of maximum 25%;
- for the condition of permeability:

$$k_{fabric} > 5\ k_{soil}$$

· after checking up the fabric capacity to retain the solid discharge its permeability observes the condition:

$$k_{fabric\ (after\ testing)} \geqslant 5\ k_{soil}$$

For these fabrics the mentioned criteria do not actually restrict a granulometric domain of optimal operation their use being possible for all soils (sandy, silty and clayey soils).

The checking up of clogging susceptibility by retaining the fines is imposed.

In order to verify the valability of the proposed criteria, laboratory tests on physical models were performed observing the protection efficiency achieved by the fabric filter and its long term behaviour.

The models have been made of soil corresponding to the geotextile optimal utilization domain as well as of non corresponding one .

For emphasizing the aspects connected to the protection efficiency of fabric filters,samples have been subjected for short periods to flows up to i = 10 gradients.

The results obtained had been edifying. All the tests performed with optimal soil registered no grain drive through the fabric and no clogging phenomenon. For tests with nonconform soil the grain drive was present in all cases, sometimes having a progressive manifestation up to the complete passing of the soil sample.

The long term fabric filter behaviour was observed on models, during more than 700 days in which the flow had been maintained at constant gradient (i = 1).

The tests have been performed for filters of MADRIL M, MADRIL V, MADRIL P and TERASIN. For comparison a classical grained filter had been tested under the same conditions.

All models proved a good behaviour for fabric filters similar to that showed in figure 5 for MADRIL V 500.

All tests proved the existence of three stages marked by: the diminution of ensemble permeability due to the particle rearrangement on the moment of flow release; the increase of permeability during the period corresponding to the reverse natural filter formation on the soil-fabric contact area; the filtering process stabilization. In this last stage attained after about 300 days, the ensemble permeability lays between the extreme priorly attained values, being a little larger than the one achieved for the grained filter.

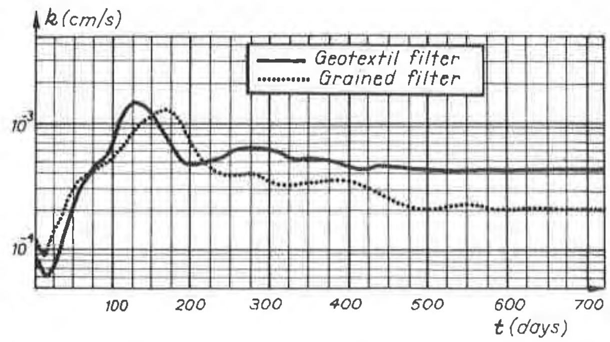

Fig.5. The long term behaviour of the geotextile filter MADRIL V 500.

Taking into consideration the obtained results it can be ascertained that in all cases the fabrics had a proper behaviour. Adding that the tests performed for emphasizing the protection efficiency leaded to the same quantitative results it is to be concluded that the fabric filtering operation evaluation on the basis of the above mentioned criteria can offer satisfactory results.

CONCLUSIONS

1. The geotextiles filtrant-drainant effect and their capacity to retain solid particles have been the object of a large experimental programme performed in the Hydraulic Engineering Research Institute.

2. The researches emphasized the necessity for grouping the geotextiles in two categories:

- geotextiles manifesting a diminution of the retaining capacity as the solid particles dimensions decrease. The activity of retaining the solid particles is predominantly of a mechanical nature;
- geotextiles manifesting an intensification of the retaining capacity as the solid particles dimensions decrease. In this case it is likely that the action of some electrochemical forces is added to the mechanical one.

3. The manifestation of a retaining capacity that cannot be expressed by the correspondence between the solid particles and the geotextiles pores dimensions had been particularly emphasized by experiments on soil with different mineral compositions and on geotextiles made of different polymers. The obtained results underline the influence exerted by the mineralogical nature of the soil and especially by the nature of the polymer the geotextile is made of over its capacity to retain particles having dimensions inferior to those of its pores.

4. The facts presented above (2;3) lead to a very interesting conclusion from the applicative point of view: the geotextiles capacity to retain the solid particles has to be experimen-

tally examined for each ensemble soil-geotextile as its evaluation on dimensional criteria (solid particles dimension and geotextiles pores dimension and porosity) only is not edifying.

5. The performed studies allowed the definition of certain characteristic indices regarding the geotextiles retaining capacity (characteristic diameters for the retained on and in the geotextile as well as for the one able of filtering through it). Taking into consideration these characteristic indices criteria are suggested for defining the conditions of optimal utilization of geotextiles working as filtrant elements particularly as substitutes of granular filters.

6. The validity of above mentioned criteria had been proved on laboratory models, long period models (more than 700 days) included, with positive results.

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Laboratory Studies on Long-Term Drainage Capability of Geotextiles

Etudes de laboratoires sur la capacité d'écoulement de long terme de géotextiles

While it is generally recognized that the initial hydraulic permittivity of geotextiles is far greater than most soils they are protecting, their long-term performance has not been firmly established. Such concepts as filter cake formation, arching, blinding and clogging are often discussed as being explanations of limiting behavior. This study focused on long-term hydraulic tests in which both the soil and the geotextile were systematically varied. Typical behavior was bi-linear where soil compaction dominated the initial flow, followed by a strong dependence on the soil/fabric interaction. Tests showed that the drainage characteristics of the soil was significantly more important than the type or manufacture of the fabric and for a particularly poor draining fine grained soil, all fabrics tested gave essentially equivalent performance. Long-term flow tests, rather than gradient ratios, are suggested as being the preferred way to evaluate a particular soil/fabric performance.

Bien qu'il soit généralement reconnu que la permutabilité hydraulique initiale des géotextiles soit bien plus grande que la plupart des sols qu'ils protègent, leur performance à long terme n'a pas été fermement établie. De telles conceptions comme la formation de résidu pâteux, courbement, obstruction et blocage sont souvent traitées comme étant des explications de limitations de comportement. Cette étude était dirigée sur des tests hydrauliques à long terme dans lesquels le sol et le géotextile étaient systématiquement variés. Là où la compacité du sol dominait l'écoulement initial, le comportement typique était bi-linéaire, suivi par une forte dépendance sur l'interaction du sol avec le tissu. Les tests ont démontré que les caractéristiques du drainage du sol étaient nettement plus importantes que le type ou la fabrication du tissu et, pour un sol de grain fin particulièrement mal drainé, tous les tissus testés donnèrent essentiellement une performance équivalente. Les tests d'écoulement à long terme sont recommandés comme étant le meilleur moyen pour évaluer une performance particulière d'un sol et d'un tissue.

INTRODUCTION

Almost every geotextile application involving separation, reinforcement, drainage and fabric forming deals with water and its proper dissipation (1,2). This feature underscores the necessity of determining a given fabrics' hydraulic properties; more specifically, its flow rate, permeability or permittivity (the permeability divided by thickness). Toward this end many organizations have recommended test methods and specifications for the laboratory determination of these fabric properties. Note should be made, however, that these procedures are generally for the fabric alone, e.g., ASTM's "Standard Method for Testing the Water Permeability of Geotextiles - Permittivity Method" as proposed by Subcommittee D13.61 on Geotextiles. While of interest in comparing one fabric to another, these tests give no indication of the hydraulic behavior of the combined soil/fabric system.

As soon as soil is placed adjacent to the fabric, it is seen that the soils' hydraulic properties dominate the initial behavior (3,4,5). Only after a period of time does the fabric begin to play a role and, in the long-term situation, not at all, e.g., when a properly designed configuration exists. In this latter instance the flow passing through the soil/fabric system becomes constant and an equilibrium situation exists thereafter.

To verify and quantify these concepts one must perform long-term hydraulic tests on various soil/fabric systems. This is the goal of this paper, where the following items are specific objectives:

- To observe the nature and rate of soil adjustment in the initial flow stages.
- To determine the time required for a given soil/fabric system to reach a stable interactive stage.
- To determine if an equilibrium flow situation exists for a wide variety of soils and fabrics in specific soil/fabric configurations.

With the above objectives at hand, it then becomes possible to hypothesize as to the possible soil/fabric mechanisms that are occurring within the system. The ultimate objective of the proper hydraulic design of soil/fabric systems can then be addressed.

EXPERIMENTAL TEST SETUP

The experimental test setup for the flow tests used in this study was quite basic. Water at a constant head, flowed downward through the soil, then through the fabric and out of the system where it was collected and a flow rate was calculated. Figure 1 shows the apparatus where four tests can be simultaneously performed with sequential variation of either soil or fabric. The 9.5 cm (3.7") diameter plastic tubes were flanged near the base to hold the fabric. A known amount of soil was placed directly on the fabric (usually 720 gm) and a constant water head of 38.0 cm (15.0") was maintained above the fabric for the duration of the test. Due to large flow rates for some of the soils tested, deaired water was not used. Temperature corrections and losses due to evaporation were included. Physical and hydraulic properties of the soil used in this study are given in Table 1. Here it is seen that soil permeabilities ranged from 0.12 cm/sec to 6 x 10^{-7} cm/sec thereby covering a wide spectrum of situations.

Presented in Table 2 are the physical and hydraulic properties of the fabrics which were used. The fabric thickness and permeability values listed were obtained using the procedures of reference 5.

Table 1 - Soil Tested in This Study

Type	Class.[1]	d_{10}[2]	CU[3]	k[4]	γ_s[5]
Sand	SP	.30	3	1.2 x 10^{-1}	2.11
Mica Silt	ML	.02	10	9 x 10^{-4}	2.07
River Silt	ML	.01	17	3 x 10^{-4}	2.10
Silty Clay	CL	<.001	-	6 x 10^{-7}	2.09

Notes:

1 - Unified Soil Classification System
2 - 10% finer than size in mm
3 - coefficient of uniformity (= d_{60}/d_{10})
4 - coefficient of permeability, cm/sec
5 - saturated unit weight, gm/cc

Table 2 - Fabrics Tested in This Study

Type	Material	Construction	Wt.[1]	t[2]	k[3]	ω[4]
nonwoven	polyprop.	needled	400	3.6	0.21	.58
nonwoven	polyprop.	spunbonded	135	.38	0.02	.53
woven	polyprop.	slit film	135	.63	0.04	.63
woven	polyprop.	plain	245	.52	0.035	.67
knit	fiberglass	WIWK[5]	350	.63	0.014	.22

Notes:

1 - weight in gm/sq. m
2 - thickness in mm
3 - permeability in cm/sec
4 - permittivity in sec^{-1}
5 - WIWK - weft insertion warp knit

TEST RESULTS

A series of tests were performed using a single fabric (the nonwoven, needled polypropylene in Table 2) with the four soils listed in Table 1. The results are shown in Figure 2 for times approaching 1000 hours. This test series was followed by another one where the silty clay soil (the most troublesome as far as long-term flow is concerned) was used with four of the fabrics listed in Table 2. Results are shown in Figure 3 for times up to 1700 hours. Additional tests using the mica silt soil with both commercial and non-commercial fabrics listed in Table 2 resulted in the curves of Figure 4.

Observing the trends in these results, the different aspects of the flow mentioned earlier can be noted. By constructing tangents to the initial portion and to the final portion of each curve (and calculating their slopes), and by intersecting these tangents for an approximate transition time, the data of Table 3 was obtained. Note should be made that data scatter did indeed occur and that some liberty was taken in the interpretation, however, the basic trends were always quite obvious.

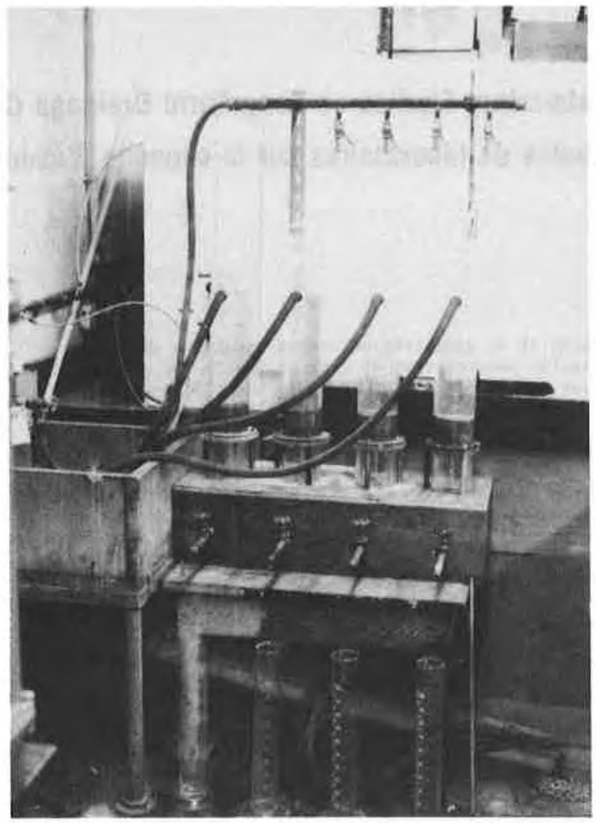

Fig. 1. - Experimental Test Setup for Long-Term Soil/Fabric Constant Head Flow Tests

Table 3 gives the averaged slopes (in units of cc/min/hr or q/t) of the flow curves of Figures 2, 3, and 4. Here it is seen that the decrease in flow rate is very great in the initial part of the test. This is completely the result of soil compaction during downward flow of the water through the initially placed loose soil. Of far greater importance is the long-term slope of these curves which when equal to zero suggests that equilibrium is established between the soil/fabric system and the particular hydraulic situation being imposed. If the slope of the curve is different from zero, interaction of the soil and the fabric is ongoing. The exact mechanism is difficult to establish but several conceptual ideas will be suggested in the next section. It is also important to note in Table 3 the time for transition between the slopes of the curves. This tells how long one must test before some type of stable soil/fabric system is established. As noted, the finer soils require relatively long periods of time, in excess of 100 hours, for testing.

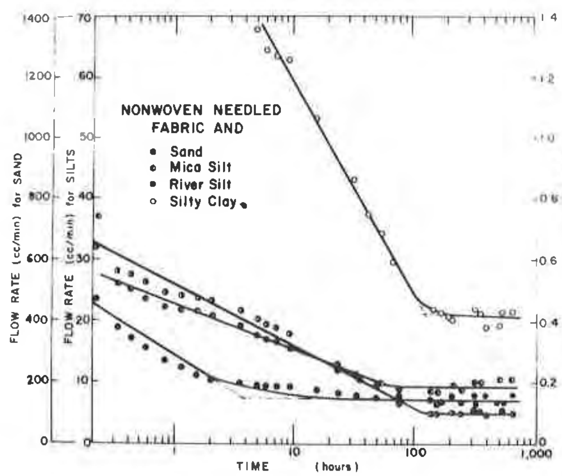

Fig. 2. - Long-Term Flow Curves for Nonwoven Needled Fabric and Four Soil Types.

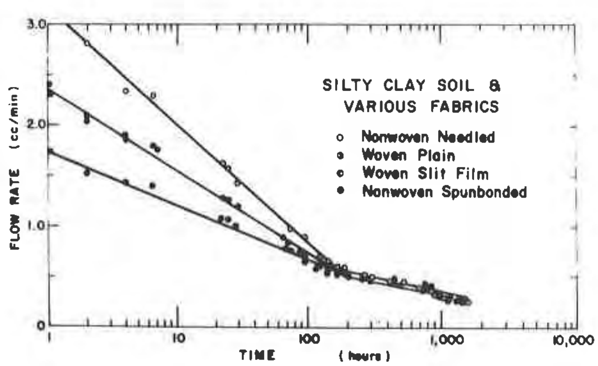

Fig. 3. - Long-Term Flow Curves for Silty Clay Soil and Four Different Fabrics

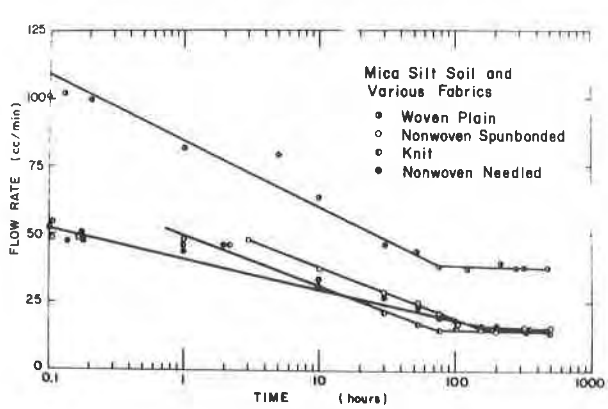

Fig. 4. - Long-Term Flow Curves for Mica Silt Soil and Four Different Fabrics.

Table 3 - Averaged Data from Long-Term Hydraulic Tests Shown in Figures 2, 3 and 4.

Soil	Fabric	m_i [1]	time [2]	m_f [3]
sand	nonwoven needled	270	4	0
mica silt	nonwoven needled	8	70	0
river silt	nonwoven needled	10	120	0
silty clay	nonwoven needled	0.7	130	.1
silty clay	nonwoven needled	1.1	160	.2
silty clay	woven plain	0.8	180	.2
silty clay	woven slit film	0.8	180	.2
silty clay	nonwoven spunbonded	0.5	180	.2
mica silt	woven plain	24	75	≈0
mica silt	nonwoven spunbonded	20	150	≈0
mica silt	knit	13	75	≈0
mica silt	nonwoven needled	11	150	≈0

Notes:

1 - slope of initial portion of flow curve in units of cc/min/hr or q/t
2 - transition time in hours
3 - slope of long-term portion of flow curve in units of cc/min/hr or q/t

Bearing this long test time in mind, additional flow tests were performed using the gradient ratio concept originally proposed by the Corps of Engineers (6). See Figure 5 for the test configuration. Figure 6 shows the results of the silty clay soil in conjunction with the nonwoven needled polypropylene fabric for times up to 2500 hours. As with previous tests, piecewise linear behavior is noted on the flow curves with reasonable agreement to values listed in Table 3. Of interest here, however, is the lower part of Figure 6 which plots gradient ratio (the ratio of the hydraulic gradient through the fabric and 2.5 cm of soil above it, to the adjacent 5.0 cm of soil) versus time. During the initial soil compaction stage, the gradient ratio is seen to be quite constant. However, beyond the transition time it increases rapidly, indicating a non-equilibrium situation. The limiting behavior for the case shown is not yet established and the test is still ongoing. It should be noted, however, that a unique gradient ratio value does not seem to be present. Other gradient ratio tests (not shown) using the river silt and the plain woven polypropylene fabric show uniform behavior up to the transition time and then major oscillations of the gradient ratio (values fluctuated between 2.5 and 7.0) throughout the duration of the test.

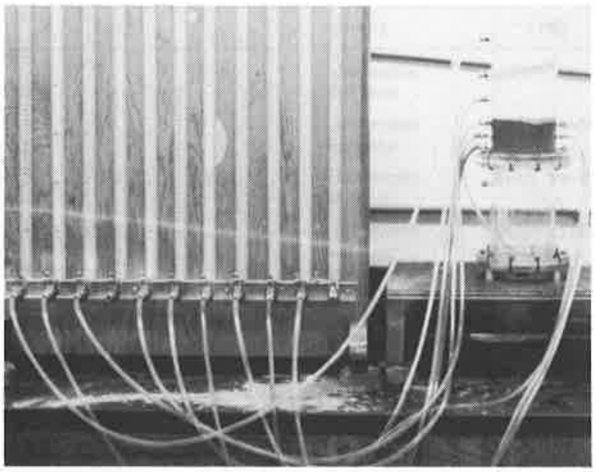

Fig. 5(a). - Photograph of Gradient Ratio Test Setup Where Hydraulic Heads are Monitored at Various Points Along the Soil Sample During a Constant Head Flow Test.

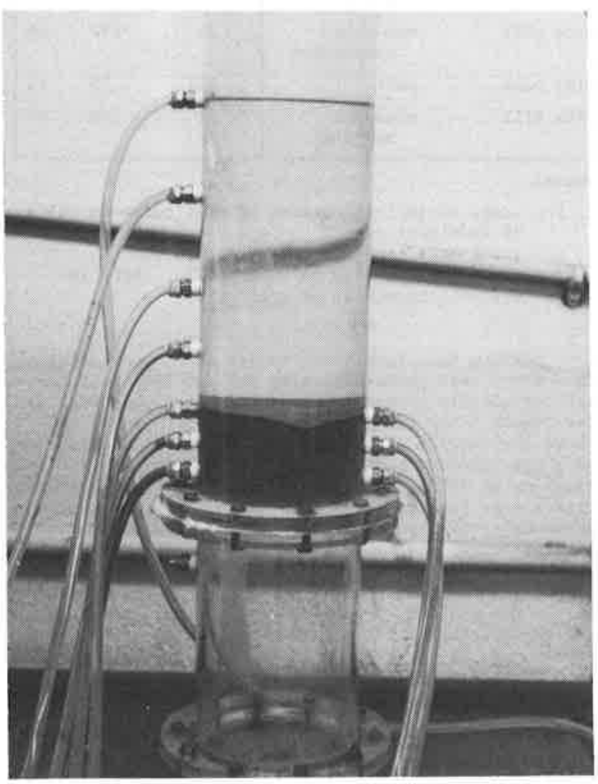

Fig. 5(b). - Closeup Photograph of Soil Sample During Constant Head Flow Test and Points Where Hydraulic Heads are Measured in Order to Calculate Gradient Ratio.

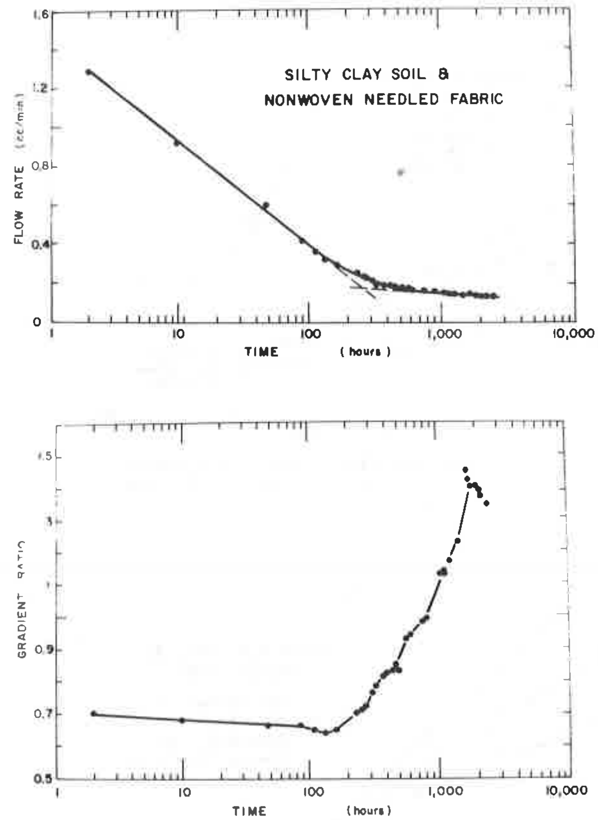

Fig. 6. - Long-Term Flow Curve of Silty Clay Soil and Nonwoven Needled Fabric (Upper) and Corresponding Values of Gradient Ratio (Lower).

SUMMARY AND CONCLUSIONS

Observation of the behavior of long-term flow tests through soil/fabric systems shows that the initial range is governed by the soil, and the final range is governed by soil/fabric interaction. It is this final range, as indicated by the slope of the flow curve, that is of primary interest. A zero, or nominal, slope is preferred over a large slope since it suggests equilibrium of the soil/fabric structure. This is perhaps explained by a stable soil filter structure at a finite distance upstream from the fabric (7), or by soil developing a stable arch over the fabric interstices at the soil/fabric interface (8). Numerous attempts (using grouts, image analyzers, etc.) at verifying these concepts were tried but with little success. A slope markedly greater than zero (as shown on these graphs all slopes are negative) indicates a non-equilibrium situation and suggests a blinding of the fabric's voids (8) or even a partial clogging of the fabric (8). These mechanisms are obviously not desirable and only with additional testing can they be further elaborated upon. In no case, however, was the flow from any of the fabrics tested, for any soil type, completely blocked off.

The idea of measuring hydraulic gradients and of using a gradient ratio (6) was also investigated. It was of interest to note that the gradient ratio began increasing at the same time that the transition between flow curve slopes occurred. However, no unique value of gradient ratio was observed.

The conclusion of the study at this point in time is that flow through soil/fabric systems is a complex phenomenon, governed by both the soil and fabric types and can only be definitively examined by long-term flow tests. The minimum time for such tests to be run varies from a few hours for sand soils, to slightly less than one hundred hours for silt soils, to approximately two hundred hours for soils with high clay content.

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Evaluation of the U.S. Army Corps of Engineer Gradient Ratio Test for Geotextile Performance

Etude de l'essai du rapport de gradient du Corps des Ingénieurs de l'US Army pour l'évaluation des géotextiles

Gradient Ratio testing, following U. S. Army Corps of Engineer test procedures, was conducted on six geotextiles (four woven, two nonwoven) to evaluate their clogging potential. Gap-graded test soil mixtures of ASTM C-190 Ottawa Sand and Vicksburg silt loess were used to simulate worst-case soil behavior. Gradient Ratio values were found to increase slowly with increasing soil silt content until a value of approximately 3 was obtained, and then increase rapidly with further small increases in soil silt content. The Corps of Engineer maximum acceptable test value of 3.0 was thus confirmed by the testing program. Geotextile EOS was not found to relate to geotextile clogging resistance, but woven geotextile percent open area was found to be directly related to geotextile clogging resistance.

L'essai du "rapport de gradient", selon la procédure du Corps des Ingénieurs de l'US Army, a été fait sur six géotextiles (quatre tissés, deux nontissés) pour évaluer leur tendance au colmatage. Des mélanges à granulométrie discontinue de sable d'Ottawa (ASTM C-190) et de silt de Vicksburg ont été utilisés pour simuler les pires conditions. On a trouvé que les valeurs du rapport de gradient augmentaient lentement, avec l'accroissement de la teneur en silt, jusqu' à une valeur de 3 et ensuite augmentaient rapidement. La valeur maximale de 3 acceptée par le Corps des Ingénieurs se trouve ainsi confirmée. La dimension des ouvertures du géotextile n'apparaît pas reliée à la résistance au colmatage, mais l'aire relative des ouvertures de tissés apparaît directement reliée à la résistance au colmatage.

BACKGROUND INFORMATION

Conventional soil used in filters, drains, and erosion control to protect existing cohesionless soil from water-caused internal piping or external erosion must satisfy two criteria to be effective (1):

a. The protective (filter, drain, etc.) soil void spaces must be small enough to prevent piping of existing (protected) soil into or through the filter, and

b. The protective soil must be more permeable than the protected soil, such that hydraulic head loss and seepage forces in the protective soil will be relatively small.

Any geotextile used as a substitute for one or more granular layers in a conventional protective soil system must satisfy these same criteria. Geotextile openings must be small enough to prevent piping or erosion of the protected soil, but the geotextile must maintain a higher permeability than the protected soil. If the geotextile clogs, it may no longer satisfy the higher permeability criterion.

In 1972, C. C. Calhoun, Jr., of the U. S. Army Corps of Engineers studied in-service geotextile piping and clogging potential by measuring head loss at various points through a laboratory-simulated soil-geotextile system (2). The apparatus used by Calhoun consisted of constant head permeameters, each with 8 piezometers, as shown in Figure 1. Piezometer No. 1 measured the tailwater or standpipe elevation, Piezometer Nos. 2-7 measured hydraulic head at locations inside the soil, and Piezometer No. 8 measured the constant head reservoir elevation. Soils used by Calhoun were selected to provide a worst case condition for internal soil piping and fines migration. Uniform Ottawa 20-30 Sand (ASTM C-190) was used as the coarse fraction, with a low plasticity Vicksburg, Mississippi, silt loess as the fine fraction. The Ottawa Sand, with D_{15} of 0.6 mm, could not be used as a protective filter for the Vicksburg loess, with D_{85} of 0.06 mm, because the D_{15}/D_{85} Ratio of 10 exceeds acceptable limits of 4 to 5 (1). Resulting sand-silt soil mixtures thus had maximum potential for internal fines migration and allowed evaluation of fabric performance under worst case conditions. The Corps testing was initiated to determine geotextile performance requirements for critical use applications under severe hydraulic loadings.

As a result of Calhoun's work, the Corps established a piping/erosion loss criterion based on fabric Equivalent Opening Size (EOS), the U. S. Bureau of Standards sieve size of the D_5 geotextile openings, and a clogging/head loss criterion based on geotextile percent open area.

Later (1977) research conducted by the Corps using Calhoun's apparatus established a direct measure of geotextile clogging potential, called the Gradient Ratio (GR), defined (for tests run with the Calhoun apparatus) as the hydraulic gradient through the lower 25 mm (1 in.) of soil plus geotextile divided by the hydraulic gradient through the adjacent 50 mm (2 in.) of soil [between 25 mm (1 in.) and 75 mm (3 in.) above the geotextile]. Gradient Ratio values exceeding 3 were found to signify excessive geotextile clogging, and a limiting value of 3.0 was established by Corps geotextile acceptance specifications (3), for testing with the soil, geotextile, and hydraulic conditions of interest.

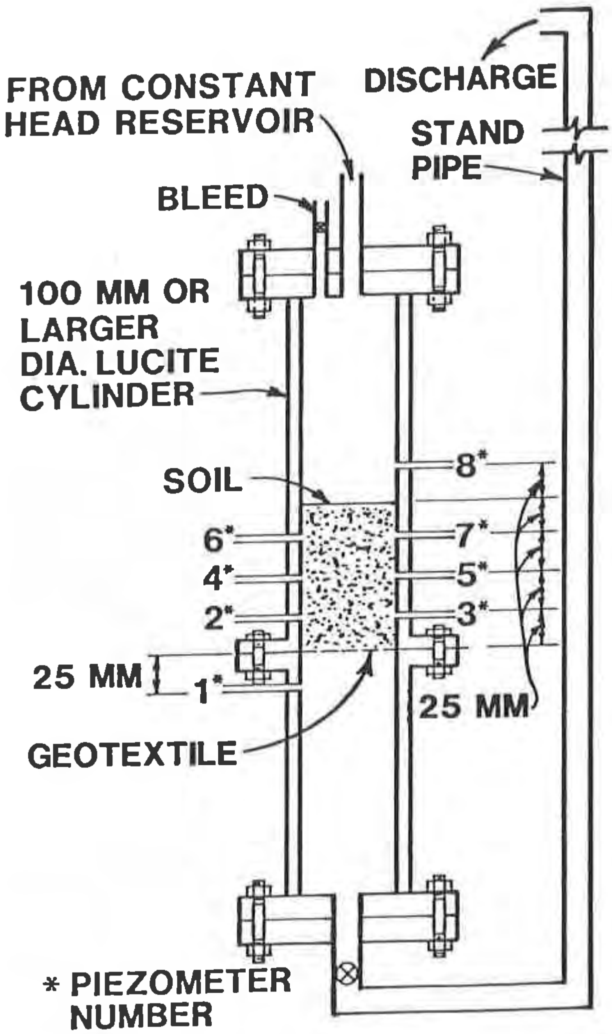

Figure 1. Cross-section detail of U. S. Army Corps of Engineer Gradient Ratio test permeameter.

TESTING PROGRAM

The testing program was designed to evaluate the comparative hydraulic performance of geotextiles, with primary emphasis on geotextile clogging potential. Six geotextiles, representing the basic types of standard geotextiles on the market, were used in the test program. Geotextile trade names, characteristics, and relevant properties are given in Table 1.

The EOS of each geotextile was determined using the glass bead sieving procedure given in Corps of Engineer Specification CW-02215 (3). Successively coarser beads were sieved until 5% or less by weight passed the cloth. The EOS is the "retained on" size of that bead diameter, expressed as a U. S. Bureau of Standards sieve number. Percent open area of the woven geotextiles was determined by projecting each geotextile image onto a screen, tracing the open area outlines, and taking the ratio of the area of the openings to the total geotextile area, expressed in percent. A precise percent open area value cannot be obtained for nonwoven geotextiles.

In order to compare geotextile clogging resistance using accepted Corps of Engineer criteria, four units of the Calhoun test apparatus (Figure 1) were constructed for the test program. Gap-graded ASTM C-190 Ottawa sand and Vicksburg silt loess test soil mixtures of 0%, 5%, 10%, 15%, 20%, 25%, 30%, 40%, 50%, 60%, 70%, and 80% silt by weight were prepared.

The D_{85} of tested soils corresponded to a U. S. Bureau of Standards sieve number between 20 and 30. All geotextiles satisfied both the original 1972 Calhoun and current Corps of Engineer piping criterion, which specifies that the EOS of a geotextile must be less than the D_{85} size of the soil it protects.

Four Calhoun-type test units (Figure 2) were used to evaluate clogging potential of the six geotextiles with increasing soil silt content, using recommended Corps of Engineer GR test procedures (3). Four replicates of each percent soil-geotextile type system were tested. Soil mixtures were tremmied in dry and placed to 100-mm (4-in.) thickness. Each unit was then slowly filled from the bottom with ordinary tap water, to minimize soil disturbance.

The outflow standpipe elevation remained constant for all geotextiles at each silt percentage and was changed with silt percentage. Hydraulic gradients used were those necessary to obtain enough flow for rapid (5 min) soil-geotextile system permeability determinations, ranging from near unity (1) at low silt percentages to over 10 at high silt percentages. Similar operating gradients were used by Calhoun (2) to maximize silt migration and geotextile silt loading. Piezometer

TABLE 1. PROPERTIES OF GEOTEXTILES USED IN TEST PROGRAM

Geotextiles Tested	Manufacturer	Description	EOS (US Bureau of Standards Sieve No.)	% Open Area (%)
Typar 3401	E.I. DuPont de Nemours & Co., Inc.	Gray, nonwoven heat-bonded polypropylene continuous filament	70-100	--
Bidim C-34	Monsanto Textiles Co.	Gray, nonwoven mechanically entangled (needle-punched) polyester continuous filament	70	--
Mirafi 500X	Celanese Corporation	Off-white, woven polypropylene slit film	70-100	1
Poly-Filter X	Carthage Mills	Black, woven polypropylene monofilament	70	5
Nicolon 70/20	Nicolon Corporation	Black, woven polypropylene monofilament	70	20
Poly-Filter GB	Carthage Mills	Black, woven polypropylene monofilament	40	30

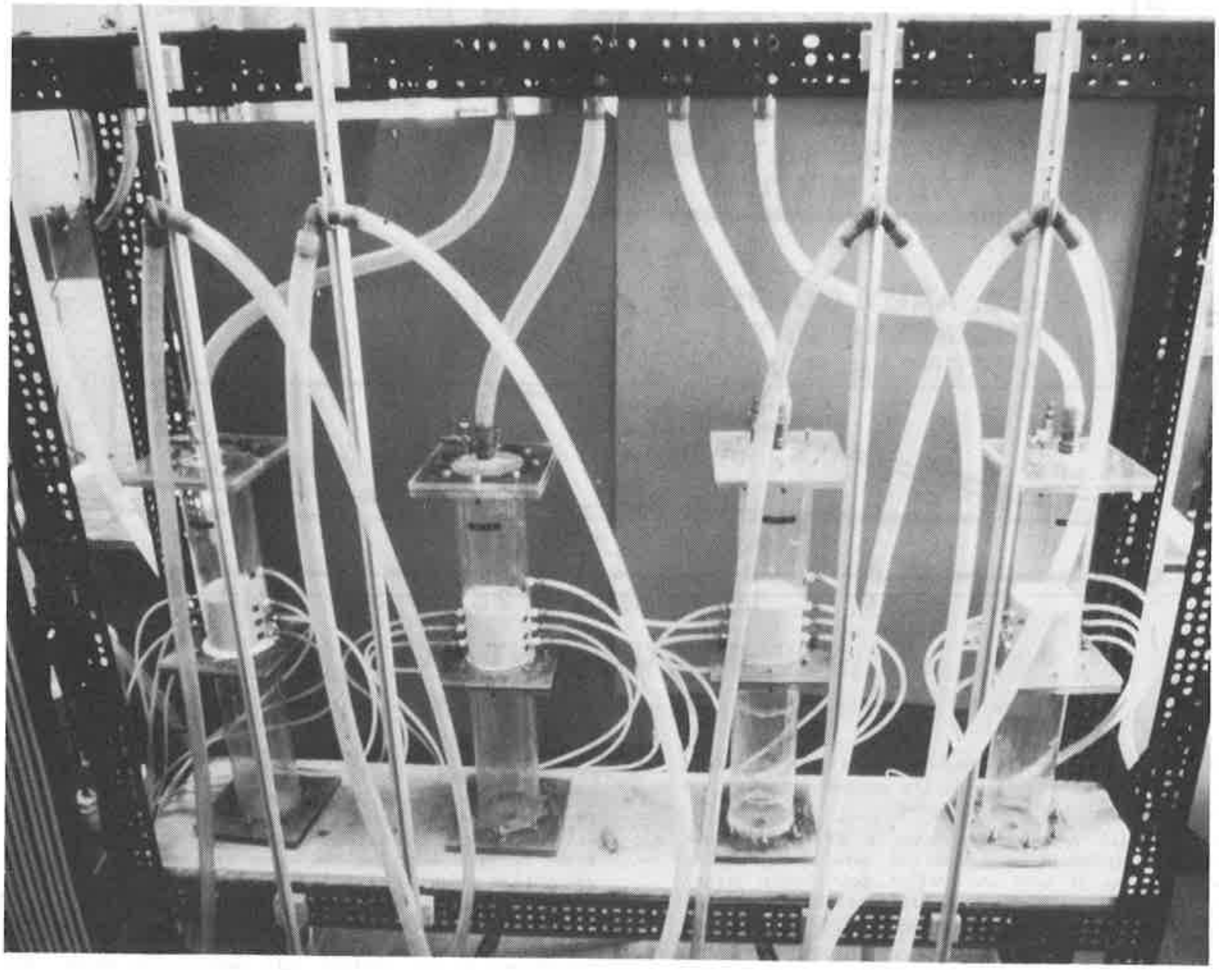

Figure 2. Four Calhoun-type permeameters used in Gradient Ratio test program.

readings were taken every 15 min until they stabilized (1 hr-2 hr) and flow rate measurements were recorded periodically. The GR was determined after 24 hr of testing (3).

After testing, soil samples were taken from each test unit, over intervals of 0 mm-6 mm (0 in.-0.25 in.), 6 mm-25 mm (0.25 in.-1 in.), 25 mm-50 mm (1 in.-2 in.), and 50 mm-75 mm (2 in.-3 in.) above the geotextile, and the final silt percentage distribution determined.

TEST RESULTS AND EVALUATION

The test system GR, defined previously, was used to quantitatively evaluate geotextile performance at each soil-silt content. Corps of Engineer Specification No. CW-02215 allows a maximum Gradient Ratio of 3.0.

Gradient Ratio values for each soil-geotextile combination were computed as the average of four individual tests and are plotted versus percent silt in Figure 3. The various geotextiles exceeded the maximum GR of 3.0 at the following silt percentages:

Geotextile	Maximum Allowable Soil Silt Percentage (GR $\leq$ 3.0)
Mirafi 500X	0% (Clean Sand)
Typar 3401	0.5%
Bidim C-34	18.5%
Poly-Filter X	25%
Nicolon 70/20	60%
Poly-Filter GB	Could not clog, maximum GR = 1.1 at 80% silt

Review of the various test data indicated that:

a. Except for the slit-film Mirafi 500X, which exceeded an allowable GR of 3.0 at 0% silt, and the 30% open area woven monofilament Poly-Filter GB, which did not exceed the maximum allowable GR of 3.0 at the highest silt percentage tested, the remaining four geotextiles showed a small increase in GR with increasing percent silt content until a GR of 3.0 was exceeded, followed by an extremely rapid increase in GR with fairly small further increases in silt percentage. These findings tend to confirm the limiting GR of 3.0 used by

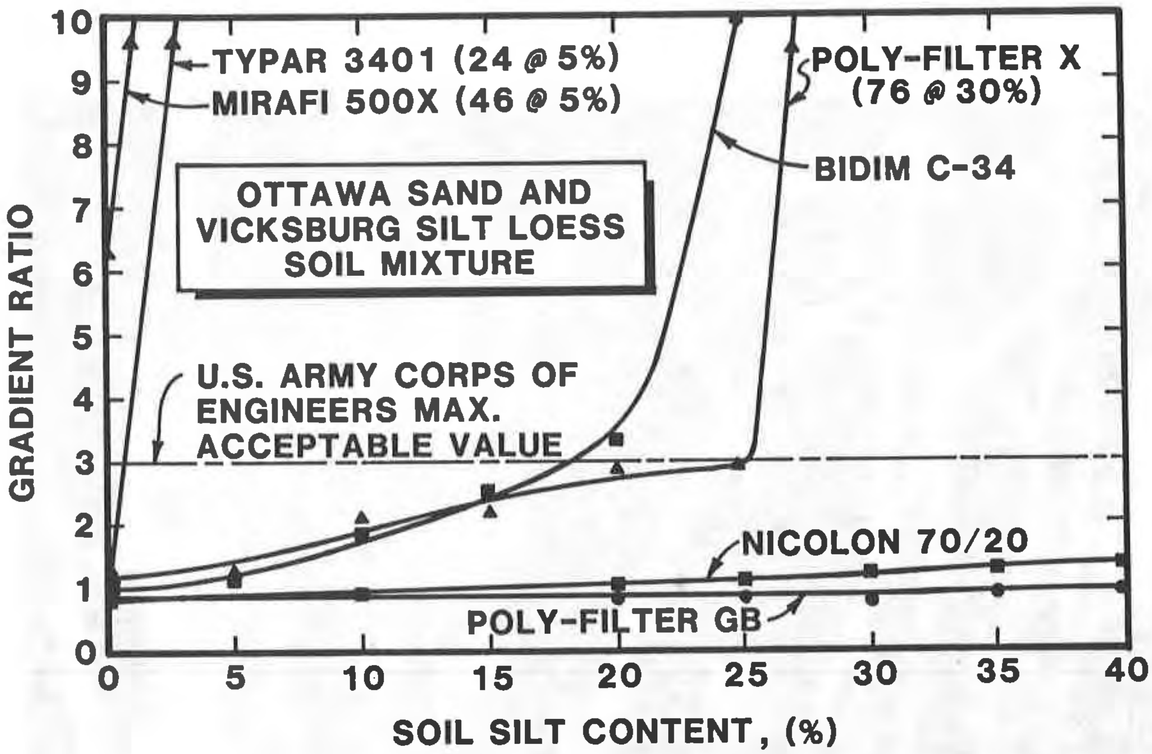

Figure 3. Gradient Ratio as a function of soil silt content for geotextiles tested.

the U. S. Army Corps of Engineers as the boundary between the minimal soil-geotextile system clogging and high soil-geotextile system clogging.

b. For all geotextiles, once a GR of 3.0 was exceeded, an order of magnitude or more reduction in system beginning of test and end of test Darcy permeability was noted to occur, indicating that the clogged geotextile had become the least permeable part of the soil-geotextile system.

c. Once a GR of 3.0 was exceeded, noticeable amounts of silt were found deposited on or in all geotextiles. However, despite the gap-graded nature of the Calhoun test soils, significant changes in soil silt content were found to occur only in the 6 mm (0.25 in.) above the geotextile, increasing slightly (2%-4% silt) for the Mirafi 500X and Typar 3401 geotextiles, and decreasing approximately the same amount for all other geotextiles tested, with the percentage of silt loss remaining essentially constant, even at higher test silt percentages. For all samples and all silt percentages where silt loss occurred, loss (noted through discoloration of test water) occurred only during the initial 10 min-15 min of the test, with the majority of loss occurring in the first 5 min. When the GR was less than or equal to 3.0, a slight (less than one order of magnitude) increase in soil-geotextile system permeability was noted to accompany the soil loss. It is believed these data tend to indirectly confirm the formation of a "mini-graded filter" in the soil immediately behind the geotextile, as originally suggested by Calhoun (2).

d. For the nonwoven needle-punched geotextile (Bidim C-34), flow occurred through the needle-punched EOS 70 holes even when the felt-like portion of the geotextile became clogged with silt.

e. The three monofilament woven geotextiles had the best clogging resistance among the six geotextiles tested, and clogging resistance increased with increase in woven fabric percent open area, substantiating Calhoun's original Corps of Engineer conclusions (2). Based on test data for the woven geotextiles, the silt percentage which caused a GR of 3.0 is plotted versus woven geotextile percent open area in Figure 4. This Figure may be used to estimate the minimum woven geotextile percent open area required for acceptable clogging resistance at the various silt percentages noted, and for the critical/severe conditions the GR test is designed to simulate.

f. For the geotextiles tested, geotextile EOS was found to have no relationship to geotextile clogging behavior.

CONCLUSIONS

Based on the test program conducted and discussed herein, it may be concluded that:

a. The 1977 U. S. Army Corps of Engineer Gradient Ratio test is an acceptable method to evaluate and quantify clogging potential of geotextiles. Geotextiles definitely should not be used in severe/critical design applications where the soil-geotextile system Gradient Ratio exceeds 3.0.

b. All tested geotextiles satisfied both the 1972 and 1977 U. S. Army Corps of Engineer EOS piping criteria, and minimal soil piping was noted for any soil-geotextile combination.

c. Geotextile EOS was found to have no relationship to geotextile clogging behavior. Woven geotextile percent open area was almost directly related to geotextile clogging potential.

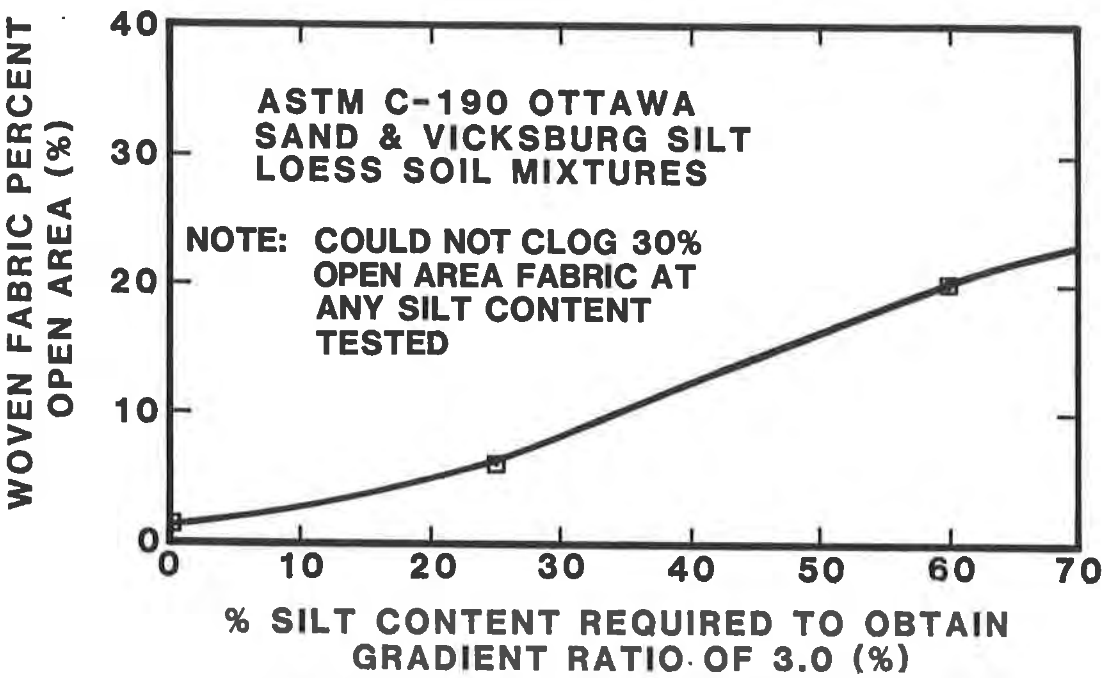

Figure 4. Woven geotextile percent open area vs. percent silt to develop Gradient Ratio of 3.0

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Filter Criteria for Geotextiles

Critères de filtre pour les géotextiles

Classical criteria used in soil mechanics for granular filters such as sand were established as a result of a combination of laboratory findings and theoretical considerations. In the recent past, considerable efforts have been devoted to experimental studies of geotextile filters. As an attempt at complementing experimental findings, a theoretical analysis of the filtration mechanism related to geotextiles is presented in this paper. Although the considered mechanism is similar to the mechanism of filtration related to granular materials, the derived criteria for permeability and opening size of geotextile filters are different from the corresponding criteria related to granular filters. Also the analyis presented in this paper shows that simplistic criteria often used to select geotextile filters can be misleading. A design example shows that the use of the simplistic criteria, instead of the criteria proposed in this paper, would have caused a significant risk of piping in a dam where a geotextile filter has been used successfully.

Les critères classiques utilisés en mécanique des sols pour les filtres granulaires, en sable par exemple, ont été établis en combinant résultats expérimentaux et considérations théoriques. Ces dernières années, de nombreux travaux expérimentaux ont été consacrés aux filtres géotextiles. En complément à ces travaux expérimentaux, une étude théorique du mécanisme de filtration avec géotextile est présentée dans cet article. Bien que le mécanisme considéré soit analogue à celui relatif aux filtres granulaires, les critères obtenus pour la perméabilité du géotextile et pour la dimension de ses ouvertures sont différents des critères relatifs aux filtres granulaires. De plus, l'analyse présentée dans cet article montre que les critères simplistes souvent utilisés pour sélectionner les filtres géotextiles peuvent conduire à des erreurs. Un exemple montre qu'utiliser les critères simplistes au lieu de ceux proposés dans cet article aurait provoqué un important risque d'érosion interne dans un barrage où un filtre géotextile a été utilisé avec succès.

INTRODUCTION

It is difficult to simulate in a laboratory the longterm behavior of a filter in the field. To decrease the testing time, unrealistic values of some parameters (such as the gradient) are often selected, and results are misleading. When tests are properly carried out, facts concerning the mechanism of clogging can be learned but no simple results are obtained because many parameters are involved. Consequently, classical criteria for granular filters were established as a combination of laboratory findings and theoretical considerations, and justified, a posteriori, by years of successful field applications.

Similarly, there is little hope to establish practical filter criteria for geotextiles from laboratory tests only. It seems appropriate to use the combined approach (laboratory and theory) that proved successful for soils. Filter criteria for geotextiles, established using theoretical considerations, are presented in this paper.

This paper is user oriented. Design criteria and examples are presented first. Definitions, derivations and comments may be found in Appendices.

1. DISCUSSION OF THE MECHANISM OF FILTRATION

A filter must retain the soil and allow the water to pass through. These two requirements are contradictory when they are formulated too strictly. If it were required that all soil particles be retained, an impervious screen would be needed, in which case water would not flow through it. Conversely, if it were required that the flow of water be absolutely unimpeded by the filter, openings of the filter should be so large that practically no soil particles would be retained.

Consequently, the two requirements must not be formulated too strictly: the filter must only negligibly impede the flow of water and must also prevent destruction of the soil structure by erosion. A good filter has openings both large enough and small enough. It needs openings large enough to allow water to flow almost freely (this could result in the loss of some of the finest soil particles) but it should have openings small enough so that the particle skeleton, which gives the soil structure stability, is not disturbed as a result of the loss of some fine particles. To evaluate these two "reasonable requirements", a filtration theory must be developed. A complete theory would be difficult to formulate due to two reasons, the variety of phenomena involved (two phase flow and capillarity, chemical and electrical interactions between filter and particles, erosion, variation of the mechanical behavior of soil as a function of water content and pore water pressures, etc.) and the large amount of parameters: (i) geometrical conditions (shape of the soil mass, location of the fluid, flow direction which may vary) and mechanical conditions (gravity, stresses); and (ii) properties of materials such as the fluid (composition, density, viscosity), the soil particles (shape, dimension, distribution, density, chemical nature), the soil (density, mechanical properties such as friction and cohesion, permeability), the constituents of the filter (shape, dimensions, distribution, density and chemical nature of solid elements (grains or fibers) of filter, and void distribution of filter), the filter (continuity, permeability, mechanical properties such as compressibility).

At the present time we are far from having a complete theory dealing with the above phenomena and parameters. We use a simplified approach for granular filters as well as geotextiles, which consists of considering two criteria, established separately by neglecting some phenomena and parameters: the permeability criterion and the filtration criterion. Consequently, soil filtration with geotextiles is neither better, nor worse understood than soil filtration with granular filters. The proposed criteria for geotextiles are probably as valid as the classical criteria used for granular filters discussed hereafter.

2. CRITERIA FOR GRANULAR FILTERS

Criteria for granular filters can be found in every soil mechanics textbook. Original work was done by Terzaghi in 1922 (1) and was followed by considerable work done by many researchers and organizations. As a result, various expressions of the criteria can be found. A typical one is:

$$d_{15}\text{ (filter)} > 4\ d_{15}\text{ (soil)} \qquad (1)$$

$$d_{15}\text{ (filter)} < 4\ d_{85}\text{ (soil)} \qquad (2)$$

Eq. 1 is the permeability criterion and Eq. 2 is the retention criterion.

Comments related to Eq. 1. It is known that the hydraulic conductivity (coefficient of permeability) of a granular material with a uniform particle size distribution is proportional to the square of the diameter of its particles. When the particle size distribution is not uniform, it is classical to consider that the hydraulic conductivity is proportional to d_{10}^2 or d_{15}^2. Therefore, Eq. 1 implies that the permeability of the filter must be greater than 16 times (say approximately 10 times) the permeability of the soil in contact.

Comments related to Eq. 2. In a granular material made of identical spheres in the most compact state (hexagonal arrangement), the ratio between the diameter of the spheres and the diameter of the largest sphere likely to go through this granular material is: $3/(2-\sqrt{3}) = 6.5$ .In a loose state (cubic arrangement), the ratio becomes $1/(\sqrt{2}-1) = 2.4$ .An approximate average value is 4. The same ratio can be applied to d_{10} or d_{15} in the case of a granular filter with a non uniform particle size distribution. In other words, Eq. 2 means that large particles of soil (d_{85}) must be larger than the openings of the filter ($d_{15}/4$).

Summary. Classical criteria for granular filter mean that: (1) the hydraulic conductivity (coefficient of permeability) of the filter must be approximately 10 times larger than the hydraulic conductivity of the soil; and (2) large soil particles (d_{85}) should be larger than openings of the filter.

3. CRITERIA FOR GEOTEXTILE FILTERS

Permeability criterion. As shown in Appendix 2, the permeability criterion for geotextile filters is:

$$k\text{ (geotextile)} > 0.1\ k\text{ (soil)} \qquad (3)$$

The hydraulic conductivity k (coefficient of permeability) of a geotextile filter must be at least one tenth of the hydraulic conductivity of the soil in contact. This is different from Eq. 1 related to granular filters. Comments are made in Appendix 2.

Retention criterion. In Appendix 3, the retention criterion is established for the case when the soil in contact with the filter is cohesionless. This criterion, presented in Table 1 and Fig. 1, depends on the density of the soil (characterized by I_D) and on the slope of its grading curve (characterized by C'_u). The required opening size of the geotextile can be larger or smaller than the soil particles, depending on the values of I_D and C'_u. Comments are made in Appendix 3.

Limitations to the use of the proposed retention criterion are: (i) the criterion is conservative in the case of a clay with high cohesion (the same limitation exists with the classical criterion for granular filters); (ii) the criterion must not be used with unstable gap-graded soils (see Fig. 3, soil 1) because with such soils clogging of filters cannot be prevented; (iii) the criterion cannot be used when the soil structure is repeatedly destroyed by a turbulent flow of water that changes periodically in both direction and velocity (bank protections).

Table 1. Retention criterion (I_D and C'_u are defined in Appendix 1). The criterion is represented graphically in Fig. 1.

	Density index of the soil (Relative density)	Linear coefficient of uniformity of the soil $1 < C'_u < 3$	$C'_u > 3$
loose soil	$I_D < 35\%$	$O_{95} < C'_u d_{50}$	$O_{95} < \frac{9}{C'_u} d_{50}$
medium dense soil	$35\% < I_D < 65\%$	$O_{95} < 1.5\ C'_u d_{50}$	$O_{95} < \frac{13.5}{C'_u} d_{50}$
dense soil	$I_D > 65\%$	$O_{95} < 2C'_u d_{50}$	$O_{95} < \frac{18}{C'_u} d_{50}$

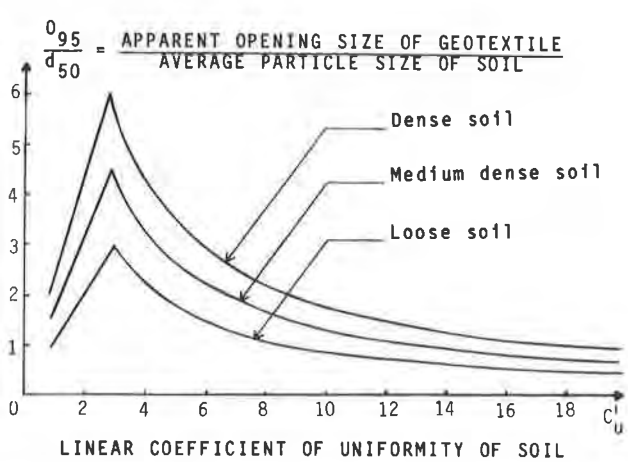


Fig. 1. Retention criterion for geotextile filter (see Table 1).

4. DESIGN EXAMPLE

The particle size distribution curve shown in Fig. 2 is related to Valcros Dam (2). At the time of the construction of this dam (1970), little was known about filter criteria for geotextiles. The only design consideration was to check that the opening size of the geotextile available on the site (then believed to be approximately 0.1 mm) was smaller than the d_{85} of the soil (7 mm, according to Fig. 2). For twelve years, Valcros Dam has performed well. Today it is an interesting exercise to verify this early design using the criteria presented above.

First, the retention criterion is considered. For d_{50} = 0.47 mm and C'_u = 49, according to Fig. 2, the required maximum opening size of the geotextile is determined using Table 1:

$$O_{95} < 18 \times 0.47/49 = 0.17 \text{ mm}$$

This requirement is satisfied by the geotextile used in Valcros Dam because its apparent opening size, O_{95}, lies between 0.13 and 0.17 mm according to various tests. It is noteworthy that the use of the simplistic criterion, $O_{95} < d_{85}$ = 7 mm, could have led to an important mistake (significant risk of piping).

The permeability criterion (Eq 3) is easily verified because the hydraulic conductivities are approximately 10^{-4}m/s for the geotextile and 10^{-7}m/s for the soil.

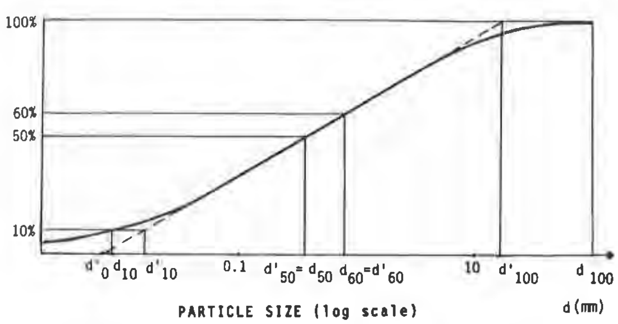


Fig. 2. Particle size distribution curve of Valcros Dam (d'_{100} = 17 mm; d'_0 = 0.007 mm; $C'_u = \sqrt{17/0.007}$ = 49).

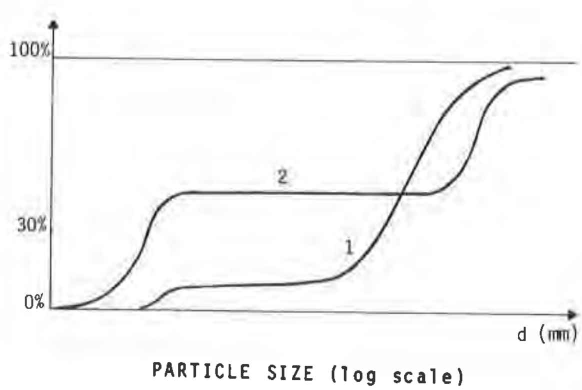


Fig. 3. Particle size distribution curves of two typical gap-graded soils.

APPENDIX 1. DEFINITIONS

Definitions related to soils. A typical curve representing the particle size distribution of a soil is shown in Fig. 2. The slope of this curve is traditionally evaluated using the coefficient of uniformity, $C_u = d_{60}/d_{10}$. To express the retention criterion, it is more appropriate to use the slope of the central portion of the curve, thus eliminating the coarsest and finest particles which have a negligible influence on the stability of the soil structure in the filtration process. A straight line is drawn as close as possible to the central portion of the curve and a "linear coefficient of uniformity" is defined as $C'_u = d'_{60}/d'_{10}$ (d'_{60} and d'_{10} being related to the straight line). Because of the log scale:

$$C'_u = \frac{d'_{50}}{d'_0} = \frac{d'_{60}}{d'_{10}} = \ldots = \frac{d'_{90}}{d'_{40}} = \frac{d'_{100}}{d'_{50}} = \sqrt{\frac{d'_{100}}{d'_0}} \qquad (4)$$

It is impossible to draw a straight line with a gap-graded soil (Fig. 3). Two typical cases are: (i) in a silty or clayey gravel including less than 30% fine particles (soil 1 in Fig. 3) the space between the gravel particles is not filled and the soil structure is not stable when water is flowing; and (ii) in a silty or clayey gravel including more than 30% of fine particles (soil 2 in Fig. 3), the gravel particles are not in contact with each other and they "float" in the fine-soil matrix; in this case, one will consider only the particle size distribution curve related to the fine portion of soil for the retention criterion.

Two values of the linear coefficient of uniformity are particularly interesting:

- C'_u = 1. The soil has a uniform particle-size distribution (in other words, all the particles have the same size), and the void space between particles is large (even after compaction).
- C'_u = approximately 3. It has been shown theoretically and experimentally (3) that soils with a coefficient of uniformity of approximately 3 obtain the highest densities (that is, the most complete filling of space with maximum interlocking of particles), if appropriate compaction is applied.

When C'_u is greater than 3, the soil grain-size distribution is too spread out to obtain a perfect interlocking: there are not enough particles of each dimension. If C'_u is between 1 and 3, it is possible, with appropriate compaction, to obtain the maximum interlocking of particles.

The stability of the structure of a cohesionless soil is related to its density index I_D (relative density):

$$I_D = \frac{e_{max}-e}{e_{max}-e_{min}} \times 100\% = \frac{\rho_{dmax}}{\rho_d} \times \frac{\rho_d - \rho_{dmin}}{\rho_{dmax}-\rho_{dmin}} \times 100\% \quad (5)$$

where: e, e_{max}, e_{min}, = void ratio of the soil in place, in its loosest and densest state respectively (dimensionless); ρ_d, ρ_{dmax}, ρ_{dmin} = dry density of the soil in place, in its densest and loosest state respectively (kg/m^3).

The soil property used in the permeability criterion is its hydraulic conductivity (coefficient of permeability), k (m/s).

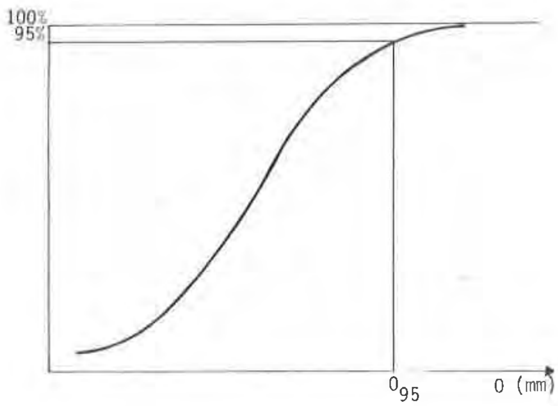


Fig. 4. Opening size distribution curve ("porometric curve") of a geotextile.

Definitions related to the geotextile. A typical "porometric curve" representing the opening size distribution of a geotextile is shown in Fig. 4. This curve can be obtained, with more or less accuracy, by sieving calibrated glass beads through the considered geotextile. The retention ability of a geotextile is governed by its largest openings. Consequently, the upper part of the porometric curve is the most important and it is usually characterized by the O_{95} (Fig. 4), often called apparent opening size.

The hydraulic conductivity, k (m/s), of the geotextile used in this study is related to a flow of water normal to the plane of the geotextile. Tests in a permeameter give the hydraulic permittivity, ψ (s^{-1}), of the geotextile. Hydraulic conductivity is deduced using $k = \psi T_g$, where T_g = thickness of the geotextile (m).

APPENDIX 2. PERMEABILITY CRITERION

Eq. 3 has been established as follows.

Principle. As mentioned in Section 1, disturbances to water flow due to the filter should be negligible. Two disturbances must be considered: the decrease in flow and the increase in pore water pressure within soil. The analysis shows that these two disturbances result in the same criterion.

Assumptions. As indicated in Section 1, it would be very difficult to take into account all the parameters and simplifying assumptions must be made. We consider the simple case of a saturated soil (to eliminate capillarity) where water flows vertically (to simplify the influence of gravity) (Fig. 5).

Theoretical analysis. The following results were established using Darcy's formula:

. Flow per unit area with a filter:

$$\frac{Q}{A} = \frac{h}{\frac{T_f}{k_f} + \frac{T_s}{k_s}} \tag{6}$$

. Flow per unit area without a filter:

$$\frac{Q'}{A} = \frac{hk_s}{T_s} \tag{7}$$

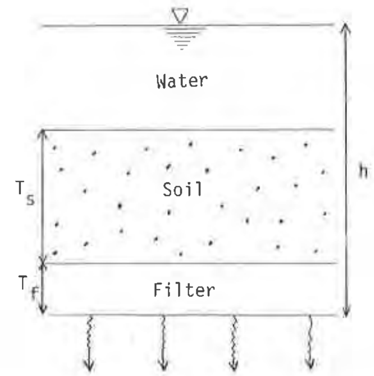


Fig. 5. Vertical flow through soil and filter.

. Increase of pore water pressure within the soil caused by the filter:

$$\Delta p = \left(\frac{h}{1 + \frac{T_s\,k_f}{T_f\,k_s}} H_f\right) \rho_w g \tag{8}$$

where: h = hydraulic head (m); T_s, T_f = soil and filter thicknesses, respectively (m); k_s, k_f = hydraulic conductivities of soil and filter respectively (m/s); ρ_w = water density (kg/m^3); and g = gravity (m/s^2).

For both the flow and pore water pressure disturbances caused by the filter to be small, $T_f k_s/(T_s k_f)$ must be small compared to 1. In geotechnical engineering, a disturbance is usually considered as negligible when it is lower than 10%. Furthermore a safety factor of 10 is recommended for calculation when soil permeability is involved. The condition may then be stated as follows:

$$\frac{T_f k_s}{T_s k_f} < 10\%/10 = 0.01 \tag{9}$$

The T_f value is of the order of 1 m for granular filters and 1 to 10 mm for geotextiles (10 mm is used since this is conservative with respect to this calculation). The length T_s corresponds to the final part of the flow through the soil where the water speed increases as it comes close to the drain. An order of magnitude of 10 m is reasonable. The following permeability criteria are then deduced from Eq. 9:

$$k_f > 10\,k_s \text{ for a granular filter} \tag{10}$$

$$k_f > k_s/10 \text{ for a geotextile filter} \tag{11}$$

Comments. (i) According to the calculations above, the hydraulic conductivity of a granular filter must be 10 times larger than the hydraulic conductivity of the soil. This is in perfect agreement with the classical Eq. 1, as discussed in Section 2 and checks the validity of our calculations. (ii) Using the same calculations for a geotextile, it appears that the hydraulic conductivity of a geotextile filter must be larger than only one tenth of the hydraulic conductivity of the soil. Therefore it may be concluded that specifications, requiring that the hydraulic conductivity of the geotextile filter be larger than the hydraulic conductivity of the soil, are exaggerated. They demand more of geotextiles than classical criteria demand of granular filters.

APPENDIX 3. RETENTION CRITERION

The retention criterion presented in Table 1 and Fig. 1 (Section 3) has been established as follows.

Principle. As indicated in Section 1 only a simplified analysis is possible. But, the analysis must not be oversimplified: it would be very simple, but unrealistic, to choose a filter as a sieve that is agitated until all the particles smaller than mesh openings pass through. It is known, by experience, that filters are able to retain soils including numerous particles which, individually, would be able to pass through their openings.

Soil retention by a filter depends on the stability of the soil structure which is governed by the following parameters: soil properties, geometric characteristics of the soil-geotextile system and applied loads (flow drag, gravity, external loads, contact actions between filter and soil).

Assumptions. As indicated in Section 1, it would be very difficult to take into account all the parameters and simplifying assumptions must be made.

Soil properties having an influence on the stability of the soil structure are cohesion (which governs attraction between particles), and density and grain size distribution (which govern interlocking between particles). In this study, only cohesionless soils are considered because cohesive soils are more stable in filtration process.

Geometric characteristics of the soil-geotextile system are the sizes and the shapes of soil particles and geotextile openings. In this study, shapes are not taken into account. Each soil particle is characterized by only one dimension (i.e., it is considered as a sphere). Each geotextile opening is characterized by only one dimension (i.e., it is considered as a circular hole). In fact, shapes of openings of various types of geotextiles (wovens, nonwovens) are quite different and might have a significant influence on the mechanism of filtration.

Flow drag (proportional to hydraulic gradient) plays an important part, but is difficult to evaluate. It is conservative to consider that the flow drag is significant enough to move soil particles that are not in direct contact with solid elements of the filter or confined in the soil structure. Nevertheless, we do not consider the extreme case of a soil structure systematically destroyed by a turbulent and alternating water flow as it is the case for filters exposed to wave action. This case is similar to sieving where the particles, permanently agitated, pass through the filter if they are smaller than filter openings.

Gravity and external loads (except vibrations and shocks) usually improve the stability of a cohesionless soil, through interparticle friction. This effect does not seem essential. It is neglected in this study, which is conservative in most cases.

Contact actions include physico-chemical filter-soil attractions (which are usually negligible in cohesionless soils) and stresses at filter-soil contact. To evaluate the effect of these stresses upon the stability of the soil structure, the mechanical behavior of the soil should be taken into account. To simplify the evaluation, the considered filter-soil stresses are such that the soil particles in contact with the solid elements of the filter are perfectly immobile. In other words, the only soil particles supposedly able to move (under a water flow effect) are those which are in front of the filter openings and those that are immediately behind them.

As a result of these simplifying assumptions, the retention phenomenon becomes a geometric problem, the parameters of which are: (i) diameter of soil particles and geotextile openings; and (ii) interlocking of soil particles (governed by density index I_D and linear coefficient of uniformity C'_u). These parameters are defined in Appendix 1.

Theoretical analysis. Filter opening sizes are compared to soil particle sizes in two cases depending on C'_u. In each case, two sub-cases are considered, depending on I_D.

If C'_u is lower than 3, all the soil particles are interlocked, yielding a stable structure, as indicated in Appendix 1. The only requirement, therefore, is that the filter retains the coarsest particles so that the entire soil will be retained. As the stability is also a function of density, let us consider two sub-cases: dense soil and loose soil.

In the sub-case of a dense soil, two of the coarsest particles must move simultaneously to leave the soil structure unstable (Fig. 6a). As mentioned in Appendix 1, the coarsest particles of the straight line distribution (d'_{100}) must be considered instead of the actual coarsest particles (d_{100}). Consequently, the retention criterion is:

$$O_{95} < 2d'_{100} \tag{12}$$

That is, from Eq. 4 and Fig. 2:

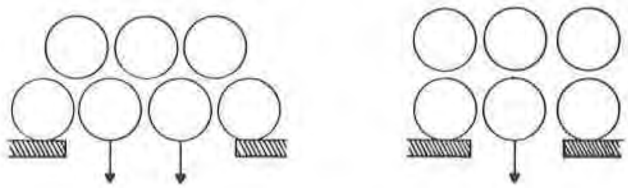
$$O_{95} < 2C'_u d'_{50} = 2C'_u d_{50} \tag{13}$$

In the sub-case of a loose soil, it is sufficient that one of the coarsest particles moves to leave the soil structure unstable (Fig. 6b). The retention criterion then becomes:

$$O_{95} < C'_u d_{50} \tag{14}$$

If C'_u is greater than 3, the soil particles are not able to interlock perfectly and, then, do not form a stable structure (as mentioned in Appendix 1). Therefore, when $C'_u > 3$, it is not sufficient that the filter retains the coarsest particles to allow the entire soil to be retained. To prevent migration of fine particles, the filter must be designed considering only the particles finer than a certain size d'_x. These particles must have a linear coefficient of uniformity of 3, in order to ensure that their structure is stable. Consequently, Eq. 12 for a dense soil becomes:

$$O_{95} < 2\,d'_x \tag{15}$$



(a) (b)

Fig. 6. Influence of the soil density on the retention criterion: (a) dense soil; (b) loose soil.

where: $d'_x = (3)^2 d'_o$ (because d'_x plays the part of d'_{100} for the finer fraction of the soil).

According to Eq. 4:

$$d'_o = d'_{50}/C'_u = d_{50}/C'_u \qquad (16)$$

Hence:

$$O_{95} < 18\ d_{50}/C'_u \qquad (17)$$

For a loose soil, the factor 2 should be removed from Eq. 15 as it is seen by comparing Eq. 13 and 14. Consequently, in Eq. 17, valid for a dense soil, 18 must be replaced by 9 in the case of a loose soil. The curve related to a medium dense soil has been arbitrarily drawn half way between the curves related to dense and loose soils.

Comments. The retention criterion established as explained above and presented in Table 1 seems logical because it depends on two parameters, C'_u and I_D, governing the interlocking of soil particles and, consequently, the stability of the soil structure. A simpler criterion is often mentioned: $O_{95} < d_{85}$. In order to compare the two criteria, Fig. 1 is transformed into Fig. 7, using Eq. 18 deduced from the equation of the straight line of Fig. 2 (assuming $d_{85} = d'_{85}$):

$$d_{85} = d_{50}\ C'_u{}^{0.7} \qquad (18)$$

It is seen on Fig. 7 that: (i) for small values of C'_u, the simplistic $O_{95} < d_{85}$ criterion leads to excessively small values of O_{95}, thus increasing the risk of clogging; and (ii) for large values of C'_u, the simplistic $O_{95} < d_{85}$ criterion leads to values of O_{95} too large, thus increasing the risk of piping, as already mentioned in section 4.

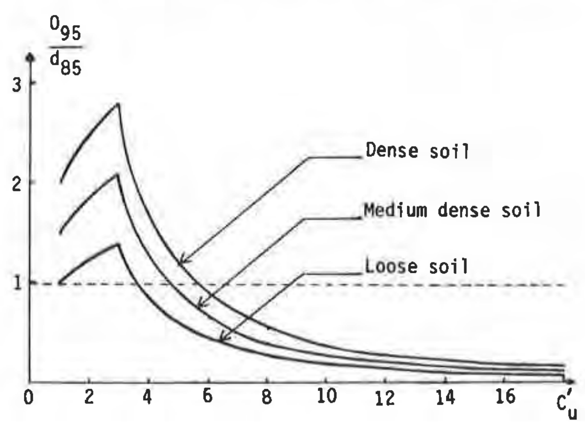


Fig. 7. Comparison of the proposed retention criterion (solid curves deduced from Fig. 1 using Eq. 18) with the simple criterion $O_{95} < d_{85}$ (dashed line).

APPENDIX 4. EXPERIMENTAL DATA

Tests on various types of geotextiles, with medium dense soils having different values of the coefficient of uniformity, have been conducted by Schober and Teindl (4). Results related to needlepunched geotextiles are close to the curve for dense soils while results for other geotextiles (wovens and heatbonded nonwovens) are close to the curve for loose soils. A possible explanation is that soil particles in contact with a smooth geotextile (such as a woven or a heatbonded nonwoven) are rather free to move and have a tendency to loosen, while soil particles in contact with a rough geotextile (such as a needlepunched nonwoven) are kept in a dense state. As a result: (i) the upper curve in Fig. 1 could be used in the case of dense soils with a needlepunched geotextile filter; and (ii) the lower curve in Fig. 1 could be used in the case of loose soils, regardless of the type of geotextile, and in the case of smooth geotextiles (wovens, heatbonded nonwovens) regardless of the density of the soil.

McGown, Murray and Kabir (5) have established correlations between test data from various investigators. These test data are in good agreement with the theoretical curves presented in Fig. 1.

CONCLUSION

The following criteria are often used to select geotextile filters: (1) a permeability criterion, requiring that the geotextile be more permeable than the soil; and (2) a retention criterion requiring that the apparent opening size, O_{95}, of the geotextile be smaller than the d_{85} of the soil. As it is shown in this paper, these criteria cannot be recommended: (1) the permeability criterion is excessively demanding and can eliminate geotextiles that are actually suitable; and (2) the retention criterion is unrealistic and can be dangerous. The two criteria are misleading because they look like well known criteria for granular filters and, therefore, are easily accepted by engineers.

Using a rational approach, filter criteria have been established for geotextiles. Although these criteria may not "look like" the well known criteria for granular filters, they are probably as valid because they are established on a comparable basis. The proposed criteria have been tested on several projects and they are now used as a routine design method by engineers at Woodward-Clyde Consultants.

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Contribution to the Study of the Clogging of Geotextiles. Morphological Approach

Contribution à l'étude du colmatage des géotextiles. Approche Morphologique

Clogging of several geotextiles withdrawn from Civil Engineering works have been studied : permeability parameter and soil contamination rate. A morphological study has been carried out in Optical and Electron Microscopies on thin and ultrathin cross section obtained in drillings of intact complex soil-textile. Three types of clogging have been defined : lower, internal, upper. Even for highly clogged geotextile, permeability remains much greater than permeability of soils. Lower clogging of geotextile is owe to soil in place, its rate of water and conditions of handling. A scheme is proposed to account for the development of the internal clogging by clay towards a spongious morphological state which corresponds to an equilibrium state between clogging and water flowing.

Le colmatage de plusieurs dizaines de prélèvements de géotextiles a été étudié à travers les paramètres tels que le taux de pollution en terre et la perméabilité résiduelle. Une étude morphologique poussée a été entreprise en microscopies électronique et optique sur des lames ultra-minces et minces réalisées dans des carottages de complexes intacts sol-textile des ouvrages. Trois types de colmatage géotextile ont été repérés : de base, interne et supérieur. Même les géotextiles très colmatés conservent une grande perméabilité par rapport au sol en contact. Le colmatage de base est essentiellement dû à la nature du sol en place, à son état hydrique et aux conditions de mise en oeuvre. Un schéma est proposé pour expliquer l'évolution du colmatage interne par l'argile qui conduit à une structure alvéolaire correspondant à un état d'équilibre entre colmatage et circulation de l'eau.

INTRODUCTION

Parmi les problèmes que pose le vieillissement des géotextiles, vu notamment sous l'aspect des intéractions de l'environnement (sol, eau de ruissellement, végétation) et les géotextiles, le colmatage est certainement un des plus critiques, car il touche à la pérénité de la perméabilité du produit. De nombreux essais de laboratoire sont proposés pour apprécier a priori les caractéristiques hydrauliques des géotextiles, qu'il s'agisse du diamètre de filtration ou de la perméabilité (1). Certes ces tests peuvent également être mis en oeuvre pour apprécier la perméabilité résiduelle de géotextiles ayant fonctionné et donc leur taux de colmatage. Cette approche ne fournit pas néanmoins d'information directe sur la manière dont sont piégées les particules de sol dans la structure du géotextile et comment s'établissent les intéractions sol-textile. L'approche morphologique au niveau du microscope optique sur lames minces ou du microscope électronique à balayage (2) peut fournir des éléments de réponse à ces questions sur les intéractions sol-fibres. Elle est d'ailleurs utilisée avec succès, avec Analyseur d'Images interposé, à l'Ecole Polytechnique de Montréal par le Pr ROLLIN (3) pour l'étude du colmatage des géotextiles après des tests de simulation en laboratoire.

Pourtant, qu'il s'agisse de géotextile prélevé dans les ouvrages et ayant donc fonctionné, ou bien de géotextile colmaté en laboratoire, des interrogations persistent sur la totale validité des images du colmatage ? Dans le premier cas, les maniements de sol et du textile lors du prélèvement n'ont-ils pas affecté les intéractions sol-fibres, ou amplifié le colmatage ? Dans le second cas, les conditions de simulation retenues au laboratoire, reproduisent-elles bien les conditions réelles d'utilisation des géotextiles dans les ouvrages ?

Dans la suite de ce texte, nous essayerons d'apporter des éléments complémentaires à la compréhension des mécanismes qui peuvent conduire au colmatage partiel ou total des géotextiles. Pour cela, nous nous référerons, d'une part à une série de 30 géotextiles prélevés dans des ouvrages en France, après parfois plus de 12 ans de fonctionnement, et d'autre part à 30 carottages effectués dans les mêmes ouvrages pour obtenir des prélèvements intacts, non perturbés du complexe sol-textile. Des techniques sophistiquées d'inclusion de résine (polyester) ont été mises en oeuvre pour maintenir et figer en position les particules de sol et les fibres, jusqu'au stade ultime de la fabrication de la lame mince et de l'observation au microscope optique. L'étude des intéractions sol-fibres ont été poussées à un niveau submicroscopique sur des coupes ultraminces (0,1 micron d'épaisseur) au microscope électronique à transmission.

1. ECHANTILLONS ET METHODES D'EXAMEN

1.1 Prélèvements des géotextiles

Quelques indications sont fournies par ailleurs (3) sur les ouvrages où les échantillons de géotextiles (1 m x 1 m) ont été prélevés. Nous ne détaillerons dans ce texte que les résultats relatifs au taux de contamination des géotextiles et à leurs caractéristiques de perméabilité transversale résiduelle. Ceux-ci ont été mesurés sur les échantillons de textile encore pollués par certaines couches de sol, et après lavage intensif du géotextile.

1.2 Carottages

Les carottages des échantillons intacts sol-textile ont été effectués à l'aide d'un tube de prélèvement de dimensions suivantes : diamètre intérieur 150 mm, épaisseur 4 mm, longueur 200 mm.

Une des extrémités du tube est taillée en trousse coupante à environ 45°. La paroi interne du tube est préalablement usinée et polie et enduite de graisse. Le tube et son prélèvement sont scellés à l'aide de couvercles rigides et étanches pour éviter toute déssication et remaniement possible du sol.

1.2.1 Fabrication des lames minces

Les techniques de fabrication de lames minces dans des échantillons meubles, comme ceux des géotextiles, sont comparables à celles utilisées par les pétrographes et les minéralogistes, mais il est nécessaire pour les matériaux meubles de passer par une phase de consolidation. Cette fabrication comprend les phases suivantes :

. séchage de l'échantillon en atmosphère sèche à 60°C ;
. imprégnation par capillarité avec un polyester dilué à l'acétone, durée de l'imprégnation environ 48 h ;
. polymérisation et durcissement du polyester, durée 3 semaines à 1 mois ;
. taille des lames minces, suivant la technique mise au point par Guilloré (1980).

Les lames minces obtenues sont épaisses d'environ 20 µm et leurs dimensions sont de 5,8 - 13,5 cm.

Les lames minces ont été fabriquées au Laboratoire de Géologie de l'INA de GRIGNON par M. P. Guilloré.

1.2.2 Fabrication des lames ultra-minces

Ces préparations de 0,1 micron d'épaisseur sont nécessaires pour l'examen au microscope électronique à transmission. Elles ont été réalisées à l'ITF par Mme G. GASTALDI en mettant en oeuvre un ultra-microtome REICHERT équipé d'un couteau de diamant.

2. RESULTATS

2.1 Contamination et Perméabilité résiduelle des Géotextiles prélevés dans les Ouvrages

Les résultats sont consignés dans le Table 1. Le taux de pollution a été calculé après lavage intensif des échantillons. Apparaît également sur le Table un classement en fonction du colmatage observé sur lames minces des complexes sol-textile ; la notation est la suivante : 0 (absent ou nul), 1 (faible), 2 (modéré), 3 (assez important), 4 (important), 5 (intense). A l'analyse des résultats, on constate d'une manière générale :
- une élimination importante du sol par lessivage des éprouvettes. Le taux de pollution résiduel est compris entre 20 et 40 %, qu'il s'agisse de géotextiles non-tissés ou tissés.
- la perméabilité des géotextiles chute jusqu'à 2 puissance de 10, à l'état pollué, mais d'une seulement après lavage. Il existe une assez bonne corrélation entre les mesures de perméabilité et les estimations de colmatage faites à l'examen des lames minces.
D'une manière plus particulière, les échantillons SODOCA présentent un colmatage de base plus important que les échantillons de BIDIM. Pour expliquer ces différences, il faut considérer :

. la contexture du géotextile : les SODOCA prélevés présentent tous des couches alternées de fibres fines et de fibres plus grosses. Ce point semble avoir de l'importance sur le colmatage.

. le sol en place, et notamment sa granulométrie, qui n'apparaît pas sur le Table 1. Pour la bonne compréhension du travail, nous rappellerons essentiellement le taux de particules argileuses dans le sol en place, dont les dimensions sont inférieures à 2 microns :

cas des colmatages importants : NOYALO (45 %), HYERES (-), LE MONTET (28 %), REDON 2 (69 %), ST FLOUR (77 %).

cas des faibles colmatages : REDON 1 (7 %), CAMBRESSOL (7 %), NEUFCHATEL (7 %), GANDELAIN (16 %), MAUREPAS 2 (20 %).
L'influence de la granulométrie du sol support sur le colmatage apparaît clairement.

. la nature du polymère : polyester ou polypropylène ?.. Il semblerait que ce paramètre puisse jouer un rôle (chimique ou électrochimique). Il est prématuré de conclure actuellement sur ce sujet.

2.2 Approche morphologique

Il est difficile ici de fournir le détail des très nombreux examens réalisés en microscopies sur les complexes intacts sol-textile. Les figures suivantes illustrent certains faits précis. Nous synthétiserons dans le texte qui suit, les types de déformation et les principaux types de colmatage notés au cours de ces examens.

2.2.1 Les déformations mécaniques des géotextiles

Les géotextiles étudiés présentes des ruptures partielles et totales, des déformations et applatissements. Seuls les prélèvements du CER-ROUEN montrent des déchirures totales : les tissés sont les plus sensibles notamment au niveau d'entrecroisement des fils. Souvent le déchirement est à associer à la perforation au niveau de graviers à angles vifs. Sous l'effet mécanique du sol en place ou du matériau de recouvrement, le géotextile peut présenter, soit des amincissements, soit des plissements. Dans le cas de NOYALO, une déformation du géotextile en forme d'hernie est visible dûe à une expansion du sol en place, très plastique en l'occurence.

2.2.2 Les colmatages minéraux

Trois types de colmatage minéraux ont été distingués à l'examen des lames minces : le colmatage de base, interne et supérieur.

Le colmatage minéral de base se présente sous forme d'une accumulation le plus fréquemment argilo-limoneuse continuedans laquelle les fibres apparaissent serties. Il est en continuité avec le sol en place, mais avec tri des éléments grossiers. Il est toujours discontinu. Le faible tri constaté témoigne en faveur d'une origine très locale du colmatage à partir du sol en place. Ce tri (élimination de grains de sable et limon grossiers) indique une relative mise en suspension des particules du sol en place. Ceci suggérerait que le colmatage de base s'est produit à la pose du géotextile ou peu après sous forme d'une injection boueuse. Conformément à ce qui a été dit au 2.1, la tourbe et matériaux grossiers ne paraissent pas donner de colmatage de base important. A l'inverse, ce sont les argiles limoneuses qui pénètrent le plus facilement dans les géotextiles. D'autres facteurs peuvent intervenir pour favoriser ce colmatage : état du sol au moment de la pose, facilité de dispersion du sol support (à mettre en relation surtout avec son taux de Na), mode de circulation de l'eau à l'interface sol-géotextile.

Le colmatage minéral interne est soit argileux, soit grossier. Le premier est constitué d'argile et de limon très fins. Ils se forment en premier lieu autour des fibres sous forme d'une pellicule épaisse au plus de quelques dizaines de microns. L'orientation de ces pellicules est moyenne à faible. En un second temps des ponts argileux apparaissent entre les fibres. Ces deux phénomènes sont très bien révélés sur les micrographies électroniques. Aux endroits où les fibres sont très rapprochées les ponts se multiplient et les argiles envahissent progressivement tout l'espace interfibres: formation des amas argileux dans lesquels les fibres sont serties. La présence de couches de fibres très fines et également très rapprochées peut favoriser ce processus (cas des SODOCA étudiés). Au stade plus avancé du

Table 1. Caractéristiques Hydrauliques et Taux de Colmatage des Géotextiles

Echantillons	Taux Pollution (%)	Taux Terre (%)	Perméabilité sous 0,02 MPa m/s 10^{-5}		Indications s/sol en place	Colmatages minéraux			Autres Colmatages
			pollué	lavé		base	interne	Superf.	
SODOCA									
Témoin	-	-	-	200		-	-	-	
1972 NOYALO	15,4	370	0,7	38,6	{argile limoneuse + quartz	4	4	2	ferrugineux et racinaires
1976 HYERES	5,7	373	20,3	53,1	{gravier + argile limoneuse	4	5	2	0
1972 LE MONTET	22,2	112	16,0	68,0	{argile limoneuse + sable	4	1	0	0
1976 MAUREPAS (2)	23,1	53,8	23,6	18,2	limon sableux	2	2	1	quelques racines
" " (3)	13,8	37,4	36,7	28,9	" "	1	1	1	0
" " (4)	43,9	135	8,4	29,8	" "	-	-	-	-
ADITEX									
1976 FOURCHON	10	73,9							
1973 THIERS	9,3	73,1							
BIDIM 300 g/m2									
Témoin	-	-	-	300		-	-	-	
1969 REDON (1)		209	16	24	gravier sableux	0	3-4	ε	0
" " (2)		357	6,1	34,6	{tourbe peu décomposée	2	5	1	0
1971 CARENTAN	9,6	284	5,1	30	{limon cohérent, calcaire	0	2	0	ferrugineux
BIDIM 400 g/m2									
Témoin	-	-	-	300		-	-	-	
1970 ST FLOUR	21	238	31	220	{argile sablo-gravillonnaire	3-4	2	0	0
1970 CHEVERNY	33	29,2	46,7	83,3	sable argileux	0	0	0	ferrugineux
1971 CAMBRESSOL	2,7	208	14,6	374,8	tourbe + argile	0	1	0	gypseux
BIDIM U 34									
Témoin	-	-	-	300		-	-	-	
1977 NEUFCHATEL	11,6	61,3	20,3	26,4	grave laitier	0	1	0	0
1977 GANDELAIN	5,5	188,3	53	133	{couverture limon. + grès schisteux	3-4	2	ε	0
1974 CER-ROUEN (1)	19,3	266	13	35,4	{massif de limon				
" " " (2)	-	575	5,4	52,1	{(70 cm)	0	3	1-2	0
" " " (3)	1,5	413	18,1	71	{reconstitué				
STABILENKA									
1974 CER-ROUEN (4)	20	75	-	-	{	0	3	1	0
" " " (5)	21	56	-	-	{massif de limon	0	2-3	1	0
" " " (6)	22,3	50,9	-	-	{reconstitué	0	3	1	0
" " " (7)	26,3	30,9	-	-	{	0	3	1	0

colmatage interne, les amas précédents se connectent par des ponts argileux (cas de l'échantillon d'HYERES) : la morphologie du colmatage prend alors l'allure typique spongieuse ou alvéolaire. Aucun des colmatages internes observés ne présentent de litage.

Dans le cas des tissés, du fait de la grande proximité des fibres à l'intérieur des fils et des fils entre eux, le stade d'un colmatage total est très vite atteint.

Le degré élevé de triage, dans ces accumulations d'argile du colmatage interne, atteste que ce colmatage se produit à partir de suspensions d'argile. Il n'existe aucun caractère micromorphologique qui permette de reconstituer l'histoire et le développement du colmatage. En outre, ce colmatage est-il susceptible de progresser ? On peut penser que la structure typique du colmatage interne en éponge traduit un état d'équilibre entre les atterrissements et la circulation de l'eau à l'intérieur du géotextile. Ce schéma permet de rendre compte de la forte perméabilité résiduelle de géotextiles jugés colmatés sur le plan morphologique.

Les micrographies électroniques indiquent une sédimentation rapide (faible orientation des particules argileuses) et relativement grossière. Ce gainage des fibres ne semble pas les affecter.

Le second (colmatage grossier) est constitué de grains de limon moyens, plus rarement de sable coïncés entre les fibres. Souvent ce colmatage provient du matériau de recouvrement.

Le colmatage minéral supérieur est rare dans les géotextiles aiguilletés, mais plus fréquent avec les tissés (comblement des espaces interfils). Dans ce colmatage les matériaux proviennent de la matrice argilo-limoneuse du sol de recouvrement: par rapport à celui-ci, ils sont pas ou très peu triés. On peut considérer ce type de colmatage comme le résultat de la chute des poussières ou particules non stabilisées présentes dans le matériau de recouvrement au moment de son épandage ou lors des premiers lessivages.

Conclusions sur le colmatage minéral : Le colmatage minéral des géotextiles n'apparaît jamais total à l'examen sous microscope de sections des complexes intacts sol-textile. Il peut être quasi total dans le cas des tissés. A l'intérieur d'une même carotte, le colmatage minéral présente de grandes variations latérales, et ceci à courtes distances. Cette variabilité diminue évidemment avec l'intensification du colmatage. Dans l'ensemble, les corrélations entre les taux de pollution, la perméabilité résiduelle et les taux de colmatage estimés sur lames minces, sont assez bonnes. Elles deviennent très bonnes avec l'intensification du colmatage, ceci est à relier à la variabilité du colmatage.

Le colmatage minéral peut avoir plusieurs origines. Il peut résulter d'une pénétration en semi-suspension dans le géotextile du sol en place (colmatage minéral de base). Il peut provenir de la sédimentation autour des fibres et entre les fibres d'une suspension argileuse grossière donc renfermant des limons fins (colmatage interne). Une telle suspension argileuse peut d'ailleurs circuler sur des distances importantes dans le géotextile (échelle microscopique). Enfin, le colmatage minéral peut être alimenté par le matériau de recouvrement ; celui-ci peut pénétrer sous forme de grains de limon et de sable fin à l'intérieur du géotextile (colmatage interne grossier) ou rester à la surface supérieure (colmatage supérieur).

On peut suggérer, que les colmatages de base, interne grossier, supérieur, se forment lors de la pose du géotextile et de son recouvrement. Le colmatage interne argileux se constituerait plus progressivement. Néanmoins, même dans les géotextiles les plus colmatés, il semble qu'un équilibre s'établisse entre les atterrissements argileux et la circulation de l'eau.

Au terme de ce travail, peut-on recommander des géotextiles résistant au colmatage ? Certes non, mais des règles peuvent se dégager pour éviter des colmatages trop importants, par exemple : éviter la mise en oeuvre de fibres trop fines qui ont tendance à s'accumuler en grappes, et donc à favoriser le colmatage ; éviter de fabriquer des tissés également avec des fibres trop fines et avec des comptes trop serrés (faible porosité). Dans le même ordre d'idées est-il possible de prévoir a priori les risques de colmatage minéral ? L'expérience acquise, par l'étude de drains agricoles (SOLE, 1979, TIBLE, FEDOROFF, 1981) et par le présent travail, permet de classer dans un ordre décroissant les sols risquant de colmater les géotextiles :

. les sols à capacité d'échange à forte saturation en Na^{+} et dans une moindre mesure en Mg^{++}. On doit considérer qu'il y a risque à partir d'un milli équivalent de Na^{+} pour un taux de saturation en Na de quelques pourcents.

. les sols non sodiques, mais présentant une faible stabilité structurale.

. les sols argilo-limoneux, en particulier ceux contenant une forte proportion de fragments de micas.

Il est certain que les modes de circulation de l'eau, dans le sol en place, dans le géotextile et le matériau de recouvrement, jouent un rôle très important, mais il n'est pas encore possible de tirer des conclusions sur le type de circulation hydrique à partir des observations morphologiques du colmatage.

2.2.3 Le colmatage ferrique

Comme dans les drains agricoles, il se développe dans les géotextiles des ferruginisations. On les observe généralement dans le sol support parallèlement aux géotextiles, à quelques millimètres en dessous ou au contact. Quand elles se produisent à l'intérieur du géotextiles, elles prennent la forme d'amas granulaire brun ou rouge d'une dizaine de microns, ancré sur des fibres. Dans le cas où des racines sont présentes dans le géotextile, elles sont ferruginisées de préférence. Tous les colmatages ferriques internes notés sur des géotextiles sont peu développés et en tous cas ne semblent pas nuire au bon fonctionnement du géotextile. Les causes d'un tel colmatage sont mal connues (bactéries...). Il faut noter l'absence d'intéractions physico-chimiques entre ces amas ferrugineux et le polymère fibreux.

2.2.4 Le colmatage gypseux

Ce colmatage n'a été observé que sur un seul géotextile, celui de la RN 89 Cambressol. Il se développe entre les fibres rapprochées sous forme de petits cristaux d'environ 20 - 50 microns.

2.2.5 Le colmatage racinaire

Les racines tendent à se développer préférentiellement à proximité immédiate et même à l'intérieur du géotextile. Elles trouvent à cet endroit, un milieu plus accessible et plus facilement pénétrable.

3. CONCLUSIONS

Les géotextiles prélevés dans les ouvrages, même pollués par les sols en contact, conservent une imperméabilité importante par rapport à celle des sols. Après lavage intensif, les géotextiles débarrassés de la plus grande partie de leur contamination, accusent une perte d'une puissance de 10 sur leur perméabilité comparativement à celle d'un produit neuf.

A l'examen morphologique des complexes intacts sol-textile, les géotextiles peuvent présenter un colmatage intense. Ce colmatage est bien sûr fonction de la granulométrie et de l'état hydrique du sol en place (plus important avec un sol argileux riche en particules inférieures à 2 microns facilement mises en suspension), un peu du sol de recouvrement et des conditions de mise en oeuvre.

Trois types de colmatage ont été répertoriés : de base, interne et supérieur. Un processus évolutionnel du colmatage interne par des argiles a été proposé : enrobage des fibres puis pontages interfibres, constitution d'amas et enfin, pontages entre amas pour la formation d'une structure alvéolaire spongieuse. Ce stade ultime du colmatage interne semble correspondre à un état d'équilibre entre les atterrissements et la circulation d'eau dans le géotextile : ceci explique la perméabilité résiduelle importante enregistrée pour des géotextiles jugés intensément colmatés.

Aucune intéraction chimique ne semble avoir été mise en évidence entre fibres et particules de colmatage. Les charges électriques présentes dans les argiles et à la surface des fibres, jouent peut-être un rôle qui permettrait d'expliquer certaines différences observées entre fibres polypropylène et polyester, mais de toute manière, ce rôle reste de faible importance sur le colmatage.

Le plus spectaculaire à rapporter dans cette conclusion est que, relativement au sol en place et au matériau de recouvrement, le géotextile constitue encore, même après 12 ans ou plus de fonctionnement, une évidente et réelle discontinuité de perméabilité; il reste aussi un milieu ouvert et oxygéné où les eaux circulent, ce qui permet d'expliquer la fréquence des ferruginisations que l'on observe dans ou à proximité du géotextile, et la propension des racines à coloniser le géotextile.

Fig. 7 & 8 : Micrographies de lames minces pour échantillons d'HYERES (drain) et REDON (anticontaminant). Visualisation nette des positions relatives géotextiles-particules de sol et du colmatage interne alvéolaire du géotextile.

(cf. page 6) →→→

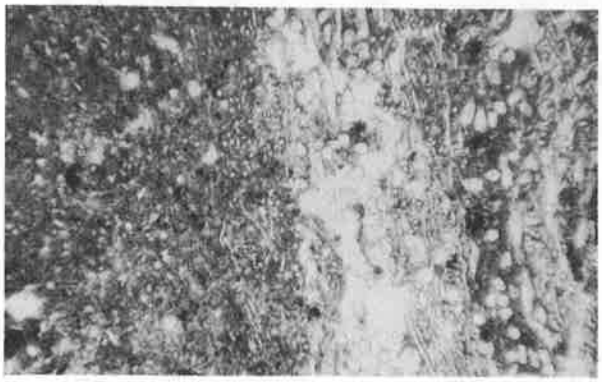


Fig. 1 Détail d'une micrographie lames minces, au niveau du colmatage de base et interne. Noter l'influence des petites fibres.

Fig. 2 Colmatage interne d'un géotextile en "éponge".

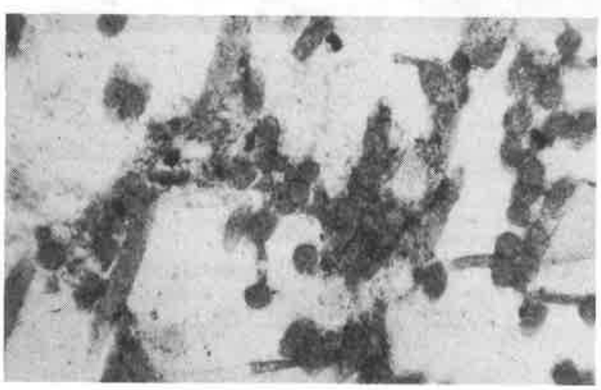


Fig. 3 Détail agrandi d'un colmatage interne : ponts interfibres et formation d'amas.

Fig. 4 Micrographie Electronique : détail au niveau d'un amas autour de 3 fibres.

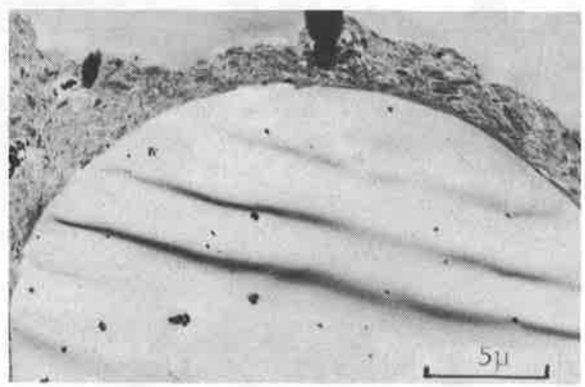


Fig. 5 Micrographie Electronique : détail au niveau de l'enrobage de sol sur la fibre.

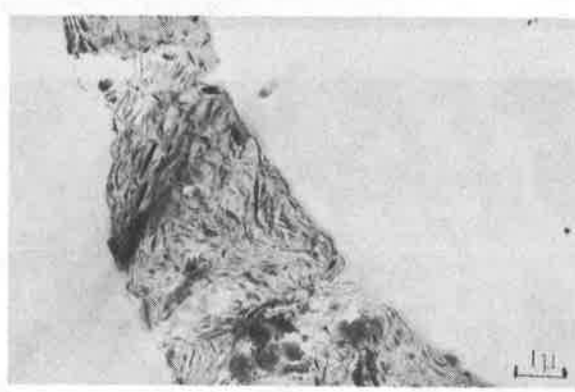
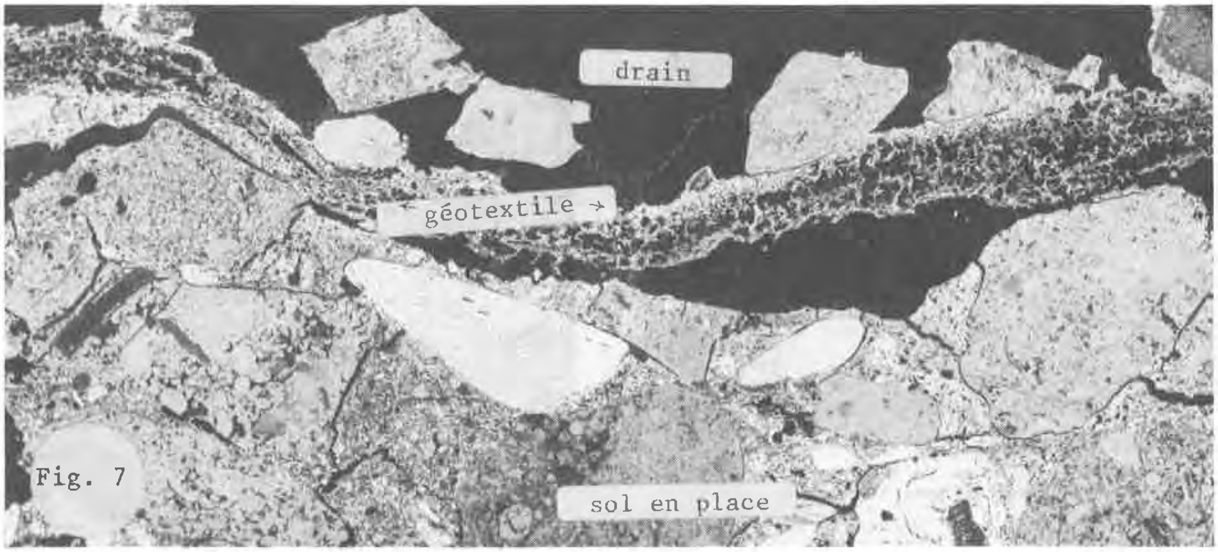
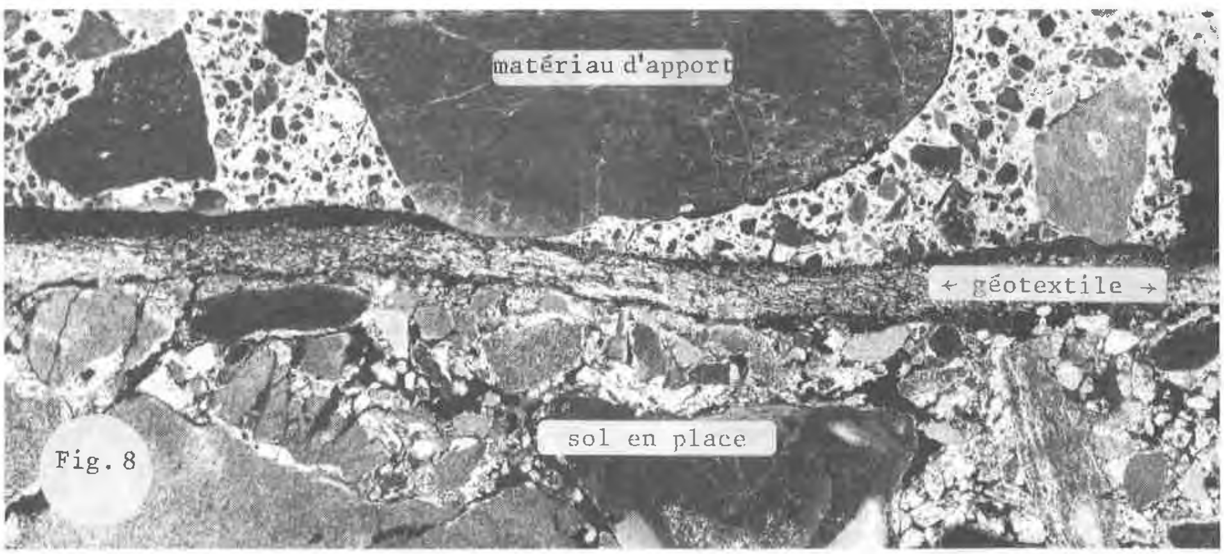


Fig. 6 Micrographie Electronique : détail montrant la faible orientation de l'argile déposée sur la fibre.





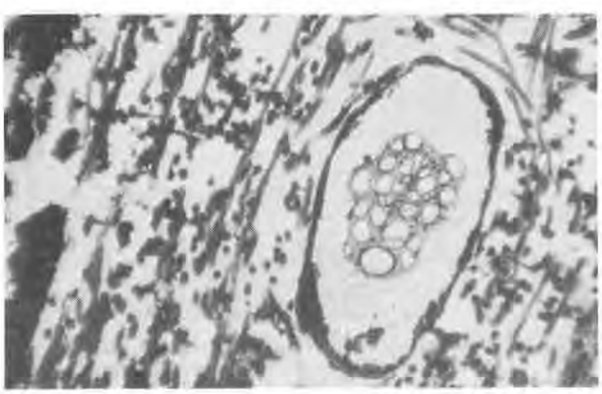


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Fig. 9 Micrographie lames minces (CHEVERNY). Détail au niveau d'une ferruginisation en bordure du géotextile.

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Dimensioning the Filtration Properties of Geotextiles Considering Long-Term Conditions

Evaluation des propriétés filtrantes des géotextiles en tenant compte des conditions à long terme

For dimensioning the mechanical and filtration properties of geotextiles there have till now been no sufficient proposals which take into consideration the interaction of geotextile and soil. Within the scope of a special research program, several geotextiles (woven and non-woven) were dug out of existing revetments on seadikes and inland waterways to test their mechanical and filtration properties. This paper gives the results of testing the filtration properties and describes a method for dimensioning the filtration properties of geotextiles. Developed filtration rules and diagrams to estimate the reduction of permeability to water in virgin fabrics provide a complete formula for solutions which takes into consideration the interaction of geotextile and soil (blocking and clogging). There is also a description of the test procedures for checking the opening size and permeability to water of geotextiles as basic data for dimensioning filtration properties.

Jusqu'ici il n'existait encore aucune proposition ni pour la mesure mecanique ni pour la mesure des propriêtês filtrantes, qui tienne suffisamment compte de la correlation entre gêotextile et sol. Dans le cadre d'un programme d'études spécial une série de géotextiles (tissés et non-tissés) ont été déterrée de déjà existants revêtements de digues de mer et de voies navigables, pour contrôler la solidité et les propriétés filtrantes. Ce contribution décrit les résultats de l'étude des propriétés filtrantes et présente une méthode pour la mésure des propriétés filtrantes de géotextiles. Avec les règles de filtrage et les diagrammes permettant de déterminer la diminution de perméabilité de géotextiles neufs sortant d'usine, il donne une esquisse complete de solution, qui tienne compte de l'influence réciproque de géotextile et sol (blocking/clogging). Les méthodes de contrôle des valeurs base du géotextile (largeur d'ouverture et perméabilité) y sont également décrites.

INTRODUCTION

In a given application, a geotextile can perform several different functions. But in many cases one of the functions is to act as a filter to prevent the wash-out or passing of fine soil particles in connexion with a sufficient permeability to water.

For describing the mechanical and filtration properties of geotextiles there exist a lot of laboratory tests but most of them only allow to compare the properties of different geotextiles and are not useful for dimensioning a geotextile at a given application. Not enough valid informations on fabric properties are gathered in order to predict accurately the behaviour of such materials in field conditions. This underlines the necessity of field investigations or large scale modelling to collect data and informations on the interaction of geotextile and soil.

In 1979 and 1980 the Franzius-Institut for Hydraulic Research and Coastal Engineering of the University of Hannover, W.-Germany, has carried out extensive field investigations on long-term behaviour of geotextiles in coastal structures. This investigations were completed by field studies of the Federal Institute of Waterways Engineering (BAW) at river and canal revetments.

This paper will only deal with the results of filtration properties of geotextiles, other properties such as mechanical and chemical will thus be discarded. Results on the long-term resistance of geotextiles in coastal structures based on the mentioned investigations are given in separate puplications in 1980 by Georg Heerten (2).

1. FIELD STUDIES ON GEOTEXTILES

1.1 Research Program

The research program carried out at the Franzius-Institut for Hydraulic Research and Coastal Engineering of the University of Hannover was devided in five parts:

a) Evaluating the experiences in the use of geotextiles in coastal and hydraulic engineering sending extensive questionnaires to the engineering authorities

b) Developing test methods to estimate the filtration properties of virgin fabrics of dug out geotextiles as well

c) Digging out geotextiles of coastal structures being in function for many years

d) Investigating the mechanical and filtration properties of the dug out geotextiles and of the virgin fabrics

e) Developing a dimensioning method to fulfil and calculate the retention of solid particles and the permeability to water considering the interaction of geotextile and soil

The most important part of the research program was the investigation of the properties of the fabrics dug out. For samples dug out of revetments the following individual investigations were carried out:

a) Condition of the revetment (profile changing)

b) Condition of the geotextile

c) Testing the tensile strength and elongation

d) Testing the filtration properties

e) Testing the fabric weight and soil content

f) Grain-size analysis and permeability test of the subsoil

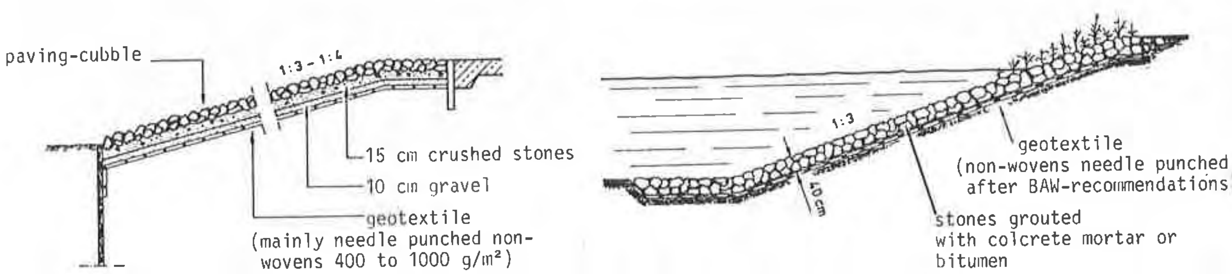
For fabric samples of other coastal structures (sand bags, sand filled tubes) the research program was reduced to the individual investigations of fabric condition, tensile strength, opening size of the fabric and grain-size analysis of packed soil. In relation to the long-term filtration properties in this paper only the results of testing d) to f) are discussed and presented.

In the meantime it is possible to consider also the first results of the investigation program of the Federal Institute of Waterways Engineering (BAW) digging out some non-woven geotextiles at revetments of canals and rivers. By order of the Federal Institute of Waterways Engineering the Franzius-Institut has investigated the filtration properties of these fabrics too.

1.2 Investigated Geotextiles and Structures

At different locations at the German Northsea coast and at canals and rivers in the northern part of Germany the geotextile samples were taken. 16 samples (12 non-woven, 4 woven fabrics) were dug out of revetments of seadikes, 6 samples of non-woven fabrics are originated from inland waterways. The general layout of the revetment structures is shown in Fig. 1.

At the seadike revetment the subsoil is covered by the geotextile and the geotextile itself is covered by a 10 cm gravel layer, 15 cm crushed stone layer and a heavy paving-cobble. In contrast to this the revetments of the inland waterways are composed of the geotextile and a heavy rip-rap layer grouted with concrete mortar or bitumen. Fig. 2 gives an impression of the sampling operation and in Fig. 3 a non-woven geotextile is shown after removing the cover layers of the revetment.



Seadike Revetment

Inland Waterway Revetment

Fig. 1 Layout of the seadike revetments and revetments of inland waterways

Fig. 2 Digging out a fabric at a seadike revetment at the Northfrisian coast

Fig. 3 Non-woven needle punched fabric after removing the cover layers

1.3 Some Special Results

The first important result which could be noticed already at the site was the extensive incorporation of soil in the non-woven fabrics. Caused by the soil incorporation the mass of the samples came up to 10 kg/m². In table 1 the characterizing data of the non-woven samples and the estimated soil content are given. This high clogging rates illustrated in Fig. 4 should be used as a scale for laboratory test, testing non-woven fabrics with soil.

Fig. 4 Non-woven needle punched fabric filled up with soil particles

In most cases the incorporation of soil particles in comparatively short laboratory test will be only a small fraction of the observed amount of the field studies. Based on the actual knowledge and experience of the author there exist only one test method giving results similar to natural clogging conditions: the turbulence test of the Federal Institut of Waterways Engineering (BAW). The BAW-turbulence test is a special developed test for investigating the filtration properties of thick multilayer needle punched non-woven fabrics being used in revetments of waterways on fine soils in Germany, H.J. List (4).

The second important result is the observed difference between the thickness of the dug out non-woven fabrics and the thickness of the fabrics under given revetment load conditions (10 kPa) after washing out most of the incorporated soil (in average 90% of the incorporated soil could be removed). Table 2 gives for example the thickness of some dug out samples at a measuring load of 2 kPa and the thickness after removing the soil and bringing up the revetment load of 10 kPa.

Table 2 Difference in thickness of dug out fabrics and fabrics after washing

Sample No.	Thickness at 2 kPa containing soil mm	Thickness at 10 kPa after washing mm
13	7,1	5,7
14	7,9	6,4
15	4,2	2,9
16	5,8	3,6

Table 1 Description and technical data of investigated non-woven geotextiles and estimated fabric soil content

No	description in brief	year of installation	fabric mass g/m²	thickness at 2 k Pa mm	soil content g/m²
1 2 3	combination of non-woven fibre fabric and woven-scrim; coarse PA 6 fibres 350 dtex; needle punched and chemical bonded	1970 1970 1970	827 1492 1162	7,5 8,0 8,0	8898 8361 9280
4	non-woven fibre fabric needle punched and chemical bonded; PES fibres ~3-6 dtex	1971	487	2,7	1187
5 6	combination of a fine (~6 dtex PES) and coarse (~350 dtex PA) non-woven layer and a woven scrim; needle punched	1974 1974	847 903	6,1 6,6	3149 4123
7 8	combination of non-woven fibre fabric (PES ~5 dtex) and woven scrim; needle punched and chemical bonded	1974 1974	354 404	2,2 2,2	956 679
9	non-woven spun fabric; ~6 dtex PES; needle punched	1974	351	2,7	1348
10	combination of non-woven fibre fabric (PA 6 6-17 dtex) and woven scrim; needle punched and chemical bonded	1974	422	3,3	1358
11	non-woven fibre fabric; 3-6 dtex PES; needle punched	1974	413	3,4	1283
12	non-woven fibre fabric; 3-6 dtex PES; needle punched; combined with a scrim	1974	406	2,9	2106
13 14 15	combination of non-woven fibre fabric and woven scrim; coarse PA 6 fibres 350 dtex; needle punched and chemical bonded	1971 1970 1972	1071 1327 872	7,1 7,9 4,2	3686 4867 2059
16	non-woven fibre fabric 3-6 dtex PES, needle punched	1977	856	5,8	2627

The observed variation of the thickness lead to the assumption that the thickness of needle punched non-woven fabrics didn't decrease during installation as given by laboratory test for testing the compressibility or the permeability under compression. In these tests the geotextile is symmetrically compressed by e.g. horizontal plates without contact to soil particles. Under field conditions and during installation the decrease of the thickness of non-woven needle punched fabrics evidently is influenced by the given interaction of geotextile and soil and the construction work itself. This observed behaviour of needle punched non-woven geotextiles may improve the conditions of permeability and friction.

In Fig. 5 some data of the virgin non-woven fabrics (porosity n, permeability k_n) and of the dug out fabrics (pore space clogged by soil, remaining porosity n', remaining permeability k_n') are given.

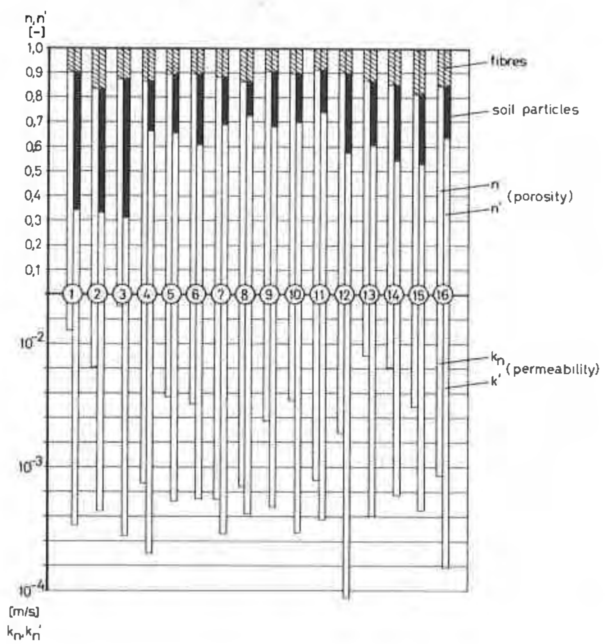


Fig. 5 Clogging of voids volume and decrease of permeability of non-woven needle punched fabrics

A significant decrease of permeability of the needle punched non-woven fabrics can be seen, but compared with the estimated permeability of the soil (permeability coefficient for location 1 to 12 $k \sim 1{,}0$ to $5{,}0 \cdot 10^{-5}$ m/s) the permeability of the clogged fabrics is 5 to 12 times higher as the soil permeability (1). The remaining porosity of n' = 0,32 to 0,74 guarantees a sufficient long-term permeability.

In contrast to these results for most of the woven fabrics investigated in this program a lower permeability coefficient as given by the soil was estimated. The relation of the permeability of the woven geotextiles and the permeability of the soils was in the range of 0,16 to 1,8.

These results underline the advantageous filtration properties of non-woven needle punched geotextiles.

2. FILTRATION PROPERTIES OF GEOTEXTILES

2.1 Dimensioning

A new dimensioning method to select a fabric allows to fulfil and calculate the requirements of sand tightness as well as the requirements of the hydraulic permeability of geotextiles. Fig. 6 shows a flow diagram explaining the procedure to select a fabric according to the characteristics of the subsoil and the construction. in the first step we have to estimate the effective opening size O_{90}. O_{90} required is given by filtration rules as a function of the particle distribution curve of the soil and the load conditions.

In the second step the hydraulic conditions have to be controlled. To prevent over-pressures in a revetment-construction the permeability of the filter fabric has to be higher than the permeability of the subsoil. Special investigations were carried out to consider the decrease of the permeability of woven fabrics by blocking and non-woven fabrics by clogging. Now, as a result of the investigations, it is possible to estimate a permeability-reduction factor as a function of fabric data and soil characteristics. Only für non-woven fabrics, when the effective opening size is small in relation to the diameter of soil particles, an additional restriction is given by $O_{90} < 0.5 \cdot d_{10}$ leading to a η_v value $\eta_v = 1.0$. The permeability of the fabric is sufficient when

$$\eta_{V/G} \cdot k_n \geq k$$

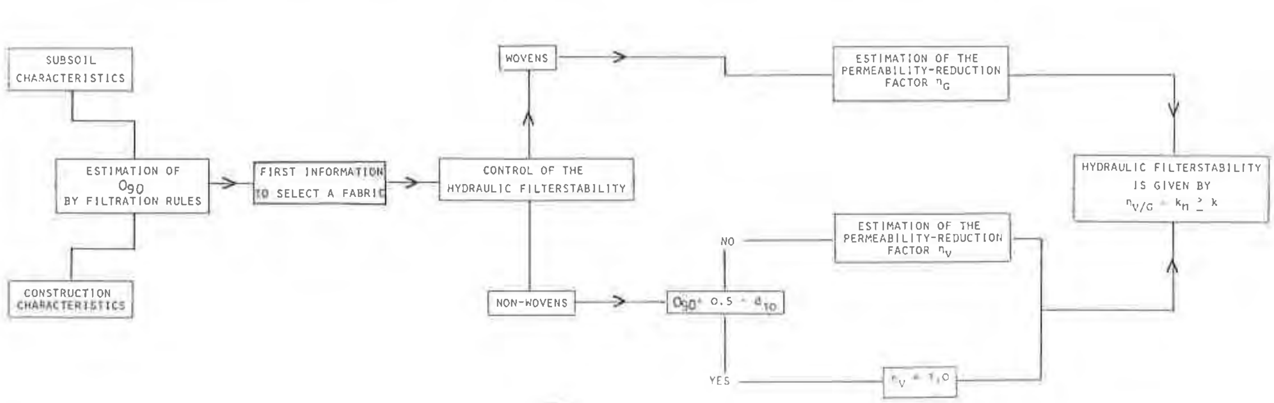


Fig. 6 Flow diagram to check the filtration properties of geotextiles

The filtration rules to fulfil the sand-tightness are determined as follows:

a) non cohesive soils

static load conditions: $C_u \geq 5$ $O_{90} < 10 \cdot d_{50}$ and $O_{90} \leq d_{90}$

static load conditions: $C_u < 5$ $O_{90} < 2.5 \cdot d_{50}$ and $O_{90} \leq d_{90}$

dynamic load conditions: $O_{90} < d_{50}$

b) cohesive soils and all load conditions:

$O_{90} < 10 \cdot d_{50}$ and

$O_{90} \leq d_{90}$ and $O_{90} \leq 0,1$ mm

Static load conditions are given by laminar flow including the change of flow direction. Dynamic load conditions are given by high turbulent flow, wave attack or pumping phenomenon.

Fig. 7 is showing the diagram to estimate η_G. η_G is given as a function of the permeability of the fabric k_n and the soil parameter d_{10}. This diagram considers the knowledge that the permeability of woven fabrics in interaction with the soil is influenced by "blocking". The soil particles are blocking the fabric openings. Blocking occurs immediately after installation with only small temporary variations.

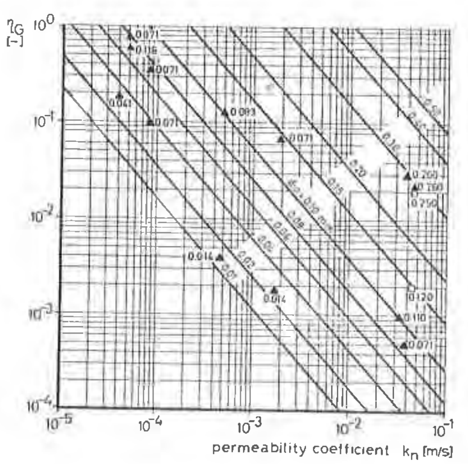


Fig. 7 Diagram to estimate the permeability reduction factor η_G for woven fabrics

In contrast to woven fabrics the permeability of non-woven fabrics is influenced by several parameters describing the "clogging"-behaviour:

- the porosity n given as the ratio between volume of voids and volume of solid elements of a geotextile (needle punched fabrics n = 0,8 to 0,9; $n = 1 - \frac{\mu}{\rho_f \cdot T_g}$)
- the thickness T_g of a non-woven geotextile. $n \cdot T_g$ is giving the total volume which could be filled up by soil particles by clogging.
- the opening size O_{90} of the fabric. O_{90} is including informations an fibre finess and the fabric structure
- the soil parameters particle size d_n and uniformity coefficient C_u and the load conditions

The results of the investigations on needle punched non-wovens showed that it is impossible to fill up the total voids volume by clogging (Fig.5). It can be seen that the interaction of fibres and soil is forming a stable filtration layer with a higher permeability as the soil permeability itself. Based on the collected datas in Fig. 8 a diagram is given to determine the permeability reduction factor η_V for non-woven fabrics as a function of the fabric parameters k_n, n, T_g and O_{90}. If the thickness T_g at 2 kPa and the permeability k_n at the given structure load conditions are considered, for practical use a sufficient safety is given.

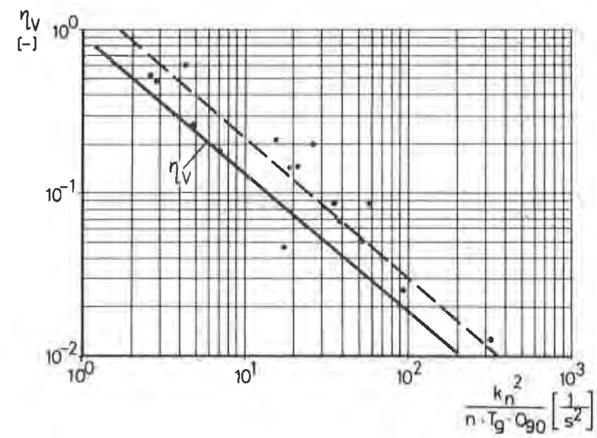


Fig. 8 Diagram to estimate the permeability reduction factor η_V for non-woven fabrics

In Fig. 9 the range of the particle distribution curves of the investigated soils are given. For similar soils and also coarser soils the Fig. 8 diagram may be used for estimating the η_V value. For finer soils additional investigations are necessary. It was surprising that there is no correlation between η_V and the soil parameters although the soil parameters are variing in a wide range (d_{10} = 0,003 to 0,1; d_{50} = 0,09 to 0,3; C_u = 1,5 to 60).

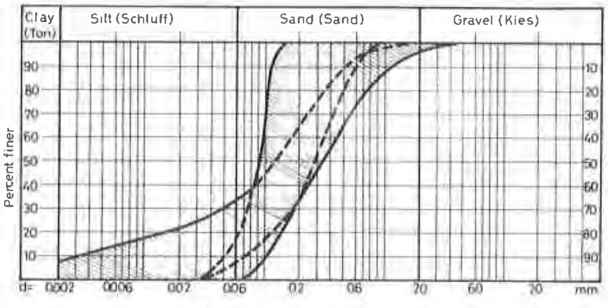


Fig. 9 Range of particle distribution curves at the sampling locations

2.2 Basic Test Methods

The investigation program also was including the development of test methods for testing the filtration properties of aged and virgin fabrics. For dimensioning the filtration properties of geotextiles only two tests have to be carried out giving the basic datas:

a) Testing the effective opening size

b) Testing the permeability as a function of superimposed load

The effective opening size of the geotextile is determined by wet sieving with a defind testing sand, composed of quartz particles. The grain-size distribution of the retained material and of the material passing will be determined leading to the effective opening size by a fixed evaluation method. Fig. 10 is showing the testing apparatus for the wet sieving operation. More details are given by Georg Heerten (1, 2).

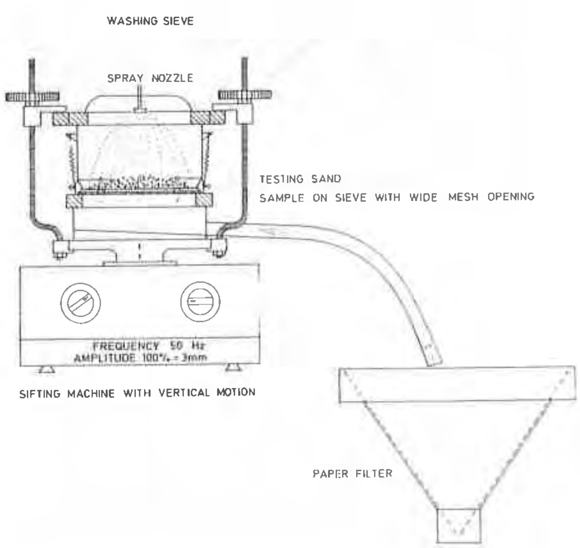


Fig. 10 Testing equipment for the estimation of the opening size of geotextiles

The permeability vertical to the plane k_n of a geotextile is determined in a permeability test with constant hydrostatic head generated by two overflow reservoirs.

Fig. 11 shows the test lay out. In the permeability cell a sample of several layers of the fabric is placed and after measuring the flow, the difference in piezometric level, the water temperature and the settlement of the sample a DARCY-coefficient can be determined. Repeating this procedure for various superimposed loads the permeability can be given as a function of load conditions.

Since 1978 these test methods are in a successful use at the Franzius-Institut for Hydraulic Research and Coastal Engineering of the University of Hannover, W.-Germany. In contrast to other known methods for determining the opening size the Franzius-Institut method is giving reproducable and trustworthy results.

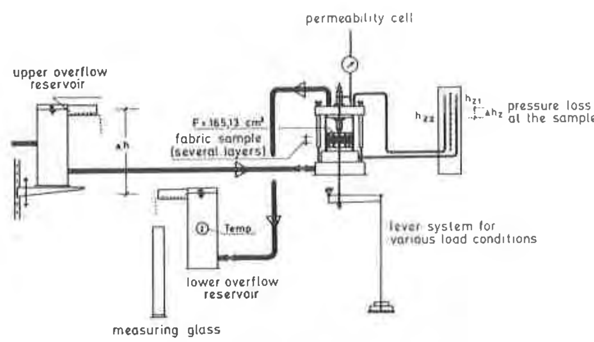


Fig. 11 Testing equipment for permeability test of geotextiles (normal to the plane)

CONCLUSION

Based on the research program (questionnaires and field investigations) especially for the application of fabrics in seadike revetments relating to the layout given in Fig. 1 some references to the mechanical properties of fabrics have been given (1, 3), requireing a minimum mass of non-woven of 400 g/m² and wovens of 225 g/m². In the meantime within a few days two severe storm floods attacked the Northfrisian coast in november 1981. These stormfloods clearly showed the safety and advantage of using heavy (~ 1000 g/m²) non-woven needle punched fabrics built up with fine and coarse fibres in seadike revetments.

These experience also underline the necessity of collecting field data for geotextile application and the author wants to request the engineers and scientists being involved on geotextile investigation to dig out fabrics, to control their conditions and to take these results as a guidepost for laboratory tests and recommendations for the application of geotextiles.

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On Hydric Properties of Geotextiles

Sur les propriétés hydriques des géotextiles

In Romania the first unwoven geotextiles have been manufactured in 1974 mainly from waste synthetic materials. Paper presents some characteristics of most used types as well as a method to determine pore mean size based on water retaining properties. This procedure was verified by microscopic tests and was found more realistic than sand sieving. For stuying normal and in-plane permeability an original oedopermeameter was developed, allowing measurements with geotextiles submitted to various pressures, thus modelling actual conditions in the ground and inside earthworks. The device is provided also with a water desaeration system which improves the quality of tests. The contamination of geotextiles by soil particles was studied both for low and high gradients ; the results proved the advantages of geotextiles as compared with classical inverted filters also from this viewpoint.

Les premiers géotextiles non-tissés ont été fabriqués en Roumanie en 1974 surtout à partir de déchêts. On présente les caractéristiques des types les plus utilisés ainsi qu'une méthode pour calculer les dimensions des pores basée sur les caractéristiques de retention de l'eau Ce procédé, verifié par essais au microscope, s'est avéré plus réaliste que le tamisage du sable normalisé. La perméabilité transversale et dans le plan a été étudiée avec un oedoperméamètre original, qui permet des mesures aux géotextiles soumis à différentes pressions, modélant ainsi les conditions réelles dans l'ouvrages. Le dispositif est doué d'un système de désaération de l'eau qui améliore la qualité des résultats. La contamination des géotextiles a été étudiée pour différents gradients hydrauliques ; les résultats ont confirmé les avantages des géotextiles envers les filtres classiques aussi de ce point de vue.

1 INTRODUCTION

In the field of foundations and earthworks the occurrence of geotextiles was much expected because, unlike granular materials, besides filtration and drainage capacity, they also possess tensile strength and separation ability /1/.

These properties, as well as advantages resulting from material and time savings, efficience and safety increasing, diminishing of earthwork mass, and simplifying of quality inspection, could explain the fact that practical application of geotextiles overpassed the studies dedicated to their properties and behaviour.

Thus in Romania the first unwoven geotextiles have been produced in the year 1974 from abnormal synthetic threads recovered from chemical and textile factories. There were the following materials :

NETESIN, made of polypropylenic, polyesteric, and polynitrylacrylic fibres, obtained by defibration and consolidated by stitching with a strong thread ;

TERASIN, made of the same fibres as NETESIN, but consolidated by interstitching and gluing with a synthetic resin named ROMACRIL L.N.1 /2/.

Later on geotextiles were produced also from new - manufactured long propylene fibres consolidated by wo - ving, as for example MADRIL /2/.

In order to design a structure containing geotextiles it is necessary to know the hydric and mechanical properties of these materials /3/. In this paper the research works concerning the hydric properties of some Romanian geotextiles are described.

2 SUCTION AND PORE SIZE DISTRIBUTION OF GEOTEXTILES

Examination by binocular microscope showed that Romanian geotextiles consist of fibres of about 20 to 30 microns, larger voids existing in-between ; when the material is saturated with water, some occluded air bubbles also remain in the voids (Fig.1). Due to this fact the synthetic geotextiles show capillary phenomena, whereas suction forces are small and correspond to a water head of several centimetres /4,5/.

For knowing the pore size distribution, in contrast with current procedures based on sieving normalized sand or glass fractions, authors resorted to suction-moisture content curves by which geotextiles are characterized as capillary-porous systems. It is known that suction represents the deficit of water pressure in the voids of a porous body as against atmospheric and is usually expressed as cm water head (h) or by the sorptional index pF = lg h /6/. The water retention curves were determined by u - sing suction plate or pressure membrane devices in the

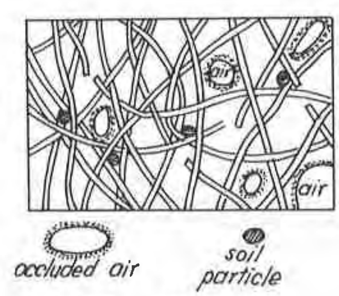

Fig.1. NETESIN extracted from an existing work - seen at the binocular microscope.

range of pF = 0 to pF = 4.8 (0 to 6 MPa) (Fig.2).

The existence of a nearly-horizontal sector of suction moisture content curves in the range of low suction (pF = = 1 to pF = 2), where moisture content exceeds 10 to 15 %

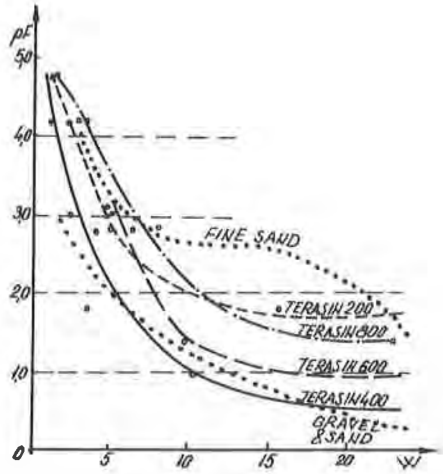

Fig.2. Suction - moisture content curves for Romanian geotextiles.

proves that most of water is located in relatively small pores (0.03 to 0.3 mm). Studies proved that natural fibres display an obviously higher retention capacity than synthetic fibres, therefore the latter ones behave like coarse granular soils (sand and gravel) which are used to inverted filters.

Synthetic geotextiles, like granular mineral materials, retain water especially due to pore capillarity, but in contrast with coarse soils, the saturation moisture content (for pF = 0) of geotextiles is extremely large , reaching as much as 1300 % forTERASIN.

Starting from the relation between the diameter of pores d_p (cm) and the suction h (cm) needed to empty them out /6/:

$$h = \frac{0.3}{d_p} \qquad (1)$$

it results that the retention curve represents in fact the distribution of pore dimensions and furnishes a good characterization of the pore system of the material. Any change in this system would induce a change also in the water retention curve.

If retained water is expressed in terms of degree of saturation S_r (%), the retention curve may be represented on a semi-logarithmic diagram like the particle size distribution (Fig.3).

In order to characterize in the same time also the soils in contact with geotextiles all these data were plotted in the third quadrant of the "print" diagram on which the soils used in experiments have been represented. The "print" is a simple geometrical figure combining particle size distribution and plasticity diagrams and used to soil identification /7/. By representing both materials on the same chart all basic information is available on a single picture. As it can be seen from Fig.3, Romanian geotextiles made from synthetic fibre wastes have similar pore size as those produced abroad - i.e. most of pores are larger than 0.03 mm. It must be also observed that the pore size distribution determined by sieving is not realistic, as it shows much finer dimensions than actual ones, because geotextile fibres hinder - by friction or adhesion - the passing through of sieved particles.

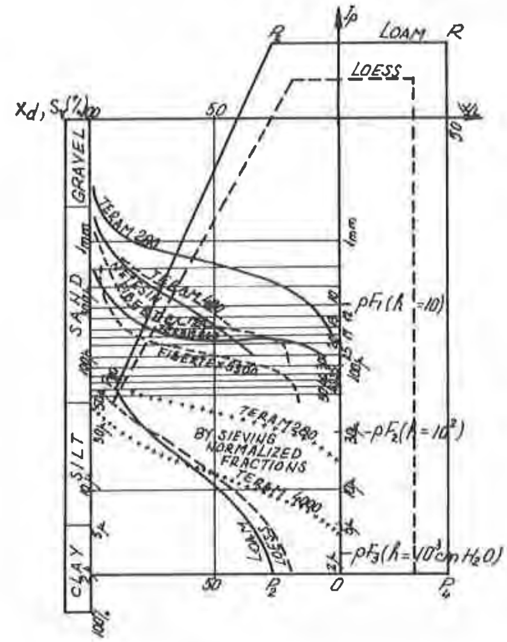

Fig.3. Identification charts ("prints") of examined soils and geotextiles.

3 NORMAL AND IN-PLANE PERMEABILITY OF GEOTEXTILES

In order to measure the permeability of geotextiles an oedopermeameter has been realized in two variants (Fig. 4). It has an essential advantage,i.e. the self-deaeration of circulating water, thus avoiding the formation of occluded air bubbles and the spreading of measurement results. The removing of air was obtained by the set-up

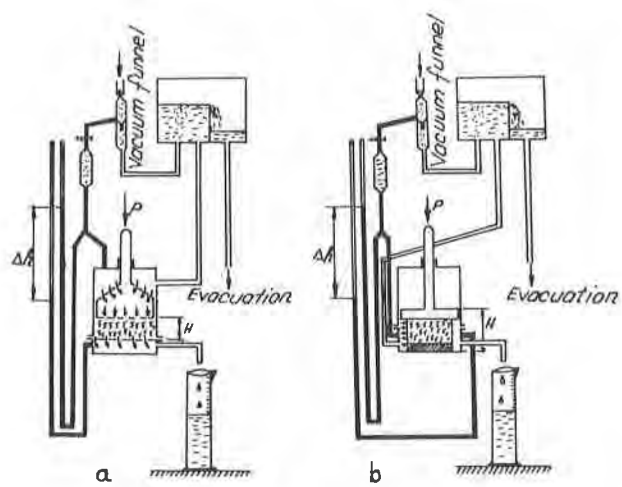

Fig.4. Schetch of oedopermeameter.
a) Measuring normal permeability.
b) Measuring in-plane permeability.

sketched in Fig.4 ; the tap water is passed through a vacuum funnel where air bubbles are released and eliminated through the constant-level jar. In the same time the lateral connection of the funnel with the upper side of the permeameter allows the accumulated air to be evacuated. After air removing repeated measurements gave almost identical results.

When oedopermeameter was used for normal permeability determinations (Fig.4a) porous stones were replaced by perforated plates, the ways to get out water from the apparatus were enlarged and the upper piston was perforated. By these measures the hydraulic losses through the empty device were lowered down to only 1 mm water head, which is negligible for measurements. The oedopermeameter allows pressures in the range of 0 to 1 MPa to be applied.

When oedopermeameter was used for in-plane permeability determinations (Fig.4b) the orifices for water entrance and exit were provided at the lower part of the box, the piston had no perforations and was tightened by gaskets on lateral box walls.

The geotextile discs of 7-cm diameter were put together in bundles of 1.5 to 2 cm thickness and saturated with water, forming a sample, which was introduced in the oedopermeameter box and submitted to an initial pressure of 2 kPa. After water de-aeration the pressure was increased and the height H (cm) of sample was read by dial gauge. If the head loss (read by piezometers) is Δh, the time required to pass 1000 cm^3 of water through the sample is t (seconds), and the transversal area of sample is A (cm^2), the coefficient of permeability is given by :

$$k = \frac{1000\ H}{A \cdot \Delta h \cdot t} \quad (cm/s) \qquad (2)$$

In order to express the density state of geotextile in the same manner as for soils /7/ the volume V_{100} was used, corresponding to 100 g dry geotextile ; by this way the need to establish fibre density - which is doubtful especially for waste materials - is avoided. Also, knowing that permeability is proportional with the square of pore mean diameter, the volume V_{100} was plotted against $\sqrt{k}$ instead of k, thus obtaining a linear dependence (Fig.5 left side) under the form :

$$V_{100} = a + b\ \sqrt{k} \qquad (3)$$

By the same way the relation between pressure (higher than 20 kPa) and V_{100} was linearized (Fig.5 right side) under the form :

$$V_{100} = V_o - C_c\ \lg p \qquad (4)$$

where C_c is the compression index of soil.

The dependence of permeability on applied pressure may be thus expressed as follows :

$$a + b\ \sqrt{k} = V_o - C_c\ \lg p$$

or

$$\sqrt{k} = \frac{V_o - a - C_c\ \lg p}{b} \qquad (5)$$

where all coefficients a, b, C_c and V_o can be experimentally determined by oedopermeameter tests. This relation is needed when earthworks including geotextiles are planned.

4 CONTAMINATION OF GEOTEXTILES

A very significant problem occurring when granular inverted filters are replaced by geotextiles is the permeability reduction during filtration process as a result of contamination. The term "contamination" is meant as the retention of a certain amount of soil particles between the fibres whereas "colmatation" results in the total impermeabilization of geotextiles.

The studies carried out by the authors showed that among Romanian geotextiles the most liable to contamina-

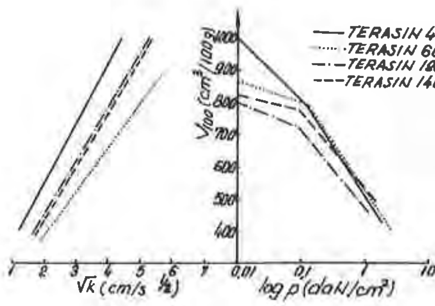

Fig.5. Relations between density, permeability and applied pressure on geotextiles.

tion are the loose unwoven materials like BIDIM (unimpregnated TERASIN, MADRIL) whereas denser types (NETESIN and especially impregnated TERASIN) are less sensitive /4,5, 8/.

The evolution of filtration process through granular filter/soil and geotextile/soil systems has been studied on different models.

The model presented in Fig.6 was intended to the

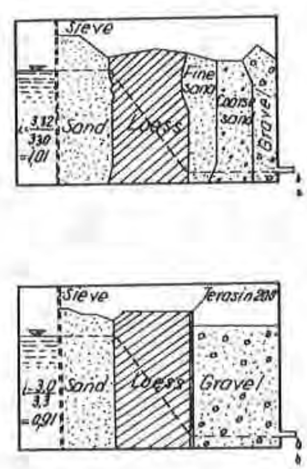

Fig.6. Model for studying geotextile efficiency.

study of filtration under low gradients. It is made of organic glass and divided in two parts by a perforated wall. In the left side water is maintained at a constant level and then reaches the filter/sand system after passing through sand and loess layers. In order to compare various draining systems the permeability was expressed by a global coefficient $\overline{k}$ deduced from Darcy's law and corresponding to a draining width b equal to 1 cm ; the upstream water height is H and the total discharge is Q :

$$\overline{k} = Q/(H\ b\ t\ i) \qquad (6)$$

The results of these tests are presented in Fig.7; they show that, on one side, the draining capacity of Romanian geotextiles manufactured from synthetic wastes is equivalent to that of foreign - made geotextiles or of granular filters and, on the other side, the contamination process is unimportant and it develops in all cases during 3 ... 4 days from the beginning of tests.

The filtration under hydraulic gradients higher than the unity was studied with the arrangement showed in Fig. 8a. The vertical water flow passed through a silty soil

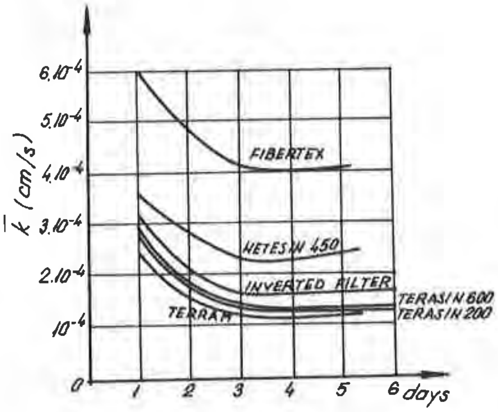

Fig.7. Variation of the coefficient of permeability for loess/geotextile and loess/inverted filter systems.

- from examined filters, none was contamined.

From these experiments it resulted also that only a small amount of soil passes through geotextiles (less than 20 % from particles in suspension) and this happens only as filtration begins ; later on the percentage decreases below 1 %.

5 A NEW COMPOSITE GEOTEXTILE DRAINING CARPET

The new prefab material recently realized in Romania consists of a gravel layer glued on a synthetic fabric 1-mm mesh placed between two sheets of NETESIN 300. Fig.9a shows a drain where this material has been employed and in Fig.9b the scale model used for studying the drainage capacity of the system is presented.

Experiments performed with this new drain showed that its efficiency is influenced by the nature of fibres which the geotextile covering the gravel fill is made from. If the geotextile contains less than 10 % natural fibres the permeability of prefabricated drains diminishes during the first 3 days of use, then it remains constant; some decontamination may also occur later on(Fig.10). If the geotextile contains up to 20 % natural fibre wastes the permeability slightly decreases during the first week of use. The influence of the nature of fibres is more significant in the case of laminary flow.

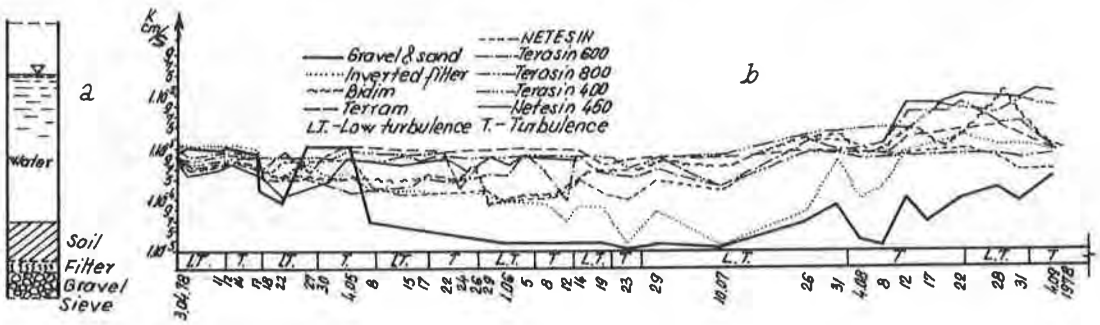

Fig.8. Filtration under high gradients.

layer (22 % clay, 53 % silt and 25 % sand fractions) then followed a variant of filtering system (classical, gravel or one of the geotextiles shown in Fig.8b).

For achieving a more realistic modelling, water was poured either directly on the soil layer (as a rainfall) or through a layer of synthetic geotextile filter (NETESIN).

The coefficient of permeability was determined under variable water head conditions. Results obtained from filtration tests carried out over a period of 6 months and represented in Fig.8b lead to following findings :

- the soil/geotextile draining system reduces the differences between the permeability coefficients of various geotextiles, and after a certain time the filtration curves get very close ;
- when water turbulence was high (during rainfalls) the permeability decreased, whereas when a filtering layer was interposed between water and soil, the permeability increased, i.e. a decontamination took place. During 1 to 3 months from test start there were sensible permeability variations ; the formation of a "natural inverted filter" may explain the stabilization of filtration process and even the further increase of permeability ;
- denser and thicker geotextiles (TERRAM, TERASIN 600 or 800) lead to a quicker stabilization of drainage flow than looser and thinner ones ;

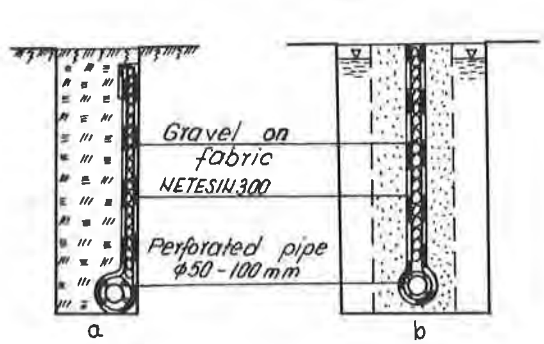

Fig.9. Drain with prefab gravel-geotextile carpet.

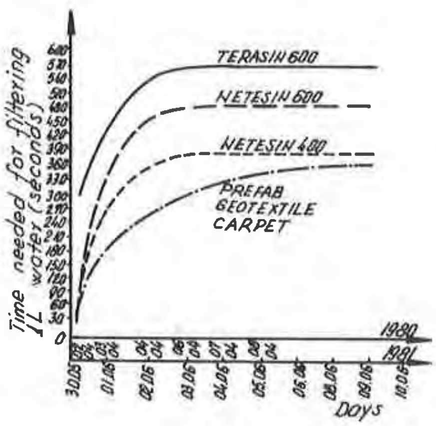

Fig.10. Results of contamination tests.

6 WORKS CARRIED OUT IN ROMANIA WITH GEOTEXTILES MADE OF WASTE MATERIALS

The first experiments were made in 1974 when protection layers and a transversal drain has been realized on a highway near the city of Iasi/9/ using NETESIN. The total area of geotextiles made from waste materials and used so far in Romania exceeds 2 mil. m^2 ; about a half of them served for draining works. Also slope protection railway, anticontaminant layers, and bank protections were executed ; they were experimented also on some asphaltic covers /10,11/.

The variation of water discharges collected by transversal drains has been recorded from 1976 on ; some variations have been observed due to rainfalls but no colmatation was reported (Fig.11). Thus during the fall of 1978 after a dry period discharges of drains 12-16 decreased towards minimum values, and after 15 hours of rain - fall the discharge grew up to almost maximum values.

7 CONCLUSIONS

The research carried out resulted in the following main findings :

- the microscopical study of geotextiles is very useful, as it allows the structural peculiarities to be outlined, the contamination effect and the presence of occluded air bubbles to be cleared up ;
- by capillary effects, geotextiles retain appreciable quantities of water, but they easy release them back when low suctions (of several dozens of cm water head) are applied ;
- in order to characterize the porous systems of geotextiles and of soils in contact with them, as well

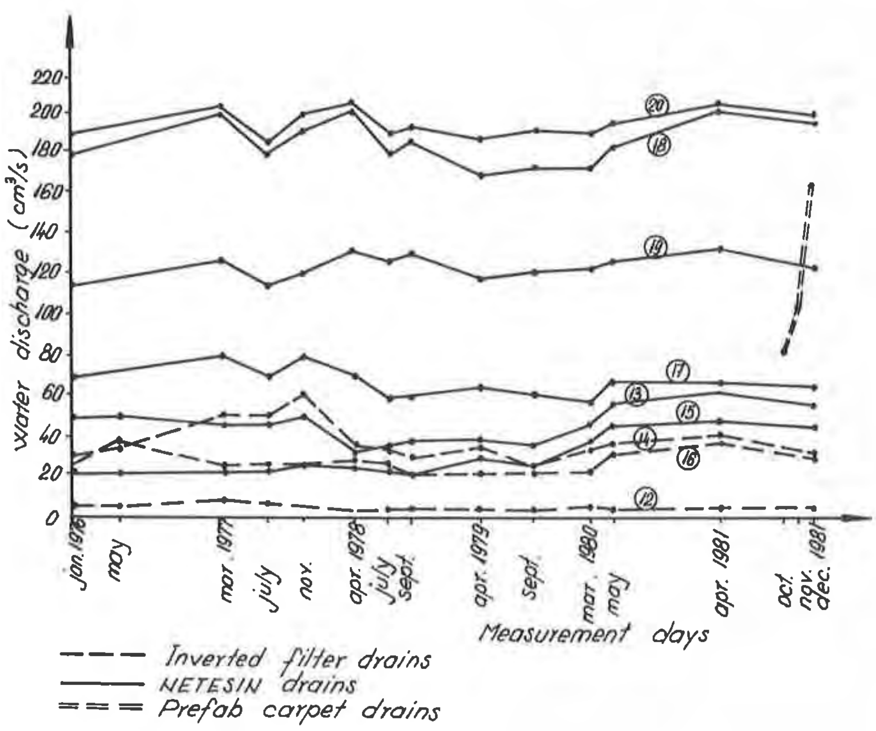

Fig.11. Variation of water discharge for transversal drains.

as the pore size distribution, it is useful to represent both particle size distribution and water retention curves on the same semilogarithmic diagram, thus obtaining a global feature of soil-geotextile systems ;

- the pore size distribution as determined by means of water retention curves is in good agreement with that observed by microscopic studies whereas the sieving of normalized sand shows smaller pore sizes than actual ones ;

- the unwoven geotextiles produced in Romania have practically the same structural and hydric properties as those manufactured abroad ;

- under hydraulic gradients as high as 0.5 to 1.0 a permeability decrease to about half the initial value occurs - this process has a duration of few days ;

- geotextiles are much more compressible than soils - from this reason the permeability of geotextiles is intensely influenced by overburden pressures exerted on them ;

- a correlation between pressure and permeability has been formulated ; it includes some experimental coefficients which may be determined by oedopermeameter tests ;

- the original oedopermeameter described in the Paper is provided with a water de-aeration device, which may be extended to ordinary permeameters for soils ;

- when hydraulic gradients are high, in the case of turbulent flow, the permeability coefficient of soil - synthetic geotextile systems is variable upon a certain time interval, then it approaches a steady value. Even in these conditions contamination of unwoven fibres does not occur, and moreover a decontamination effect was observed. The laminary flow through drains is influenced by the proportion of natural fibres in the composition of geotextiles.

- when unwoven filters made of synthetic wastes are used instead of classical inverted filters significant economical advantages are obtained : water discharge through geotextile drains remained practically unchanged after 6 years of operation.

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About Longitudinal Permeability and Draining Capacity of Non-woven Geotextiles

La perméabilité longitudinale et la capacité de drainage des géotextiles non-tissés

The paper presents a device and a laboratory methodology allowing the simulation of a parallel streamlined flow under a loading perpendicular on the fabrics surface. The results of the experiments reveal the influence of fibre characteristics and consolidation technologies over the studied parameter value. The same conclusions are obtained by a theoretical analysis starting from the idea of an existing laminar flow along a "corridor" limited by two fibres. A comparison between the permeability coefficients experimental values k_p and k_n shows that $k_p = C\ k_p$ $(3 < C < 25)$. The final part of the paper defines the non-woven fabrics drain capacity diminution ratio in relationship both with the loading and the clogging state.

L'article présente un dispositif et une méthodologie de laboratoire, qui permet la simulation d'un écoulement à lignes de courant parallèles en conditions de charge normale sur la surface des géotextiles. Les résultats des expérimentations relevent l'influence des caractéristiques des fibres ainsi que des technologies de fabrication sur la valeur du paramètre étudié. On peut arriver aux mêmes conclusions aussi par un traitement théorique, en partant de l'hypothèse d'un courant laminaire au long d'un couloir bordé par deux fibres. Une comparaison entre les valeurs expérimentales des coefficients de perméabilité k_p et k_n montre que $k_p = C\ k_p$ $(3 < C < 25)$. Dans la partie finale du rapport, on définit théoriquement des indices de réduction de la capacité de drainage des géotextiles non tissés par rapport à la charge et l'état de colmatation.

1. INTRODUCTION

If the fact is accepted that fabrics are porous media having the ability to pass water according to Darcy's Law, the two permeability coefficients can be defined by comparing them to the flow direction as related to the fabrics plane. Thus:

k_n = coefficient of normal permeability (water passing through the fabric perpendicular on its plane);

k_p = coefficient of permeability in the plane (water passing through the fabric parallel with its plane).

Non-woven geotextiles permeability is influenced by many factors reflecting the utilisation conditions of the "soil-geotextile complex". Some of the main influencing factors are:

- fabrics plane normal preasure and loadings dynamics;
- type of fabric, raw material included, fibres caracteristics and dimensions, bonding agents, formation of the fabric, a.s.o.;
- mineralogical composition, shape and dimensions of the neighbouring soil particles;
- physical and chemical water characteristics.

For the study of normal permeability (k_n) the last decade literature offers a large range of methods of determination with devices and methodologies original or adapted after the geotechnical laboratory techniques.

For the longitudinal permeability (k_p) most of the published works notice experimental arrangements for which the flow is radially and symetrically going through the fabric layer. Actually the flow through non-woven fabrics runs symetrically and in the plane taking well established directions, lengthwise or crosswise.

It is of use to remember the fact that one of the non-woven geotextiles feature is the anisotropy of all characteristics especially in comparison with the two directions.

2. DEVICE FOR THE DETERMINATION OF NON WOVEN GEOTEXTILES PERMEABILITY IN THE PLANE

To reunite and simulate all the factors influencing a certain characteristic during the test is known to be almost impossible. Therefore a device has been designed to maintain the equally distributed normal loading as unique factor of influence over the environment.

The operation of the system shown in fig.1 allowes the determination of the discharge passing in the plane of the non-woven fabric for various loadings applied by means of the elastic membrane "2". It is also possible to strictly controle the hydraulic gradient (i) having values that range from $i = 0.01$ up to $i > 1.0$.

The permeability coefficient value obtained with this devices is:

$$k_p = \frac{q}{\pi (DT_\sigma - T_\sigma^2)\cdot i} \qquad (cm/s) \qquad (1)$$

where:

q = rate of discharge (cm^3/s);
D = fabric sample exterior diameter (cm);
T_σ = fabric thickness for σ loading (cm);
i = hydraulic gradient.

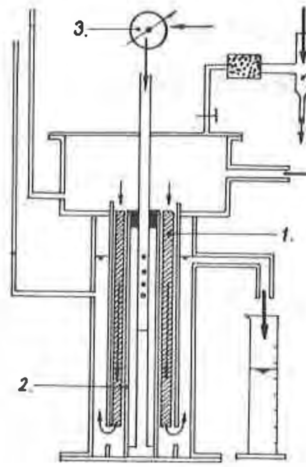

Fig.1. Device for measuring the lengthwise permeability: 1 - cylindrical sample of geotextile; 2 - pressurized elastic membrane; 3 - manometer; 4 - vacuum pump.

3. TESTED MATERIALS. CONDITIONS OF EXPERIMENTS

With the use of the above mentioned device experiments have been made on a large range of non-woven fabrics among which the following are discussed in the present paper:

- MADRIL(R) M - non-woven fabric made of polypropylene fibres having the fineness (linear density of fibres) 0.66 tex (d_f = 31 μm) and with the length 60 mm, mechanically bonded by needlepunching.

The mass for 1 cm^2 of fabric (μ_s) is 2.85×10^{-2} - 5.92×10^{-2} g/cm^2.

- MADRIL(R) V - non-woven fabric made of polypropylene fibres having the fineness 2 tex (d_f = 53 μm) and the length 100 mm, mechanically bonded by needlepunching (μ_s = 3.56×10^{-2} - 7.15×10^{-2} g/cm^2).

- BIDIM(R) - non-woven fabric made of continuous filament polyester fibres having d_f = 28 μm spun and then mechanically entangled by needlepunching (μ_s = = 1.49×10^{-2} - 5.41×10^{-2} g/cm^2).

- V.P.P. Im.- non-woven fabric made of polypropylene fibres having the fineness 1.7 tex and the length 100 mm bonded mechanically by needlepunching and chemically by Romacryl (μ_s = 3.24×10^{-2} - 5.81×10^{-2} g/cm^2).

Before being positioned in the device the MADRIL and BIDIM samples have been dipped in water for 24 hours and the V.P.P. Im. sample for 30 days their setting being also performed under water.

Flow rates were measured after one hour from the loading application and the hydraulic gradient stabilization. For each stage three values had been scored up the tests being performed on three samples in three parallel devices.

4. OBTAINED RESULTS

Analysing the diagrame from figure 2 (σ = 0.1-0.22 Pa) one can notice that for any value of σ the $q = \Omega k_p i$ dependence takes place between the limits of domains determined by the intrinsic fabric characteristics (μ_s, T_σ, fibre fineness - λ, non-woven fabric denseness - ∂_σ (3) a.s.o). The observation made for the non-woven fabric BIDIM(R) is valide for all the tested fabrics. In the figure the hatched areas represent experimental values scattering domain.

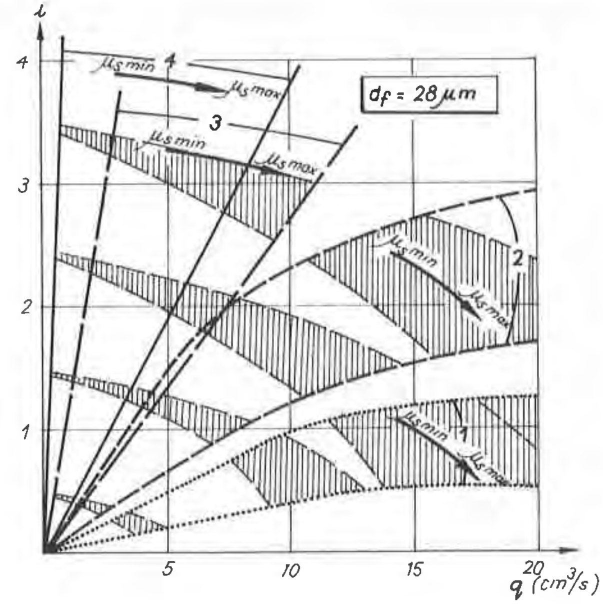

Fig.2. 1 - σ = 10 k Pa; 2 - σ = 14 k Pa; 3 - σ = 18 k Pa 4 - σ = 22 k Pa; $\mu_{s\ min} = 1.49 \times 10^{-2}$ g/cm^2; $\mu_{s\ max} = 5.41 \times 10^{-2}$ g/cm^2

Such a behaviour is to be explained by the re-setting of fibres due to the σ loading as well as to the appearance of several by-phenomena generated by the water movement (i.e. fibres vibrations) and depending of the polymer specific weight (γ_p), λ, ∂_σ and μ_s. Thus, in the expression of q discharge the term influenced by this fibres re-setting is the product between Ω (flow cross section) and k_p. For emphasizing the variation law $\Omega k_p = f(\sigma)$ the diagram shown in figure 3 had been drawn (for BIDIM(R)).

In the representation $\Omega k_p = f(\sigma)$ the hatched area corresponds to the areas from figure 2 in which the discharge velocities v are directly proportional to the hydraulic gradient i and the domain limited by the dotted lines correspond to the areas in which $v = f(i)$ has an exponential form.

From the point of view of permeability as a function of non-woven fabric fibres fineness (fig.4) for any value of σ the diferences are obvious and they diminish when $\sigma_{n+1} > \sigma_n$.

The diagrams from figure 5 comparatively present the variation $k_p = f(\sigma)$ and $k_n = f(\sigma)$ for the four tested materials. One can notice differences of behaviour in comparison with the fabric layer bonding technology. Thus for the same value of σ the mechanically bonded fabrics have permeabilities almost one order of magnitude higher than those both mechanically and chemically bonded.

The marked permeability diminution tendency for V.P.P. Im. fabrics can be due to the bonding film in time capacity to swell when being in contact with water.

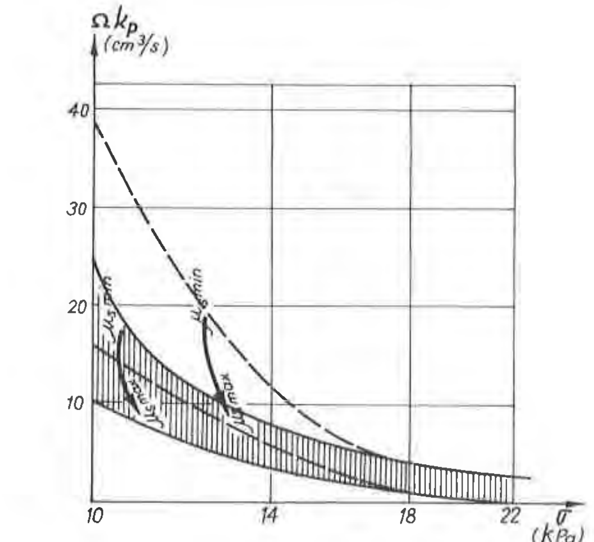

Fig.3. $\mu_{s\ min}=1.49\times10^{-2}$ g/cm²; $\mu_{s\ max}=5.41\times10^{-2}$ g/cm².

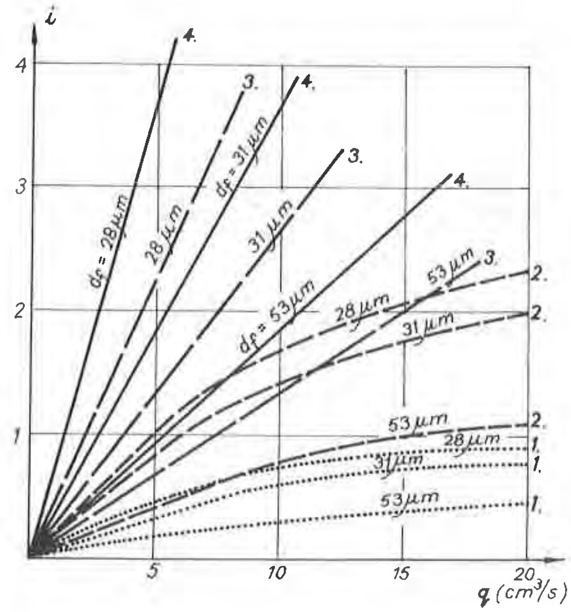

Fig.4. Fibres fineness influence over the drained discharges; d_f = fibre diameter.

For the determination of k_n a modified oedometer box had been used (1). Comparing the results obtained for k_n and k_p one can notice an existing relation of the type $k_p = C\ k_n$ in which C is varying as a function of λ and σ. In table 1 the ratio k_p/k_n values obtained by experiment are presented.

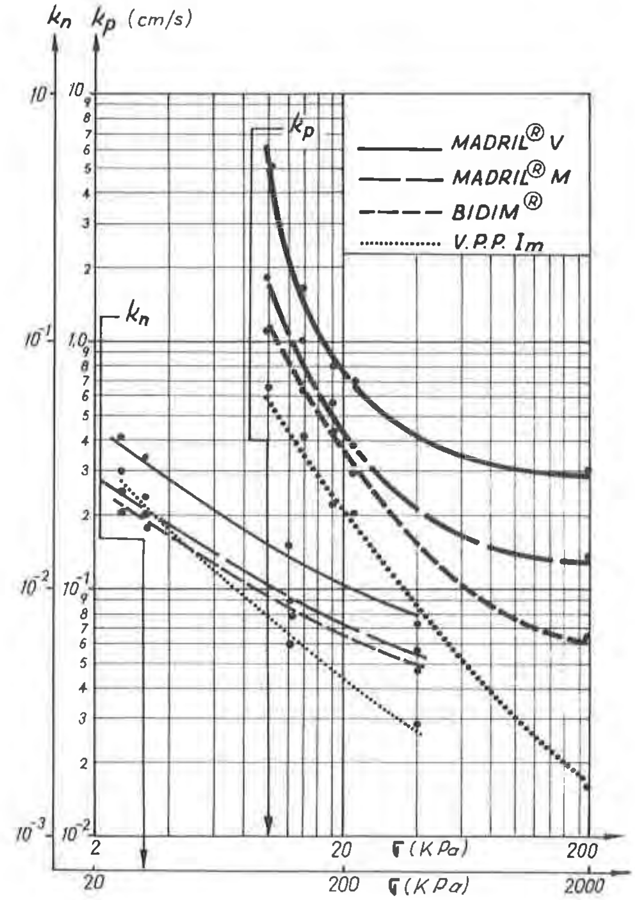

Fig.5. Variation of permeability (k_p and k_n) as related to the loading normally applied on the geotextile plane

Table 1. Variation of the k_p/k_n ratio as related to λ and σ.

	k_p cm/s		k_n cm/s		k_p/k_n	
	σ (KPa)		σ (KPa)		σ (KPa)	
	22	200	22	200	22	200
MADRIL® V	$7\cdot10^{-1}$	$3\cdot10^{-1}$	$4\cdot10^{-2}$	$1{,}2\cdot10^{-2}$	17,5	25
MADRIL® M	$3{,}6\cdot10^{-1}$	$1{,}4\cdot10^{-1}$	$2{,}6\cdot10^{-2}$	$7.3\cdot10^{-3}$	13,8	19.2
BIDIM®	$2{,}8\cdot10^{-1}$	$6\cdot10^{-2}$	$2.2\cdot10^{-2}$	$6{,}5\cdot10^{-3}$	12,7	9,2
VPP Im	$2\cdot10^{-1}$	$1{,}7\cdot10^{-2}$	$2.8\cdot10^{-2}$	$4{,}5\cdot10^{-3}$	6,4	3.8

5. THEORETICAL CONSIDERATIONS CONCERNING THE PERMEABILITY IN THE PLANE OF GEOTEXTILE

The relation (2) given by Poiseuille (2) is supposed for the average laminar flow velocity through a capillary tube

$$v = \frac{\gamma_w}{8\eta_w} R^2 i \qquad (cm/s) \qquad (2)$$

in which:

γ_w = unit weight of water;

η_w = dynamic viscosity of water;

R = radius of the capillary tube

is also true in the case of non-woven fabrics considering a flow along a capillary "corridor" limited by 2 fibres having a d_f diameter (fig.6).

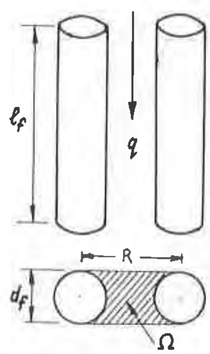

Fig.6. Model of laminar flow: R = distance between fibres; Ω = flow cross-section.

If the hydraulic radius R_H is to be defined as the ratio between the amount of water passing through the capillary "corridor" with Ω section and the wetted surface

$$R_H = \frac{4R - \pi d_f}{4\pi} \qquad (3)$$

the following expression is obtained for R

$$R = \frac{\pi(4R_H + d_f)}{4} \qquad (4)$$

Replacing the value of R given by (4) in the relation (2) one obtaines

$$v = \frac{\pi^2}{128} \frac{\gamma_w}{\eta_w} (4R_H + d_f)^2 x i \quad (cm/s) \qquad (5)$$

If refference is to be made now to the ensemble of fabric the hydraulic radius R_H can be defined as the ratio between the water amount passing through the fabric pores and fibres wetted side surface. Thus if it is supposed that the fibres have an absolute value volume (for a normal loading σ) they make a void equal to e_σ (e_σ = = void ratio) and their side surface is equal to A_l (A_l= = fibres side surface corresponding to μ_s mass).

$$R_H = \frac{e_\sigma}{A_l} \qquad (6)$$

Replacing the R_H value given by (6) in the relation (5) one obtaines:

$$v = \frac{\pi^2}{128} \frac{\gamma_w}{\eta_w} \left(\frac{4e_\sigma + A_l \cdot d_f}{A_l}\right)^2 i \quad (cm/s) \qquad (7)$$

The relation (7) represents the expression of the flow velocity through a section entirely full of water. In order to meet the real flow conditions through porous media the multiplication of (7) by porosity n_σ or by the ratio($(e_\sigma/1+e_\sigma)$) is necessary. Thus:

$$v = \frac{\pi^2}{128} \frac{\gamma_w}{\eta_w} \left(\frac{4e_\sigma + A_l \cdot d_f}{A_l}\right)^2 \cdot \frac{e_\sigma}{1+e_\sigma} \cdot i, \qquad (8)$$

The expression (8) has the same form as the Darcy's Law ($v = k_p i$). For d_f, A_l and e_σ relations can be written as a function of laboratory measured elements,namely

$$d_f = 2\sqrt{\frac{\lambda}{10^5 \pi \gamma_p}} \qquad (cm) \qquad (9)$$

$$A_l = 2\mu_s \sqrt{\frac{10^5 \pi}{\lambda \gamma_p}} \qquad (cm^2) \qquad (10)$$

$$e_\sigma = \frac{T_\sigma \cdot \gamma_p}{\mu_s} - 1 \qquad (11)$$

Replacing in (8) the values given by (9),(10),(11) one obtains for k_p from the velocity expression the value

$$k_p = \frac{\pi}{32} 10^{-5} \frac{\gamma_w}{\eta_w} \lambda \left(\frac{T_\sigma \gamma_p^2 - \gamma_p \mu_s + \mu_s^2}{\gamma_p \mu_s^2}\right)^2 \frac{T_\sigma \gamma_p - \mu_s}{T_\sigma} \quad (cm/s) \qquad (12)$$

Analysing the expression (12) one can notice the non-woven fabrics physical characteristics influence over the permeability value, fact also revealed by the results of the above described experiments. The values of k_p obtained by using the relation (12) are quite resemblant to those obtained by experiments. Between the calculated value of k_p and its experimental one (k_{p_c} and $k_{p_{ex}}$) there is a relation of the type

$$k_{p_{ex}} = C \cdot k_{p_c} \qquad (13)$$

where C is a coefficient varying as a function of the fabric compaction. For C values had been experimentally obtained ranging between 1,4 and 0,25 inversely proportional to the σ value increment. Thus, the expression (12) can be formulated:

$$k_p = C \frac{\pi}{32} 10^{-5} \frac{\gamma_w}{\eta_w} \left(\frac{T_\sigma \gamma_p^2 - \gamma_p \mu_s + \mu_s^2}{\gamma_p \mu_s^2}\right)^2 \lambda \frac{T_\sigma \gamma_p - \mu_s}{T_\sigma} \quad (cm/s) \qquad (14)$$

6. ASPECTS CONCERNING THE NON-WOVEN FABRICS DRAIN CAPACITY

If one considers a non-woven fabric having T thickness and B width the following expression can be written (4)

$$k_p = \frac{q}{T \cdot B \cdot i} \qquad (cm/s) \qquad (15)$$

If it is noted: k_{pi} = initial permeability; $k_{p\sigma}$ = permeability under loading; permeability diminution ration "r" can be thus defined

$$r = \left(1 - \frac{k_{p\sigma}}{k_{pi}}\right) 100 \qquad (\%) \qquad (16)$$

when i = ct the relation (16) becomes

$$r = \left(1 - R \frac{q_\sigma}{q_i}\right) 100 \qquad (\%) \qquad (17)$$

where

$$R = \frac{T_i}{T_\sigma} \quad or \quad R = \frac{1}{1-\varepsilon} \qquad (18)$$

in which ε = fabric strain under loading.

The index "r" thus expressed can be considered as fabric drain capacity efficiency ratio. The coefficient R is a typical element for each fabric depending on the fabric type and the σ loading. Fabrics drain capacity is liable to be influenced in time by the clogging with solid flow or by a biological clogging.

Thus in the case of solid flow clogging, the water containing fine particles passes through the fibred material lengthwise reaching the collector in a time interval $\Delta t_m = t_2 - t_1$. Obviously the length of the material favours the particles retention to a greater extent than in the case of a normal flow on the fabric plane. Consequently a corresponding transport capacity of solid flow will be imposed to the fabric. This requirement can be satisfied if the fabric is correctly selected taking into consideration the fibres characteristics and bonding technology influence over its permeability in the plane.

As far as the biological clogging is concerned the phenomenon although depending on less measurable environmental factors can be simply expressed as $q_{to} > q_t$ in the time interval $\Delta t_b = t - t_o$. In comparison with environmental conditions Δt_b is much greater then Δt_m.

If it is considered that in Δt_m interval σ, T and i are constant one can write

$$r_c = \left(1 - \frac{\Omega_{t_2}\, k_{pt_2}}{\Omega_{t_1}\, k_{pt_1}}\right)\cdot 100 \qquad (\%) \qquad (19)$$

where Ω represents the effective surface of flow through the drainant fabric before (Ω_{t_1}) and after (Ω_{t_2}) clogging. Thus (19) acts as a ratio expressing the drain capacity diminution by clogging.

If it is admitted that by clogging with solid flow μ_s increases by a weight corresponding to n soil particles (considered as being spherical) having the unit weight γ_s and the diameter d

$$\Delta\mu_s = \mu_{st_2} - \mu_{st_1}$$

$$\Delta\mu_s = \frac{n\pi d^3 \gamma_s}{6}, \qquad (g) \qquad (20)$$

the surface Ω_s, by which the n particles obturate the initial effective surface of flow Ω_{t_1} will be:

$$\Omega_s = \frac{3}{2}\,\frac{\Delta\mu_s}{d\cdot\gamma_s} \qquad (cm^2) \qquad (21)$$

Thus if for B = 1, Ω_{t_1} has the value

$$\Omega_{t_1} = T - d_f \cdot l_f \qquad (22)$$

in which l_f = nominal length of fibres (3)

$$\Omega_{t_1} = T - 2\mu_s\sqrt{\frac{10^5}{\lambda\pi\gamma_p}} \qquad (cm^2) \qquad (23)$$

then

$$\Omega_{t_2} = \Omega_{t_1} - \Omega_s \qquad (24)$$

or

$$\Omega_{t_2} = T - 2\mu_s\sqrt{\frac{10^5}{\lambda\pi\gamma_p}} - \frac{3}{2}\,\frac{\Delta\mu_s}{d\gamma_s} \qquad (24')$$

Thus (19) can be written:

$$r_c = \left\{1 - \frac{k_{pt_2}}{k_{pt_1}}\left[1 - 1{,}5\,\frac{\Delta\mu_s}{d\gamma_s}\,\frac{\sqrt{\lambda\pi\gamma_p}}{\left(T\sqrt{\lambda\pi\gamma_p} - 2\mu_s 10^{2,5}\right)}\right]\right\} 100\ (\%) \qquad (25)$$

Analysing the expression (25) one can notice that the reduction by clogging of the drain capacity is influenced by the dimensions of the retained particles and their amount as well as by the fabric intrinsic characteristics.

As far as the ratio between the permeability before and after the clogging is concerned the problem is more difficult as after clogging the flow passes through a grain-fibre-structured porous medium.

7. CONCLUSIONS

- The present paper submits to the attention of the specialists a device and a laboratory methodology for the determination of lengthwise permeability (k_p) for non-woven fabrics that simulates the hydraulic and loading real conditions.

- The study deals with the geotextile intrinsic permeability aiming at emphasizing its characteristics influence over the viewed parameter. The results of the experiments over a large range of non-woven fabrics (from which four are presented) demonstrate the lengthwise permeability dependence on the fibres fineness, the fabrics strain under loading as compared to the applied loading, the materials production technology a.s.o. The theoretically obtained expression of k_p on the basis of a laminar flow through a "corridor" limited by two fibres confirm the results of the experiments.

- Comparing the experimental values obtained for k_p and k_n it is noticed that $k_p/k_n > 1$ (3 ... 25).

- Analysing the intrinsic drain capacity of the fabric as well as that existing in clogging conditions diminution and efficiency indices are defined for the studied parameter. In this case too the necessity to take into consideration the geotextile fibres characteristics is emphasized. Furthermore the drain capacity under the circumstances of the existing clogging phenomenon is influenced also by the retained solid particles dimensions and amount.

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Test Areas with Vertical Drainage Systems

Champs d'essai aux systèmes de drainage vertical

In recent years the supply of different types of prefabricated drains for vertical drainage systems has been considerably increased. At the same time laboratory test methods have been developed to analyse the essential characteristics of drain materials and of the drains themselves. However the explanation of the measured difference between laboratory test results i.e. the interpretation of laboratory test results into actual practice, confront the research-worker often with difficult problems. In order to support and verify the laboratory test results of prefabricated drains, six test sites have been equipped between 1979 and 1982 for this purpose in the sub-grade of the State Highway Nr.1 south-east of Amsterdam. In these test areas the behaviour of five types of prefabricated vertical drains have mutually been compared and also to the conventional sand-drains. This paper considers the most important results of these in-situ tests. Both the measurement of settlements as well as the piezometer readings indicate a difference of behaviour amongst the test areas i.e. a difference between the functioning of the different drain types.

Dans les dernières années l'offre de divers types de drains préfabriqués pour le drainage vertical a augmenté considérablement. En outre des essais de laboratoire pour la détermination des qualités des matériaux drainantes et des drains considérées essentielles ont été développés. Toutefois, l'interpretation des différences de comportement en laboratoire des matériaux et des drains, c'est à dire la traduction des résultats d'essais de laboratoire en performance pratique, souvent pose de grands problèmes au chercheur. Pour donner assistance aux recherches et pour vérifier les résultats des essais de laboratoire obtenus avec des drains préfabriqués, entre 1979 et 1982 une demidouzaine de champs d'essai a été installée sur la route nationale no.1 au Sud-est d'Amsterdam. Dans ces champs d'essai on a comparé la performance pratique de cinq types de drain préfabriqué tout à titre mutuel qu'auprès du fonctionnement des drains de sable. Les mesurages de tassement si bien que ceux de la pression interstitielle révèlent des différences du comportement pratique des champs d'essai et, par conséquent, des drains préfabriqués.

1. INTRODUCTION.

In the field of vertical drainage systems, a number of important developments have been presented in recent years. The prefabricated band-shaped drain has been recognized more and more as an alternative for the conventional vertical sand-drain for vertical drainage projects. The increasing supply of different types of prefabricated drains is the first indication of this fact. The usual way of in-situ testing to establish the suitability of new drains is not only time-consuming, but also very costly and therefore not very attractive. Therefore producers and users of drains proceeded to develop laboratory test methods to analyse and detect a number of properties of prefabricated drains, which are considered to be essential. At present a good impression of the usability of new materials for drainage systems can be obtained with laboratory tests, by means of permeability tests on the filter and discharge capacity tests of the drains. However the main problem is still to implement the differences found with laboratory test results of different materials and drains and the consequences thereof for the usability in practice. This of course, because the circumstances during testing in a laboratory differ considerably compared to those of a drain in-situ, buried into the ground. Therefore in situ-testing will remain indispensable on a modest scale, to support and verify laboratory investigations.

The State Road Engineering Division, in cooperation with Delft Soil Mechanics Laboratory and producers of prefabricated drains have prepared six test sites. Except for the conventional sand-drain, five prefabricated drains have been installed in these test-sites, of which three drains have been equipped with a non-woven fabric filter. The aim of these tests is in first instance, to mutually compare the behaviour of the installed drains in situ. In a later stage it will be investigated whether the recorded differences of the drain behaviour in situ can be related to the laboratory test results of certain material properties and drainage properties.

In the present paper however, only the results of the in situ-tests at the different test sites will be discussed.

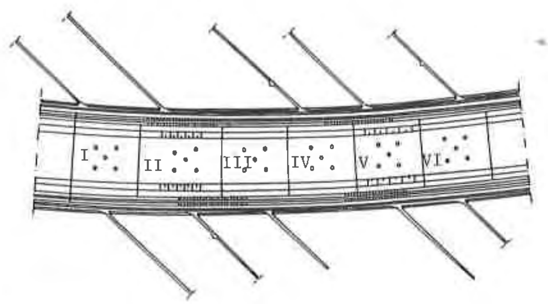
2. DETAILS OF THE TEST-SITE AND THE INSTALLED DRAINS.

2.1. Location.

The test-sites comprise a part of the fill for the 2x2-lane state highway, which is under construction in the neighbourhood of Amsterdam. The test-sites are located in such a way in the alignement of the road, that in each site an area of 30x30 m^2 is not intersected by (former) open boundary-ditches. The width of each test-site varies between 50 to 55 m and the length of each site varies between 65 and 80 m. The six test-sites are bordering each other and occupy approximately a total length in the road alignement of 500 m (see figure 1).

2.2. Applied prefabricated drains.

From the considerable supply of prefabricated drains five drain types have been selected, based on laboratory test results and on the cost price, in such a way that the most used types of drain-constructions are represented. The five selected drain types and the conventional



o settlement plates

figure 1. Location of settlement plates and test-sites

sand-drain are specified in more detail in table 1. For reasons of construction the principal of the road project did not allow to prepare a reference site without vertical drains at all.

Due to the small differences in stratigraphy of the compressible layers, an arbitrary lay-out of the position of different drain types is possible. However, for historical reasons a choice has been made in such a way that conventional sand-drains could directly be compared to paper filter type drains and the latter to non-woven filter drains. Both drains with a non-woven fabric filter have been installed in such a way, that a straight comparison between these two drain types was possible. Finally the lay-out of test sites as shown in table 1 has been achieved.

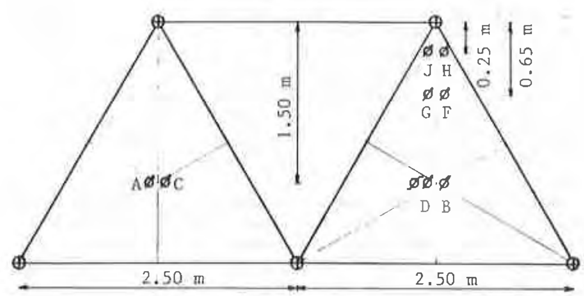
2.3. Instrumentation of test-sites.

All measuring equipment, necessary to record the behaviour of the drains in situ, has been installed and concentrated within an area of 20x20 m^2 for each test-site. The centre of this area coincides with the centre of the test-site, so that, in view of the thickness of the compressible layers, the influence of bordering test-sites with another drain type on the one hand and the sloping edges of the sandfill on the other hand, can be regarded negligible.

Settlement plates have been installed directly on the ground surface , which was N.A.P.-1.40 m (N.A.P. is the national reference level), in each corner and in the centre of the measuring area. These settlement plates have been constructed in such a way that they can also be used as open piezometers in order to record the groundwater table in the fill. Besides, an open piezometer with its filter reaching down into the sand formation below the compressible layers, with a thickness of approximately 9 m, has been installed in the centre of the test site.

Per test-site eight piezometers of the Kistler type have been installed close to the centre of the measuring area These piezometers reaching down to different depth have been installed at distances from the drains of respectively 1.50 m, 0.65 m and 0.25 m. The configuration of the locations of piezometers in conjunction with the position of the drains and the depth of their filters are indicated on figure 2. In order to restrict as much as possible the influence of negative skin friction and hence the possibility of pushing down the piezometers, the stand pipes of each piezometer have been cased over the height of the sandfill.

Recording of the settlement plates and piezometer-readings will be continued up to mid August 1982.



depth of installation of water pressure meter filters:

- A 3.2 à 3.3 m -N.A.P.
- B, F, H 4.7 à 4.9 m -N.A.P.
- C 7.0 à 7.1 m -N.A.P.
- D, G, J 8.6 à 8.8 m -N.A.P.

⊕ drain

figure 2. Location of water pressure meters in relation to drain positions.

In order ro record the waterpressure potential over the drain filter, three drains per drain type have been equipped with miniature piezometers at both sides of the filter. Due to a progressive zero-point fluctuation after installation of these miniature piezometers, recording of these measurements had to be stopped in an early stage.

Table 1. Applied drains and lay-out of test sites.

Test site	Drain name	Type	Particular details	Dimensions
I	Bidim	monolith	polyester non-woven fabric 550 g/m^2	150x4 mm^2
II	Desol	monolith	perforated polyethylene tubes	95x2 mm^2
III	Colbond CX 1000	composite	core : polyester Enkamat filter: polyester non-woven fabric 130 g/m^2	150x5 mm^2
IV	Mebra-ester	composite	core : grooved polyethylene filter: polyester non-woven fabric 120 g/m^2	100x3 mm^2
V	Mebra-paper	composite	core : grooved polyethylene filter: paper 115 g/m^2	100x3 mm^2
VI	Conventional sand-drain	monolith	drainage sand: k= 12 m/24 hrs	Ø 260 mm

Table 2. Composition of the compressive formation.

soil description	γ	Consolidation coëfficients ($x10^{-7}$ m^2/sec) and compressibility constants								thickness of layers per test-site (m)					
	$\frac{kN}{m^3}$	Δp = 10à20 kN/m^2 (I)				Δp = 50à90 kN/m^2 (II-VI)									
		c_v	c_h	c_p	c_s	c_v	c_h	c_p	c_s	I	II	III	IV	V	VI
peaty clay	14.5	–	–	50	350	–	–	30	240	0.35	0.40	0.40	0.40	0.25	0.50
peat	10.0	100 25	50 15	15à25	80à100	3 1	1 1	6	54	2.00	2.00	2.00	2.00	2.00	2.00
peat	10.2	50 15	25 5	15à25	80à100	2.5 0.5	1 0.5	6	54	1.90	1.85	2.10	1.60	2.10	1.90
clay with remnants of vegetation	15.0	2.5	2.5	25	180	1.5	1.5	12	150	1.35	1.50	1.10	1.35	1.50	1.10
sandy clay	16.5	30	7	50à60	250à300	30	6	25	160	2.10	1.65	1.05	2.25	1.40	1.40
peaty clay with shell fragments	14.0	1	1	20	60à90	0.25	0.4	9	80	1.25	1.65	2.20	1.15	1.35	1.30
base-peat	–	1	1	15à25	80à100	4	2.5	6	54	0.35	0.35	0.45	0.20	0.30	0.40

3. SUBSOIL INSTRUMENTATION.

3.1. General

In the centre of each test-site one medium-range cone penetration test and one boring, continuous sampling Ø 66 mm, down to a depth of 12.0 m below original grade, have been performed. Laboratory tests, concerning the compressibility, the friction parameters and horizontal- and vertical permeability, have been performed on selected samples of this borings. From the results of these borings and cone penetration tests a geotechnical cross-section, from ground level down to about N.A.P. - 25 m, in longitudinal direction over the road-section, in which the test-sites are situated, has been made.

3.2. Subsoil description.

The original ground level of the test-sites decreases gradually from N.A.P.-1.40 m at test-site I to N.A.P.-1.45 m at test-site VI.
Below ground level the Holocene formation consists of a thin peaty top clay layer, resting on a peat layer with a thickness of approximately 4 m. These two layers belong both to the so called Holland-peat-formation. Between N.A.P.-5.5. m and N.A.P.-6.0 m this peat layer gradually changes into a clay formation belonging to the Calais-deposits. This clay formation can be subdivided in three layers, based on the nature of their mixtures. The thickness of each of these layers varies slightly from one test-site compared to another. At the top, between N.A.P. -5.80 m and N.A.P.-7.20 m the admixtures consist of remnants of vegetation. Between N.A.P.-7.20 and N.A.P.-8.80 m sandy enclosures are present. The lowest layer of this clay formation between N.A.P.-8.80 m and N.A.P.-10.50 m is peaty and contains shellfragments. Between N.A.P.-10.50 m and N.A.P.-10.80 m appears a thin layer of Base-peat, forming the conjunction to a sand layer, belonging to the Pleistocene formation, in which not only thin loam layers but also thin peat layers occur . Continuing downwards, between N.A.P.-14.00 m and N.A.P.-15.00 m the top-horizon of a clay formation can be distinguished, which contains thin sand layers to a depth of N.A.P.-18.00 m to N.A.P.-19.00 m and deeper down to approximately N.A.P.-23.00 m, contains much shells. Below this formation a dense sand formation occurs. The stratigraphy and structure of the compressible layers resting on top of the intermediate sand layer at approximately N.A.P.-10.80 m has been recorded in table 2. In this table also some laboratory test results, from tests performed on undisturbed samples from the distinctive layers, are given. Based on the results of the Dutch Cone Penetration Tests, the continuous borings and the laboratory test results, it could be concluded that there is little difference in structure and nature of the compressible layers between the different test sites.

3.3. Groundwater tables.

The phreatic level in the polder in which the test-sites are located, is maintained at N.A.P.-1.58 m. With the aid of open-piezometers the phreatic level in the intermediate sand layer has been established and declines from N.A.P.-1.30 m at test-site I to N.A.P.-1.60 m at test-site VI.
During ample time before the start of construction works and applying the fill, it has been established with the aid of piezometers, that the groundwater pressure in the compressive layers declines almost hydrostatically.

4. CONSTRUCTION OF THE TEST-SITES.

At the location of the test-sites a sand fill with a total thickness of 5.80 m had to be put in place. Following the installation of the settlement plates in mid-July 1979, a base-layer of drainage-sand with a thickness of 0.80 m has been applied. This base-layer has been dry-filled in two layers of 0.40 m.
Using this sand layer as a working platform, the vertical drains, reaching down to approximately N.A.P.-11.00 m i.e. reaching down into the intermediate sandlayer, have been installed in September 1979. For all drain types a mutual distance, centre to centre, of 2.50 m has been chosen for all test-sites. In practice the drains have been installed on cross-lines. The distance between the cross-lines is 2.50 m and the distance centre to centre of the drains on the cross-line is also 2.50 m. The prefabricated drains have been installed using a vibrating shaft with an outside cross-section of 150x50 mm^2. The conventional sand-drain has been constructed using wash-boring techniques. Following the original time-schedule for construction, the remaining sand fill with a thickness of 5.0 m would have been applied in 5 stages in layers with a thickness of 1.0 m each, immediately after the installation of the drains. However, problems with the supply of sand have caused a delay of approximately 60 weeks in applying the first stage (stage II) of the remaining fill on top of the base-layer (stage I). In November 1980 the application of the fill (stage II) started by dry-filling methods, working

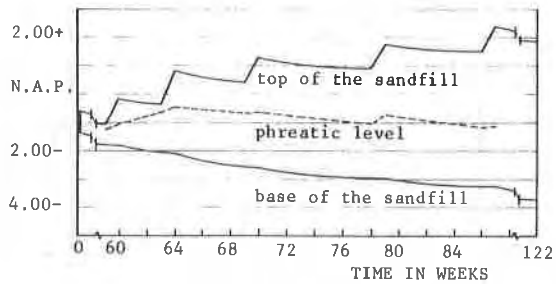


figure 3. Diagram of the application of the fill

in a direction from test-site VI to test-site I. The capacity of sand supply and transport was such that as an average two test-sites could be filled over a thickness of 1.0 m in 5 working days. In order to secure the stability of the side-slopes of the fill, a rest period was obeyed between the application stages of minimal 4 weeks (between stage II and stage III), increasing with two weeks for every following stage. Schematically the following filling schedule has been performed:

stage II 60 weeks after stage I
stage III 4 weeks after stage II
stage IV 6 weeks after stage III
stage V 9 weeks after stage IV
stage VI 8 weeks after stage V

The applied filling-schedule resulted in a loading scheme as shown in figure 3. In this loading scheme the recorded settlements of the fill and the phreatic level in the fill, measured between the application of the filling stages, are also given. The application of the fill was completed in May 1981. Hence the application of the remaining fill of the stages II-VI lasted for each test-site a period of 28 weeks.

5. MEASUREMENTS AND INTERPRETATION OF RESULTS.

5.1. Frequency of measurements.

During the execution of the work, the settlements of the fill and the phreatic levels have consistently been measured at the beginning and at the end of each working week. During the rest period, between the completion of stage I and the start of stage II, as well as in the period one month after completion of the fill, the frequency of measuring has been reduced to once per month.

Piezometer measurements and pore pressure meter readings have been performed with a frequency of three times per week, except for the rest period between stage I and stage II. These readings will be continued with a frequency of once per two weeks until and including the execution of the pavement construction in August 1982.

5.2. Porewater pressures.

During the rest period of approximately 60 weeks after the application of the drainage base layer, the compressive layers have had more than enough opportunity to consolidate under the applied load of 13 to 15 kN/m^2 of this base layer. No settlement increments, nor further decrements of porewater pressures, have been recorded during the last weeks of the rest period. Just before the application of stage II, almost hydrostatic groundwater pressures have been measured in the compressive layers, at all test-sites, with a phreatic level of N.A.P.-1.20 m in the drainage base layer and a phreatic level of N.A.P.-1.90 m in the intermediate sand layer. The hydrostatic water pressure line, mentioned above has been used as a reference (time-set t=0) for the recorded excess pore pressures during and after the application of the fill in stages II up to stage VI. The filling-schedule, given in figure 3 for the stages II to VI corresponds, due to the recorded settlements and phreatic levels, with a single load increase of 83 kN/m^2 applied at once at time t=0.

With the aid of piezometer readings and pore water pressure meter readings, the excess pore pressure-curve has been recorded as a function of the depth in the compressive layers, immediately after the completion of the fill at stage VI (t_1= 196 days) and also 90 days (t_2= 286 days) and 244 days (t_3= 440)days after completion date. Figure 4, for example, illustrates the curve of schematised porewater pressures in test-site IV. The points A to E in this figure are the recorded excess porewater pressures of the pore pressure meters of the same character, placed in the centre of gravity of the triangle in which the drains are installed.

Based on the open-piezometer readings, it is expected that the porewater pressure curve in the compressive layers will become stable at the end of the hydrodynamic period (t_e), resulting in an almost hydrostatic pressure line, with a phreatic level of N.A.P.-1.10 m in the fill and a phreatic level of N.A.P.-1.80 m in the intermediate sand layer.

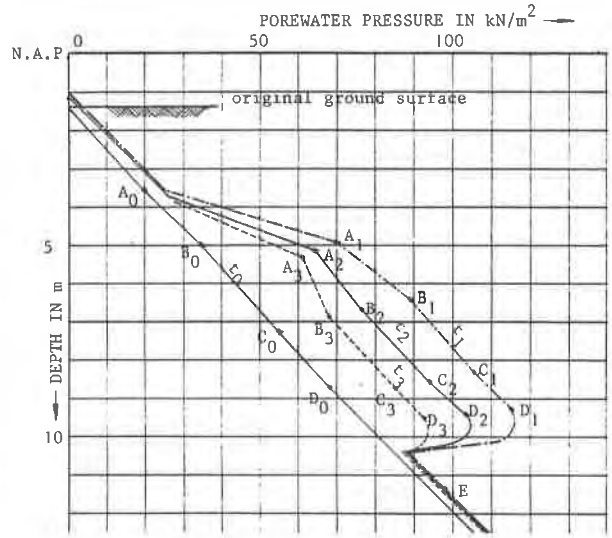


Figure 4. Porewater pressure curves of test-site IV at t_0, t_1, t_2 and t_3.

The excess pore pressures for the pore pressure meters A and C recorded at t_1, t_2 and t_3, can be expressed in percentages of the maximum excess pore pressure (which would have appeared theoretically), compared to the expected final hydrostatic pressure (see table 3). By plotting these proportional excess pore pressures as a function of time, it is possible to obtain a graphical impression of the progress of dissipation of the excess pore pressures (figures 5 and 6). These figures show clearly that for the peat-layer as well as for the clay layer, significant differences in dissipation of excess pore pressures occur for the different types of drains. Until today (January 1982), the dissipation of excess pore pressures progressed best, as expected, at test site VI, in which the conventional sand drains have been installed. The dissipation of excess pore pressures progresses very slowly at test-site II and falls clearly behind compared to the remaining test-sites.

Table 3. Excess pore pressures at different time-points (meters A and C).

test site	excess pore pressures in kN/m^2 and % at t_1=196 days, t_2=286 days and t_3=440 days											
	peat-layer						clay-layer					
	t_1		t_2		t_3		t_1		t_2		t_3	
	kN/m^2	%	kN/m^2	%	kN/m^2	%	kN/m^2	%	kN/m^2	%	kN/m^2	%
I	42.8	54.8	35.2	45.2	27.5	35.3	49.5	63.5	35.1	45.0	23.4	30.0
II	42.4	54.5	38.9	50.0	36.1	46.4	45.9	58.9	41.1	52.8	35.1	45.1
III	31.0	39.7	23.6	30.2	17.0	21.8	34.3	43.8	22.6	28.8	12.8	17.0
IV	31.6	40.2	24.0	30.5	19.0	24.1	35.9	46.0	22.4	28.8	11.7	15.0
V	36.2	47.8	30.1	39.6	26.0	34.2	42.4	55.0	35.5	46.1	29.4	38.2
VI	22.4	29.5	7.8	10.3	3.4	4.5	25.3	32.8	12.2	15.8	7.8	10.1

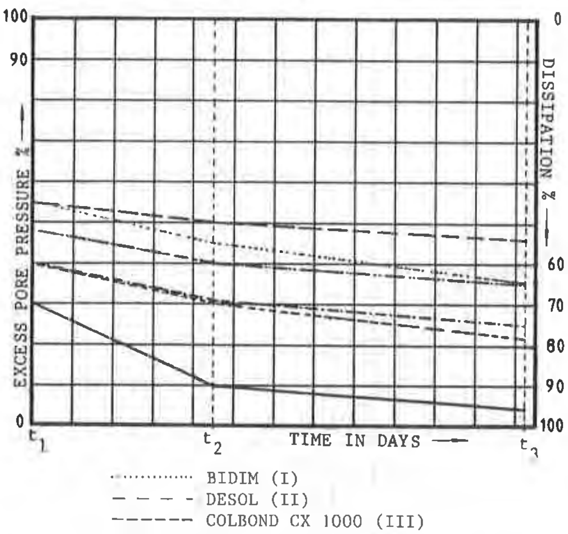


figure 5. Excess pore pressure in peat-layer.

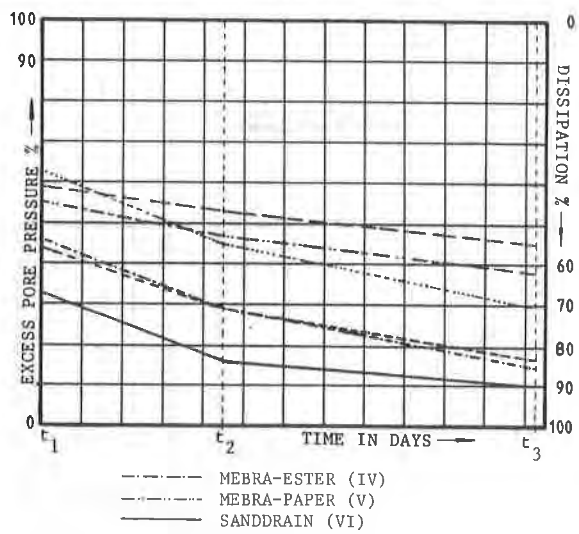


figure 6. Excess pore presssure in clay-layer.

5.3. Development of the settlements.

When the properties of compressive clay- and peat-layers, as determined in the laboratory, are compared to each other, it appears that there is only little difference between the test-sites for the distinctive layers. The variation in the expected total settlements of the different test-sites has mainly to be explained by the mutual differences in thickness of the peat- and clay-layers per test-site. As can be seen from table 4, in which the thickness of the different layers per test-site have been recorded, it appears that mainly the stratigraphy of the subsoil at the test-sites IV and VI differs from those in other test-sites. The thickness of the peat-layer at test site IV and the thickness of the clay-layer at test-site VI are smaller compared to the other test-sites, so that less settlements can be expected in these two test-sites.

Based on the soil-parameters, obtained from laboratory tests, a prognosis has been made of the total settlement for each test-site. For the test-sites IV and VI a total settlement of the compressive layers, resting on the intermediate sand layer, of approximately 3.25 m is expected and for the other test-sites the total settlment has been estimated to be between 3.40 m and 3.50 m.

The compression of the deeper clay-layers below the intermediate sand layer has been estimated to be between 0.10 m and 0.15 m for all test-sites.

In figure 7 the average of recorded settlments has been plotted for each test-site, covering the period between t_0=0 (just before the application of the fill in stage II) and t_3=440 days. The final settlements of the previously applied base-layer, stage I, achieved at t_0=0, as well as the settlements, referred to original grade, at time-points t_2= 286 days and t_3= 440 days (end of January 1982) are tabulated in table 4. In this table

Table 3. Summary of settlement data.

test site	thickness of compressive layers (m)		settlement prognosis	average of recorded settlements at time-points t_0, t_2 and t_3(m)			settlement at t_3 as a percentage of the
	peat	clay	(m)	t_0=0	t_2=286 days	t_3=440 days	settlement prognosis (%)
I	3.90	5.05	3.40	0.47	2.33	2.48 ± 0.10	73.0
II	3.85	5.25	3.45	0.46	2.13	2.22 ± 0.12	64.5
III	4.10	4.75	3.45	0.43	2.57	2.67 ± 0.15	77.5
IV	3.60	5.15	3.25	0.49	2.45	2.55 ± 0.12	78.5
V	4.10	4.50	3.40	0.50	2.10	2.20 ± 0.09	64.5
VI	3.90	4.30	3.25	0.53	2.48	2.57 ± 0.15	79.0

the settlements recorded in January 1982, are also expressed in a percentage of the total expected settlements.

By comparing the achieved settlments, i.e. the development of the settlements as a function of time, to the settlement prognosis, it appears that the test-sites III, IV and VI give the best proportional results, followed by those of test-site I. The results of test-sites II and V are almost identical and fall clearly behind.

The spread of the recorded settlements for each test-site varies from one test-site to another between 0.10 m and 0.15 m. Taking this variance into account, it can be concluded that, up to January 1982, the test-site III, IV and VI have settled 0.50 m to 0.55 m more and test-site I has settled 0.30 m more than the test-sites II and V.

6. CONCLUSIONS.

Based on the results of in situ measurements combined with soil mechanical data and calculations, it has to be concluded that the established differences in behaviour in practice of the different test-sites can only be explained by the better or worse functioning of the different drain-types.

Further analyses of the different measurements have to indicate which proporties of the drainmaterials or drains have resulted to the observed differences in behaviour in practice.

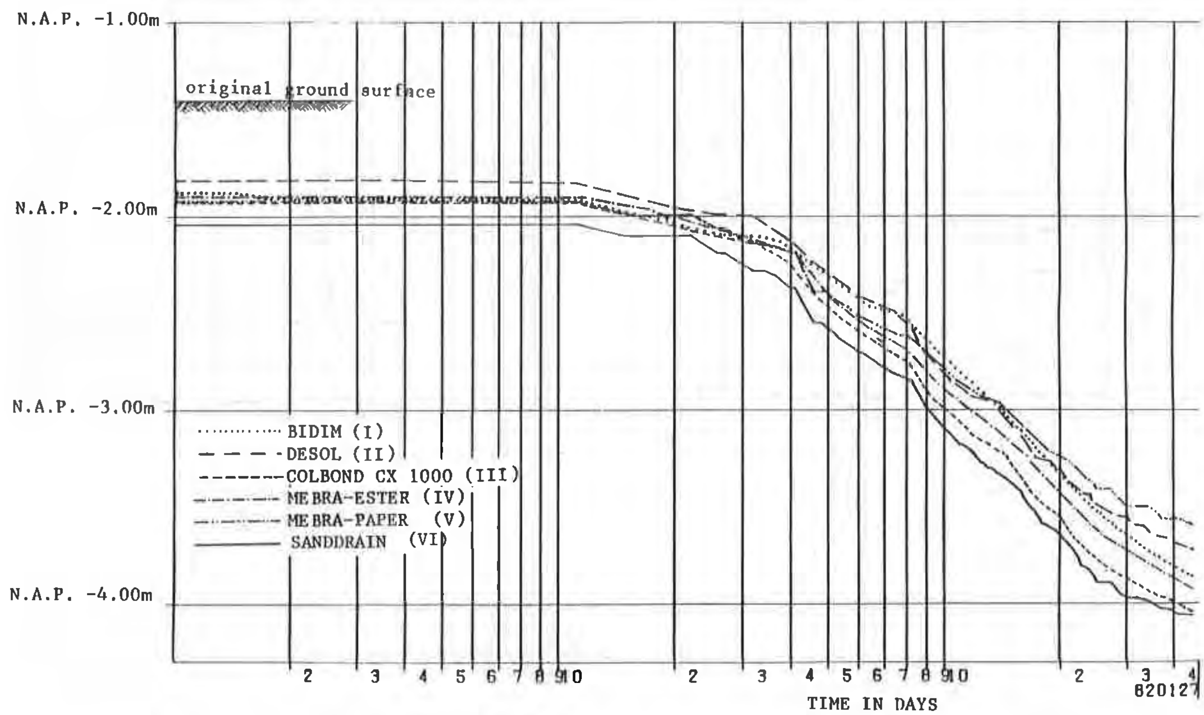


figure 7. Development of the settlements.

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Standard Test of Permittivity and Application of Darcy's Formula
Essai standard de permittivité et respect de la "loi" de Darcy

Permeability measurements in direction normal and parallel to plane of geotextiles under compression were performed with needle-punched, spunbonded and woven fabrics The obtained data coupled to measured structural parameters of the geotextiles indicated that their permeability behavior is affected only slightly with a greater variation in their transmissivity than in their permittivity. On the other hand their filtration behavior is expected to be more altered because of the very large decrease in the distance between the fibres. Finally a relationship was developped between the permeability coefficient and structural parameters of geotextiles.

Un important programme d'essais de perméabilité a été effectué sur non tissés aiguilletés et thermoliés, et tissés ; la perméabilité est mesurée sur géotextiles comprimés, soit normalement au plan de la nappe textile, soit suivant ce plan. Nous avons corrélé la perméabilité mesurée aux paramètres de structure du géotextile. On notera, par ailleurs, que la permittivité varie relativement peu avec la compression, mais qu'il n'en est pas de même pour la transmissivité. Enfin, le comportement en filtration est sans doute beaucoup plus sensible à la compression du fait de la diminution importante de la distance entre les fibres.

INTRODUCTION

Even though geotextiles have been used extensively in civil engineering work, very little is known about their behavior under compression. In applications such as storage area, dams, vertical drains, embankments and others, synthetic fabrics are under very large static loads that indeed might change their structure. While these applications are recognised to be important, most of the published research works were performed with geotextiles under atmospheric conditions or submitted to very low compression level. In this study, permeability's measurements were performed under compression level ranging in value from 0 to 2000 kPa.

The compressibility of a geotextile may be expressed as a variation of its porosity n (function of the normal stress σ_N applied normally on the fabric). For a geotextile of thickness b, constituted of fibres with a unique diameter D, and having a known mass per unit area m_s, the porosity (ratio of void volume on total volume) can be estimated from

$$n = 1 - m_s/\rho_s b \qquad (1)$$

where ρ_s is the fibre's polymer density

As shown on figure 1a and 1b, the change in the thickness and the porosity of compressed geotextiles is a function of the different structures. In this case, the mass per unit area value is reported as 200g/m2 (1) for comparison purpose of the following fabrics : tx a woven polyester ; BD a needle-punched continuous fibres polyester ; GSM and PR needle-punched short fibres polyester ; TP a polypropylene spunbonded.

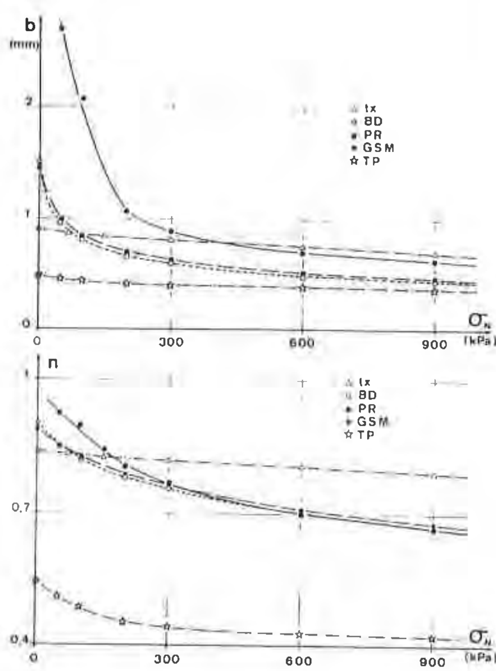

Fig. 1 : Thickness and porosity variations for geotextiles of varius structures (m_s = 200 g/m²).

$\lambda^* = 2.g.i_e.d_p/V_p^2 \qquad R_e^* = V_p.d_p/\nu \qquad (3)$

régime laminaire : $\lambda^* = A/R_e^*$ ($R_e^* < 2000$)

régime turbulent : $\lambda^* = B$ ($R_e^* > 500\,000$)

$A = 64$ B, variable suivant la rugosité des parois

Dans le cas des conduites présentant des coudes et des variations de section (r correction de section non circulaire ; diamètre moyen $\overline{d}_p = d_H$ diamètre hydraulique) :

$\lambda^* = (A/r)/R_e^* + B$ pour tout R_e^* (4)

soit $i_e = (\nu/g).(A/2r\ \overline{d}_p^2).V_p + (B/2g\ \overline{d}_p).V_p^2$ (5)

Donc, même en régime laminaire (pas de turbulence), la linéarité entre i_e et V_p ne constitue qu'une approximation, d'autant plus juste que R_e^* est faible (loi de Poiseuille).

B. ECOULEMENT EN MILIEU POREUX :

Les pores intersticiels sont assimilables à des tubes de direction et de section variables. Considérons le milieu poreux homogène et isotrope obtenu à partir d'un empilement de grains.

Définition de nouveaux paramètres d'écoulement : le milieu granulaire est défini par sa porosité n et son diamètre hydraulique $d_H = 4.D_e/f.n/(1-n) = \overline{d}_p$ (6) avec f coefficient de forme des grains et D_e leur diamètre équivalent (2). Mais il n'apparaît pas possible de définir λ^* et R_e^*, car si l'on considère le pore intersticiel moyen, sa longueur Le ainsi que le débit unitaire V_p du fluide au travers ne sont pas mesurables.

On assimilera donc le vide intersticiel à un ensemble de pores de même volume relatif (même n) parallèles à la direction moyenne d'écoulement et de diamètre hydraulique d_H (fig. 2). Le débit unitaire V_n de ces pores est tel que :

$V_n = 1/n.Q/S \qquad L/V_n = L_e/V_p \qquad (7)$

et le nouveau gradient caractéristique $i = \Delta H/L$

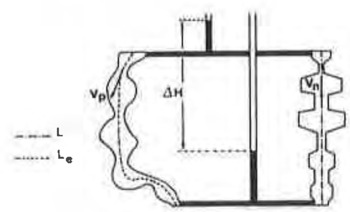

fig. 2 : Modélisation du milieu poreux

Nous définissons donc de nouveaux paramètres λ et R_e pour ce modèle équivalent :

$$\lambda = \frac{2.g.i.\overline{d}_p}{V_n^2} \quad \text{et} \quad R_e = \frac{V_n.\overline{d}_p}{\nu} \qquad (8)$$

et en posant $t = (L/L_e)^2$ tortuosité du pore moyen

$\lambda = \lambda^*.t^{-3/2} \qquad \text{et} \qquad R_e = R_e^*.t^{+1/2} \qquad (9)$

La relation (4) appliquée à l'ensemble du vide intersticiel devient :

$\lambda = (A/rt)/R_e + B.t^{-3/2} \qquad (10)$

Cette relation est indépendante du fluide de mesure, mais alors que la relation entre λ^* et R_e^* (4) était quasi-indépendante du tube considéré (à la rugosité de paroi près), puisque A et B sont des constantes, la relation entre λ et R_e (10) dépend de la structure géométrique des pores (r et t variables, mais dans de faibles proportions cependant).

Nombre de Reynolds critique : ces nouvelles définitions de λ et R_e nous ont permis de présenter sous un jour nouveau des résultats obtenus par différents auteurs et de regrouper de façon très synthétique les valeurs obtenues sur divers milieux poreux et plusieurs fluides (figure 3). Nous en avons tiré des conclusions générales sur l'application de la "loi" de Darcy :

(a) correspond à un ensemble de billes de même diamètre (n = 0.345, f = 6,cf(4)) ; (b) correspond à un sable lache d'angularité moyenne (n = 0.270, f = 7,5,cf(5)) et (c) et (d) correspondant à un arrangement de cylindres régulièrement espacés (entraxe 1.25 D) parallèles entre eux et perpendiculaires à la direction moyenne d'écoulement (n = 0.497, f = 4,cf(6)), les cylindres étant soit en ligne (c), soit en quinconce (d).

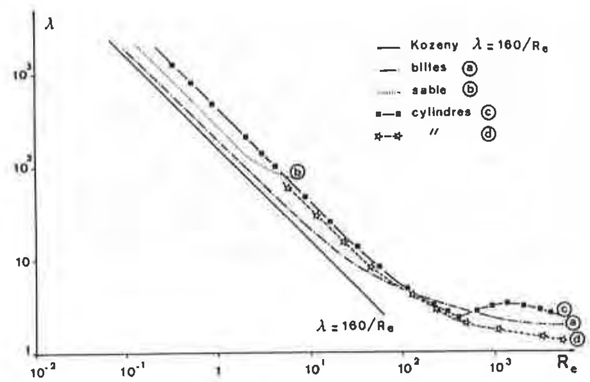

fig. 3 : Loi d'écoulement pour différents milieux poreux

Chaque courbe expérimentale de la figure 3 présente une partie gauche quasi-rectiligne qui correspond au domaine d'application de la "loi" de Darcy ;

$\lambda = a/R_e \qquad V = (g/\nu).(2n\ \overline{d}_p/a).i \qquad (11)$

On définira un nombre de Reynolds critique R_{ec} au-dessus duquel cette approximation n'est plus acceptable (le régime turbulent n'apparaitra que pour un nombre de Reynolds nettement supérieur à R_{ec}). R_{ec} sera fonction de la précision de mesure recherchée.

* $R_e < R_{ec}$: malgré des différences géométriques importantes, les courbes (a), (b), (c), (d) sont très regroupées. Théoriquement, d'après (10), on a $\lambda = (A/rt)/R_e$ avec A= 64.

Pour un sable rond à granulométrie étroite ou un ensemble de billes de même diamètre, CARMAN-KOZENY proposent rt = 2/5

soit $\lambda = 160/R_e \qquad V = (g/\nu).(n\ \overline{d}_p^2/80).i \qquad (12)$

qui est effectivement en bon accord avec (a).

Pour l'ensemble de cylindres parallèles, qui n'est pas sans analogie avec la structure d'un géotextile, on préfèrera :

$\lambda = 350/R_e \qquad V = (g/\nu).(n\ \overline{d}_p^2/175).i \qquad (13)$

ce qui correspond à rt = 1/(5.5)

* $R_e > R_{ec}$: la "loi" de Darcy n'est plus acceptable pour un R_{ec} d'autant plus faible que la structure géométrique des pores présentera de singularités : $R_{ec} > 100$ pour les cylindres (c), (d) ; $R_{ec} > 10$ pour les billes empilées aléatoirement et $R_{ec} > 1$ pour un sable d'angularité moyenne. Les géotextiles constitueront un cas intermédiaire.

C. ECOULEMENT AU TRAVERS D'UN GEOTEXTILE

L'utilisation des quatre paramètres présentés au chapitre III nous a permis de couvrir un grand domaine pour les nombres de Reynolds : nous présentons (fig. 4) les résultats obtenus sur quatre géotextiles non tissés non comprimés, deux thermoliés TER 230 (Terram 2000, n = 0,79) et TP 270 (Typar 3807, n = 0,54) et deux aiguilletés BD 280 (Bidim U34, n = 0,92) et SDC 370 (Sodoca AS 400, n = 0,86). Les conditions d'obtention des résultats seront détaillées dans le chapitre III, pour le TP 270 et le BD 280. On constate une bonne corrélation, pour un même géotextile entre les résultats obtenus sur les quatre perméamètres :

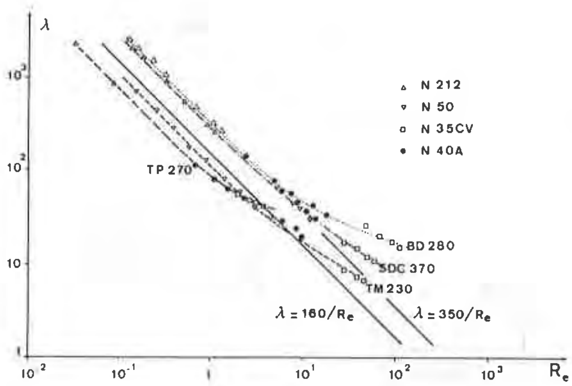

fig. 4 : Corrélation entre les résultats obtenus sur 4 perméamètres différents pour plusieurs géotextiles

* $R_e < R_{ec}$: pour les aiguilletés, la courbe est pratiquement confondue avec celle obtenue pour les cylindres parallèles. On admettra donc qu'ils suivent une loi du type (13) :

$$\lambda = 350/R_e \qquad V = (g/\nu).(n\,\overline{d}_p^2/175).i \qquad (13')$$

avec $\overline{d}_p = d_H$

L'écart entre les thermoliés et les aiguilletés a été justifié (2) par une sous-estimation de D_e, donc de d_H (6), les fibres étant souvent soudées entre elles, dans le cas des thermoliés. λ et R_e sont alors difficiles à estimer correctement.

* $R_e > R_{ec}$ pour les aiguilletés, R_{ec} est bien inférieur à celui constaté pour les cylindres parallèles (fig. 3) alors que leur comportement était du même type dans le domaine de la "loi" de Darcy : en fait les pores ont une forme beaucoup plus complexe dans les cas des non tissés et les singularités de l'écoulement sont d'autant plus nombreuses.

Quand à l'écart entre les deux aiguilletés, il peut s'expliquer par l'hétérogénéité des nappes ou la précision discutable du perméamètre (N 35CV) utilisé pour obtenir ces résultats (chapitre III).

R_e étant lié à la précision de mesure souhaitée, l'étude présentée en III permet de proposer :

$$R_{ec} = 5 \text{ pour BD 280} \qquad R_{ec} = 30 \text{ pour TP 270} \qquad (14)$$

si l'on considère admissible une sous-estimation de 50 % de la permittivité. Soit, avec D diamètre des fibres, et (6) et (7), on détermine

$$\text{avec } R_e = Vn.\overline{d}_p/\nu = (V.D)/(1-n)\nu \qquad (15)$$

une limitation du débit unitaire :

$$V = Q/S < R_{ec}.\nu.(1-n)/D \qquad (16)$$

III. ETUDE EXPERIMENTALE SUR QUATRE PERMEAMETRES :

A. NOTIONS DE PERMEABILITE ET PERMITTIVITE :

La relation générale (10) est équivalente à :

$$i = V/K + V^2/K_t^2 \qquad (17)$$

$$K = (g/\nu).(2nrt\ \overline{d}_p^2/A) \text{ perméabilité de Darcy} \qquad (18)$$

$$K_t = g.(2n^2t^{1.5}\overline{d}_p/B) \text{ perméabilité turbulente} \qquad (19)$$

Pour $R_e < R_{ec}$, on appliquera la "loi" de Darcy ($i = V/K$) tandis que pour $R_e \gg R_{ec}$, on considérera ($i = V^2/K_t^2$). Certains auteurs (Ogink) ont choisi comme paramètre, K_t, en se plaçant systématiquement à $R_e \gg R_{ec}$, mais cette condition entraine des vitesses d'écoulement fortes, susceptibles de modifier la structure du géotextile. Nous avons donc choisi K, ce qui impose $R_e < R_{ec}$.

A la différence de K_t, K est fonction de la viscosité du fluide ν. Pour faciliter la comparaison des résultats obtenus avec l'air et l'eau, nous utiliserons la perméabilité intrinsèque indépendante du fluide (18) :

$$k = K.(\nu/g) = K_{air}.\nu_{air}/g = K_{eau}.\nu_{eau}/g \qquad (20)$$

Dans la plupart des expériences, l'épaisseur de géotextile (L = N.b, empilement de N nappes d'épaisseur b) n'est pas mesurée et l'on introduira la notion de permittivité intrinsèque pour une nappe de géotextile : (1)

$$(k/b) = (Q/S).(\nu/g).(N/\Delta H) \qquad (21)$$

Les résultats expérimentaux sont donc présentés dans les axes (Q.ν/S.g) et (ΔH/N) ; (k/b) est inversement proportionnel à la pente de la droite de permittivité (fig.8).

B. PRESENTATION DES QUATRE PERMEAMETRES :

Perméamètre oedométrique à eau (N 212) : (fig. 5). Il s'agit du perméamètre utilisé par ailleurs (2) pour la détermination de la perméabilité normale sous compression (σ_N=0 dans le cas présent). Cet appareillage de recherche n'est pas conçu pour des tests de contrôle et d'identification, mais ses mesures serviront de référence pour les autres résultats, une fois vérifié (7) que l'empilement ne modifie pas la valeur mesurée pour la permittivité d'une nappe unique.

- La mesure se fait sur un empilement de N nappes.
- La section normale à l'écoulement S = 35 300 mm^2.
- Le fluide est de l'eau désaérée.
- Q est mesuré par débimètre à bille.
- ΔH est mesuré par différence entre la tête et le pied de l'échantillon en faisant ainsi abstraction des pertes de charge ΔH_a de l'appareillage.

On constate (fig. 6a et b) que la "loi" de Darcy est bien vérifiée, mais les débits mesurés sont très faibles (R_e < 0.1 pour TP 270 et R_e < 1 pour BD 280). Remarquons que le thermolié est 9,5 fois moins permittif que l'aiguilleté, pour des masses surfaciques équivalentes.

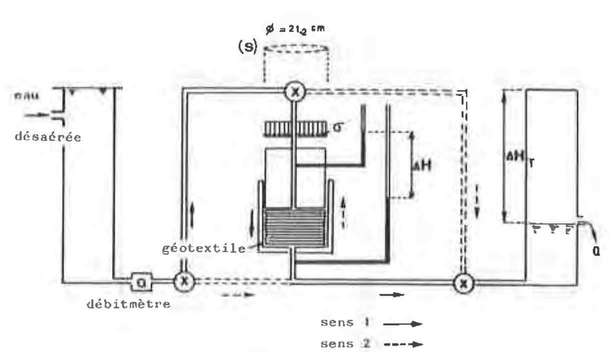

fig. 5 : Perméamètre normal N212

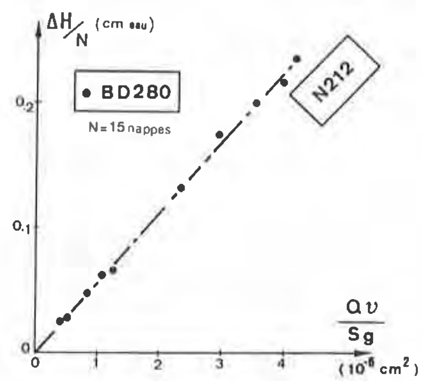

fig. 6a : Essai au perméamètre N212 sur BD280

Tube de perméabilité à eau (N 50) : (fig. 7). La cellule de permittivité est branchée sur le même circuit de mesure que le N212, mais la section S = 1960 mm² et beaucoup plus faible, ce qui permet d'atteindre des débits Q suffisamment élevés pour dépasser le domaine d'application de la "loi" de Darcy. ΔH est mesuré entre les points 1 et 2 (ΔH_a de l'appareillage entre 1 et 2 est négligeable.

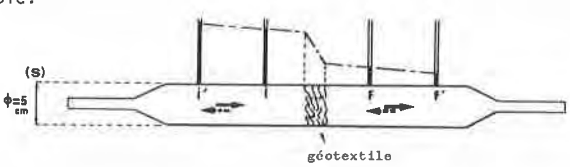

fig. 7 : Perméamètre normal N50

On définira une erreur E sur la permittivité par rapport aux mesures du N 212. Il s'agit d'une estimation par défaut de (k/b) à même perte de charge :

$$\frac{(k/b)N50}{(k/b)N212} = \frac{Q_{N50}\ (\Delta H)}{Q_{N212}(\Delta H)} = 1 - E \qquad (22)$$

Dans le cas du BD 280 (fig. 8), pour un débit cinq fois supérieur au débit max. du N212, l'erreur E sur la permittivité est de 20 %.

Perméamètre à air (N40A) : (fig. 9)
- La mesure se fait sur un échantillon de section S = 1260 mm².
- Le fluide est de l'air.
- Q est mesuré par débitmètre à bille.
- On mesure ΔH_T entre les points 1 et 2 situés à 8.5 cm en amont et en aval du plan moyen du géotextile et ΔHa (mesuré à vide pour le même Q) n'est pas négligeable du fait des variations de section de la conduite : $\Delta H = \Delta H_T - \Delta H_a$ (fig. 10).

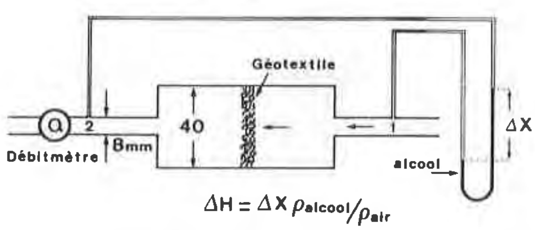

fig. 9 : Perméamètre normal à air N40A

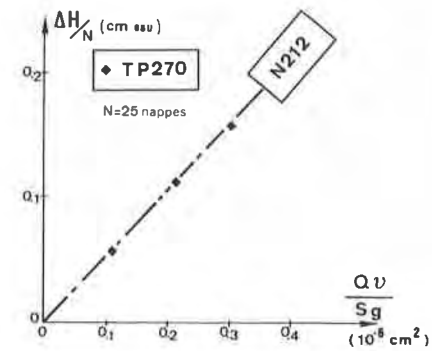

fig. 6b : Essai au perméamètre N212 sur TP270

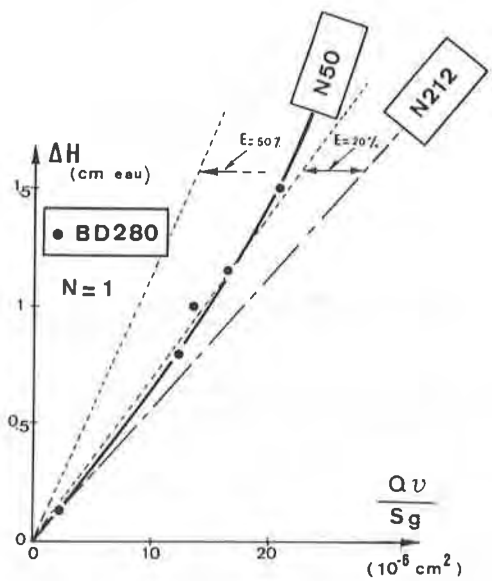

fig. 8 : Essai au perméamètre N50 sur BD280

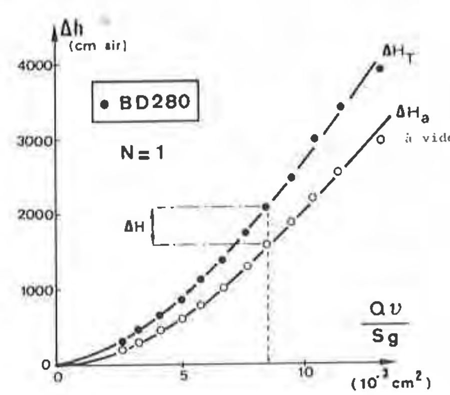

fig. 10 : Pertes de charge avec et sans géotextile (perméamètre N40A).

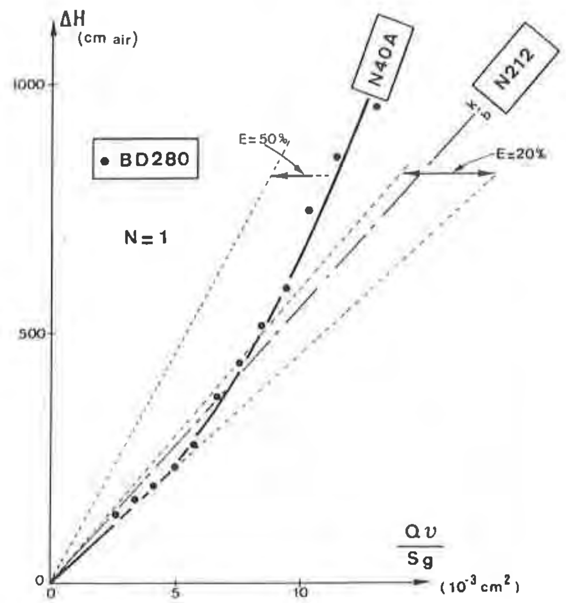

fig. 11 : Essai au perméamètre N40A sur BD280

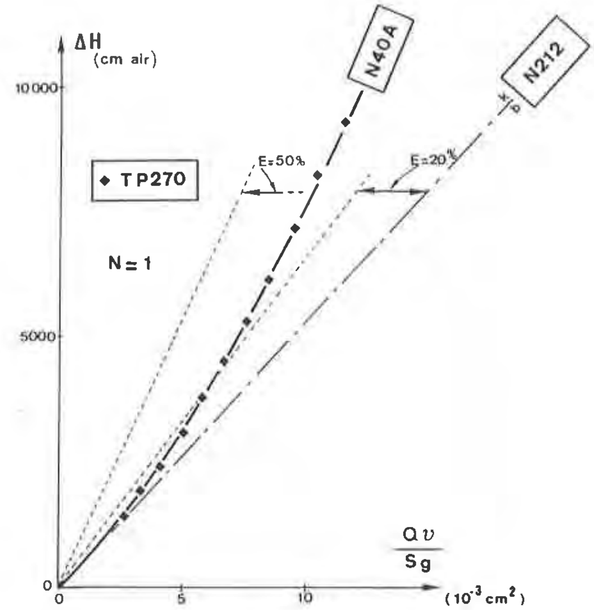

fig. 12 : Essai au perméamètre N40A sur TP270

Pour le BD 280 (fig. 11) et de faibles débits, (k/b) mesuré est supérieur à la valeur du N212, ce qui s'explique certainement par une surestimation de ΔH_a : la viscosité dynamique $\mu = \nu.\rho$ de l'air est beaucoup plus faible que pour l'eau et l'amortissement des turbulences est donc d'autant plus faible. ΔH_a mesuré sur l'appareil à vide sera modifié par la présence du géotextile.

Par ailleurs le défaut de cet appareil est de présenter des ΔH_a trop forts vis à vis des ΔH produits par des géotextiles très permittifs. C'est le cas du BD 280, mais pour le TP 270 (fig. 12) beaucoup moins permittif, une erreur sur ΔH_a n'est pas sensible sur (k/b).

Perméamètre à eau à charge variable (N 35CV) : (fig. 13). Cet appareil est couramment utilisé pour les géotextiles : on mesure le temps de vidange entre les niveaux 1 et 10 distants de 500 mm La section S du géotextile est de 960 mm²

Le régime est non permanent, débit et charge variant continuement pendant l'essai, à la différence des trois perméamètres précédents. Si l'on considère le régime comme pseudo-permanent entre deux niveaux j et j+1 écartés de 5 cm, l'estimation de la permittivité entre les temps t_j et t_j+1 correspondants est donnée par :

$$\frac{k}{b}(t) = \frac{\nu}{g} \cdot \frac{1}{S} \frac{Q(t)}{\Delta H(t)} \qquad (23)$$

Soit Q_j^{j+1} le débit moyen, $\Delta H_{T_j}^{j+1}$ la perte de charge moyenne (= différence de niveau amont-aval). Pour le même Q_j^{j+1}, on mesure $\Delta H_{a_j}^{j+1}$ sans géotextile. Ce qui nous permet d'obtenir pour chaque essai 10 couples de valeurs

$$(Q_j^{j+1}, \Delta H_j^{j+1} = \Delta H_{T_j}^{j+1} - \Delta H_{a_j}^{j+1}).$$

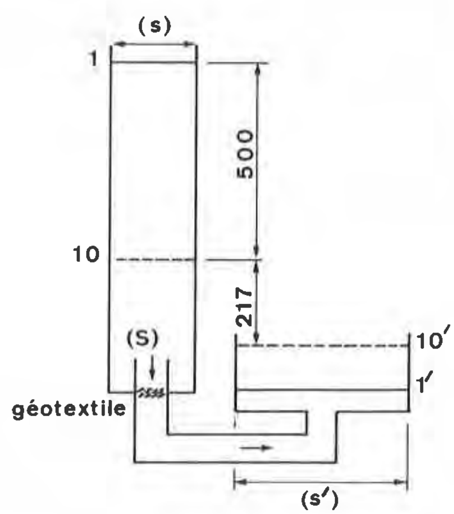

fig. 13 : Perméamètre normal à charge variable N35CV

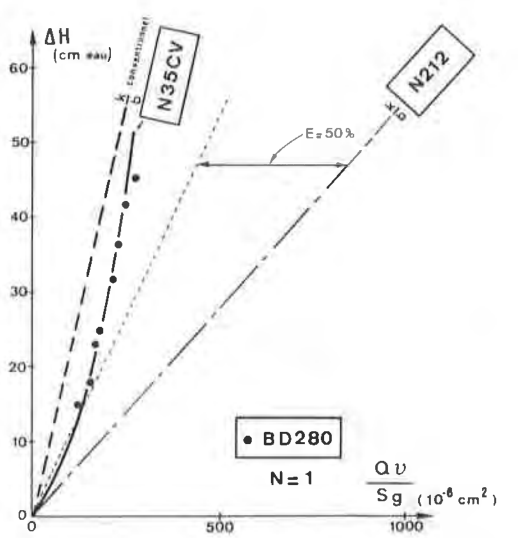

fig. 14 : Essai au perméamètre N35CV sur BD280

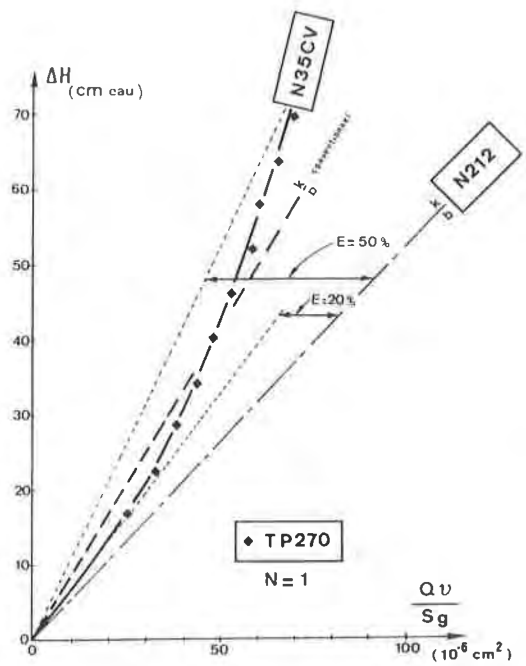

fig. 15 : Essai au perméamètre N35 CV sur TP270

On constate que pour les deux géotextiles (fig. 14 et 15), l'écart par rapport à la "loi" de Darcy est important pour tout $\Delta H_T > 15$cm : $R_e > 1.5$ pour TP270 et $R_e > 45$ pour BD 280 d'après la figure 4. Ce perméamètre ne convient donc absolument pas pour la gamme de permittivité des géotextiles.

Nous avons aussi noté la valeur conventionnelle de (k/b) correspondant à l'interprétation classique de cet essai : la "loi" de Darcy est supposée vérifiée et ΔH_T est pris égal à ΔH. Par intégration de (23) entre j = 1 et j = 10, on obtient :

$$\left(\frac{k}{b}\right)_{conv.} = \frac{\nu}{g} \cdot \frac{s'}{s} \cdot \frac{s}{s'+s} \quad \mathrm{Log}\, \frac{(\Delta H_T)_1}{(\Delta H_T)_{10}} \cdot \frac{1}{t_{10}-t_1} \qquad (24)$$

$$(k/b)_{conv} = \beta/(t_{10}-t_1) \qquad (25)$$

L'erreur E ainsi faite est de 37 % pour le TP 270 peu permittif et de 75 % pour le BD 280.

IV. CONCLUSIONS :

L'évaluation de la permittivité est correcte si elle est indépendante de (Q/S) ou ΔH. ceci suppose acceptable la "loi" de Darcy, dans les conditions de mesure. Nous avons montré (II) que le domaine d'application de la "loi" de Darcy pouvait être borné par un nombre de Reynolds maximal R_{ec}, fonction de la précision demandée. Malheureusement R_{ec} n'est pas totalement un paramètre indépendant du géotextile (II.C) : en fonction des résultats obtenus sur quatre non tissés et en considérant que les tissés présentent d'après les études sur cylindres (fig. 3 (c) et (d)) un R_{ec} supérieur, nous pensons pouvoir garantir une précision de 50 % sur la permittivité en prenant R_{ec} = 5 pour tout géotextile.

Ceci correspond à une limitation du débit unitaire Q/S au travers de la nappe, fonction du géotextile, mais en prenant des conditions assez extrèmes, la formule (16) donne avec l'eau pour fluide : n = 0.92   D = 40 µm

$V = Q/S < 1$ cm/s

Pour de nombreux perméamètres (III) cette condition ne peut être respectée car la charge minimale imposée ΔH est trop forte. La solution proposée consiste à empiler un nombre N d'échantillons jusqu'à ramener Q/S à une valeur acceptable : $Q/S = (k/b).(g/\nu)\Delta H.\frac{1}{N}$.

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Geotextile Soil Drainage in Siphon or in Siphon-Capillarity Conditions

Drainage à l'aide des géotextiles travaillant comme siphon ou siphon capillaire

In some particular circumstances where the soil drainage is foreseen some geotextile types may perform in siphon or in auto-initiated siphon flow following the capillary rise. The consitions are discussed necessary to establish siphon function and the experimental method used is described.
The first testing results shows that the investigations should continue and used in drain designing with geotextile.

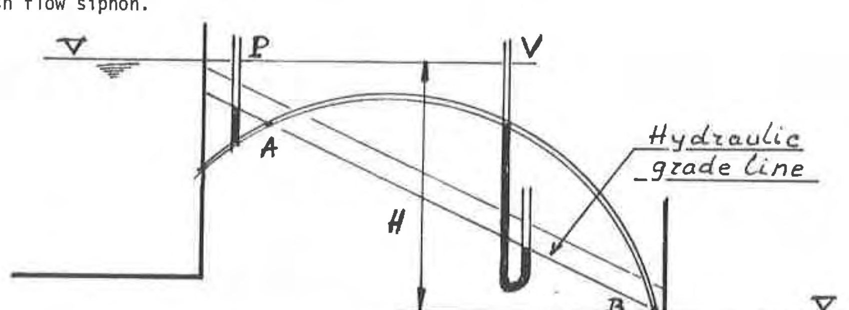
Dans certains cas particuliers où le drainage du sol est prévu, certains types de géotextiles peuvent travailler en siphon ou en siphon auto-amorçable grâce à la remontée capillaire.
Les conditions de siphonage sont discutées et le dispositif expérimental est décrit.
Les premiers résultats d'essais montrent que la recherche devra être poursuivie et exploitée dans le dimensionnement de drainage en siphon.

1. INTRODUCTION

Three principal functions of geotextiles have been clearly described by J.P. GIROUD in his valuable paper untitled : Design with geotextiles {1}. There are geotextile drains, geotextile filters and geotextile separators.
To perform any of these functions, a geotextile must have some mechanical resistance less or more in level depending on environnement forces balance. Nevertheless in some particular circumstances it would be useful to see a geotextile drain performs as siphon and especially as an auto-re-establish flow siphon.

2. SIPHON FUNCTION

It would perhaps be convenient to explain the siphon function shortly at this stage.
The figure 1 showsa water reservoir fed by another reservoir through a pipe, but the pipe joining them rise above the hydraulic grade line. So in this case, there will be a vacuum between the points A-B. The levels of the water pressure at P and of the vacuum at V are shown at this figure 1. But should the pipe crack at any point between A-B, air will be sucked in and the flow will stop.



$$H = H_{entry} + H_f + H_{exit}$$

$$H = \frac{0.5V^2}{2g} + \frac{4fLV^2}{2gD} + \frac{V^2}{2g}$$

fig. 1.

For the matter of that the flow will also stop when the lover reservoir is free of water and when the air pressure at the pipe end will be higher than the water max pressure in the pipe. But in some particular circumstances the flow in the pipe may be re-established or auto-re-established. In any way to re-establish the water flow, the water pressure in the pipe must become higher than air pressure. To do it, a pump for exemple can be used. But the capillary rise may also help to re-establish the water flow. This is the geotextile drain siphon function. We will see that some geotextiles and especially some types of needle punched nonwovens fullfilled the conditions to perform the siphon function.

3. DRAIN FUNCTION

It is now very well known that the application of man-made synthetic fabrics in civil engineering to solve geotechnical or hydro-geotechnical problems have incrased rapidly in last years.
As more theoretical knowledge and practical experience are gained on geotextile behaviour in soil wider developpement of its applications will result. Should a geotextile perform as auto-re-established flow siphon, it must at first perform well as geotextile drain.
In order to fullfil this first condition a geotextile must exhibit a high transmissivity.
When uniform (Darcy's) flow is considered, the drain discharge Q (m^3/s) is :

$$Q = A\ k_d\ t\ \frac{\Delta h}{L} \qquad (1)$$

where
L (m) and A (m) are respectively the length of the geotextile drain in the flow and in perpendicular to the flow direction;
t (m) is the geotextile drain thickness;
k_d (m/s) is its permeability; and Δh (m) is the hydraulic head loss.
If Δz (m) and Δp (Pa) are respectively levels and pressure differences between geotextile drain ends, the hydraulic head loss is :

$$\Delta h = \Delta z + \frac{\Delta p}{\rho_w\ g} \qquad (2)$$

where
$\gamma_w = \rho_w\ g$ is the specific weight of water that is the product of ρ_w (kg/m^3) the mass of water per volume unity and of gravity
$g = 9{,}8066\ m/sec^2$

Thus the transmissivity of a geotextile drain is

$$tk_d = \theta\ (m^2/s)$$

herefore the geotextile drain discharge per unity of its length perpendicular to the flow direction is directly proportional to its transmissivity and to hydraulic gradient :

$$\frac{Q}{A} = \theta\ i\ (m^2/s) \qquad (3)$$

Let us analyse now the work of this geotextile drain in siphon conditions following the soil settlement under an embankment.

The settlement proportions are exaggerated for the clearnece of description.
At the point A the geotextile drain is under vacuum and to establish a water discharge from the settlement area S-S the siphon flow must established.

4. DISCUSSION OF CAPILLARY RISE

The establishment of the siphon discharge through the geotextile drain is connected with its impregnation which may be accomplished by penetration or diffusion process or by both phenomenous acting simultaneously.
The force (N) normal to the capillary void cross section perpendicular to its length is :

$$F = 2\ \pi\ r\ T_s\ \cos\alpha \qquad (4)$$

where
r (m) is the radius of curvature of meniscus; in the capillary tube r may usually be taken as the radius of the tube;
T_s (N/m) surface tension of liquid; typical value for water is $7{,}5 \times 10^{-3}$ N/m;
liquid-solid contact angle.
Thus F may be, depending of value, negative or positive, and we have respectively down stream or rise in the capillary tube {3}.

In this respect the pressure (Pa) in the capillary tube is {2} :

$$P = 2\ T_s\ \cos\alpha\ /\ r \qquad (5)$$

inversely proportional to the meniscus radius.
For α small and $r = 10^{-6}$ m for exemple, the capillary pressure will be of $1{,}5 \times 10^{-3} \times 1\ /\ 10^{-6} = 15 \times 10^3$ Pa and the capillary rise of ~ 1,5 m results.
Regarding the capillary flow through Poiseuille's equation, we have :

$$Q = \frac{dv}{dt} = \frac{\pi\ r^4\ P}{8\ L\ \eta} \qquad (6)$$

where
P (Pa) is the active pressure calculated above
η (NS/m^2) the dynamic viscosity and
L (m) the capillary length.

Introducing (5) to (6) we get :

$$Q = \frac{\pi\ r^3\ T_s\ \cos\alpha}{4\ L\ \eta} \qquad (7)$$

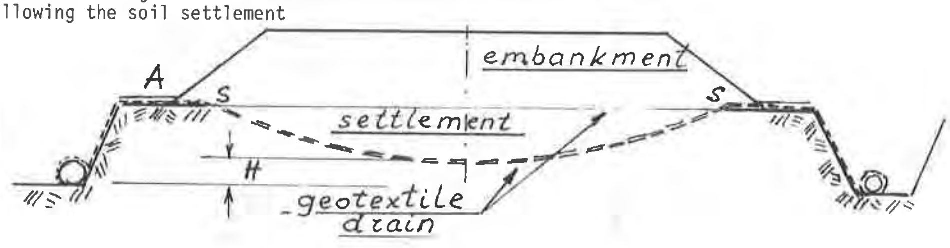
But capillary void is not straight so its tortuous form can be expressed by the ratio :

$$L_T = L_{actual}\ /\ L$$

Thus the number n of capillary voids of mean radius r per transvers cross section of geotextile drain caracterised by a porosity ε is :

$$n = \frac{\varepsilon}{L_T\ \pi\ r^2}$$

fig. 2.



and the discharge through this section is :

$$\frac{dV}{dt} = \frac{\varepsilon \; r \; T_s \cos \alpha}{4 \; L_T^2 \; L \; \eta} \qquad (8)$$

Therefore the impregnated volume per unity of transverse cross section is :

$$V^2 = \frac{\varepsilon \; r \; T_s \cos \alpha}{2 \; L_T \; \eta} \; t \qquad (9)$$

The validity of this simple model is restricted by the differences between r.
The FICK's second low describes the unidirectional diffusion :

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} \qquad (10)$$

where
c : concentration of diffused matter (water) at a distance x, at the time t; and
D : is the diffusion constant.
It is assumed that the water and its vapour diffusion may play a role in the geotextile drain flow auto-re-establishement but for the moment the part of this phenomen in the flow process can not be surelly evaluated.
Nevertheless some interesting models was bild for study the impregnation process by diffusion {4}.

6. TESTING ARRANGEMENT AND RESULTS

The capillary rise of water in soil is depending of its nature, grain finesse and voids ratio. So it can be from few mm to few m.
The capillary rise in needle punched nonwovens is small in order of few mm but this figure changes for wet soaking material. The water molecules adsorbe to the fibers producing many meniscus and reducing voids volume. Than by capillary rise with aid of diffusion process, the water partly impregnated geotextile helps to establish a water flow. The true process of this flow should be rather complex one but some figures can be reached using a simple testing arrangement showed at the figure 3.

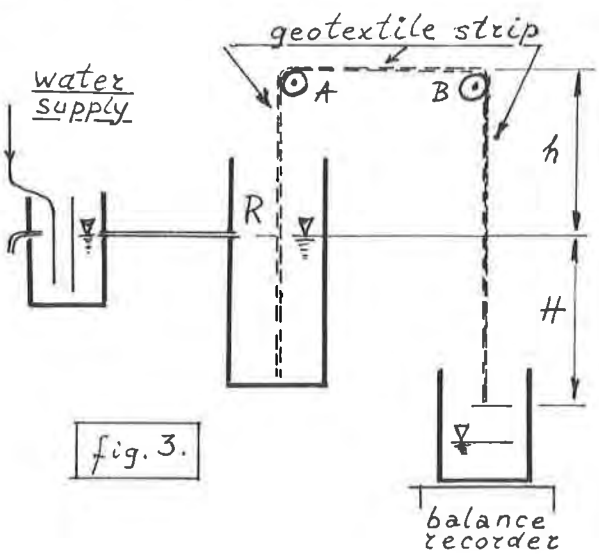


fig. 3.

A soaked wet strip of needle punched polyester nonwoven, 50 mm in wide (length perpendicular to the flow direction) is immersed in a constant level water reservoir (R), passed over two rollers (A)(B) and hangs down.
In view to avoid a drying of the strip the device is kept in a wet air (100 % R.H.).
Driving by the hydraulic head, H, the water flows through the strip and draps to the lover reservoir stayed on a balance recorder.

Thus, for a given type of geotextile, the strip width and the overhang, h, are kept constant.
Than the discharge is measured for some hydraulic heads H.
The results reached for some types of geotextiles are consigned in the Table 1.

6. CONCLUSIONS

This investigation shows that in some circumstances, some types of geotextiles can perform in a siphon function.
Other tests and a beter theoretical knowledge of the phenomenon of capillary rise are needed for designing.

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TABLE 1

Sample nr		Units	1	2	3	4	5
Mass per unit area		g/m^2	150	210	270	340	550
Porosity under :	0,5 kPa	%	93	92	91	91	91
	2 kPa	%	91	90	89	89	89
	200 kPa	%	82	81	81	81	81
Thickness under :	0,5 kPa	mm	1,5	1,9	2,3	2,8	4,4
	2 kPa	mm	1,5	1,9	2,2	2,7	4,0
	200 kPa	mm	0,6	0,8	1,05	1,3	2,1
Permeability (radial) in plane of geotextile under :	2 kPa	m/s	6×10^{-4}	6×10^{-4}	6×10^{-4}	6×10^{-4}	6×10^{-4}
	200 kPa	m/s	4×10^{-4}	4×10^{-4}	4×10^{-4}	4×10^{-4}	4×10^{-4}
Transmissivity (radial) under :	2 kPa	m^2/s	9×10^{-4}	$11,4 \times 10^{-4}$	$13,2 \times 10^{-4}$	$16,2 \times 10^{-4}$	24×10^{-4}
	200 kPa	m^2/s	$2,4 \times 10^{-4}$	$3,2 \times 10^{-4}$	$4,2 \times 10^{-4}$	$5,2 \times 10^{-4}$	$8,4 \times 10^{-4}$
Max force per unit width with 0,8 m width and 0,1 m lenght		kN/m	6,52	7,94	11,81	15,3	-
Discharge for H =	0,05 m	$10^{-6}\ m^3/s$	$0,113.10^{-6}$	$0,113.10^{-6}$	$0,272.10^{-6}$	$0,131.10^{-6}$	$0,186.10^{-6}$
	0,1 m	$10^{-6}\ m^3/s$	$0,185.10^{-6}$	$0,247.10^{-6}$	$0,439.10^{-6}$	$0,258.10^{-6}$	$0,319.10^{-6}$
	0,15 m	$10^{-6}\ m^3/s$	$0,237.10^{-6}$	$0,286.10^{-6}$	$0,528.10^{-6}$	$0,317.10^{-6}$	$0,400.10^{-6}$
	0,2 m	$10^{-6}\ m^3/s$	-	-	$0,556.10^{-6}$	-	$0,385.10^{-6}$

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Structural Permeability Law of Geotextiles

Loi structurale de perméabilité pour les géotextiles

The permittivity test is often used for identification and quality control of geotextiles. Many geotextiles are thin and very permeable. Consequently the velocity of flow during the test is very high and Darcy's formula is not applicable, as it is shown using general laws governing flow through porous media. However, a theoretical analysis shows that an approximate value of permittivity can be obtained in the test if the Reynolds'number is kept smaller than a certain value.

The theoretical analysis is used to interpret the results of an experimental study carried out using four permeameters (one with air and three with water). It is confirmed theoretically and experimentally, that air and water lead to consistent values of permittivity and it is concluded that the falling head water permeameter is not satisfactory.

L'essai de permittivité est fréquemment utilisé pour identifier ou contrôler les géotextiles. Mais de nombreux géotextiles présentent une résistance à l'écoulement si faible que les vitesses du fluide dans l'essai sont très fortes. Dans ces conditions, comme nous le montrons à partir des lois générales d'écoulement dans les milieux poreux, la "loi" de Darcy n'est pas vérifiée. On peut cependant garantir une certaine précision sur la valeur de permittivité mesurée à partir du critère proposé qui correspond à une limitation systématique du nombre de Reynolds du fluide en écoulement dans le perméamètre.

L'étude théorique s'appuie sur un travail expérimental comparatif sur quatre perméamètres, l'un utilisant l'air comme fluide et les trois autres l'eau. Cette étude permet de confirmer l'équivalence des résultats obtenus avec les deux fluides et permet aussi de rejeter l'utilisation du perméamètre à eau à charge variable, en raison des erreurs introduites sur la valeur de permittivité.

I. OBJECTIF :

Le test de permittivité sur géotextile non comprimé est un de ceux les plus couramment effectués sur ces produits. Il présente en effet un grand intérêt comme essai de contrôle ou d'identification car la valeur de la perte de charge du fluide s'écoulant normalement au plan de la nappe est très sensible aux variations de structure du géotextile.

A l'instigation du Comité Français des Géotextiles (1), nous avons entrepris une étude sur l'essai de permittivité : il s'agit de définir la relation entre le débit unitaire $V = Q/S$ de fluide (Q débit fluide à la pression atmosphérique et à 20°C, S section du milieu poreux, normalement à l'écoulement) et la perte de charge ΔH (diminution de potentiel à la traversée d'une épaisseur L de milieu poreux, exprimée en hauteur de fluide à la pression atmosphérique).

Il est couramment admis que la loi est linéaire ("loi expérimentale" de Darcy), pour les écoulements à faible nombre de Reynolds (2) :

$V = Q/S = (K/L).\Delta H$ K/L permittivité (1)

ou $V = Q/S = K.i$ K perméabilité au fluide (2)
$i = \Delta H/L$ gradient de décharge

Or il apparaît que les géotextiles peuvent présenter des épaisseurs et des porosités très différentes de l'un à l'autre et par conséquent des permittivités très différentes. Comme nous le montrerons ci-dessous, il sera alors difficile de définir un perméamètre à la fois simple et convenable (c.à.d. permettant de vérifier la "loi" de Darcy) pour l'ensemble des géotextiles.

L'étude présentée comprend deux parties, une partie théorique où l'on adapte aux géotextiles la théorie des écoulements en milieu poreux et une partie expérimentale où l'on a comparé les résultats obtenus pour deux géotextiles non tissés avec quatre perméamètres différents.

II. LOI GENERALE D'ECOULEMENT EN MILIEU POREUX :

A. ECOULEMENT DANS UN TUBE :

Pour un tube droit à section circulaire (diamètre d_p) de longueur L_e et un fluide de débit unitaire V_p (viscosité cinématique ν), on a $i_e = \Delta H/L_e$ et on trouve une relation unique entre les paramètres adimensionnels λ^* et R_e^* (3) : (fig. 1)

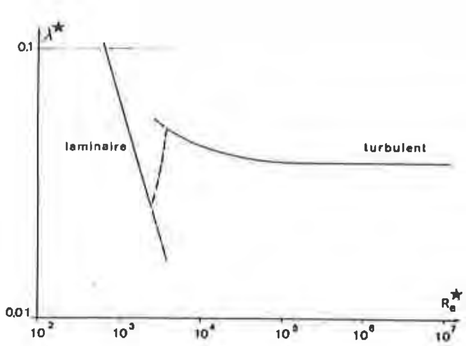

fig. 1 : Ecoulement dans un tube

Even under a very large compression level, the porosity values are higher than the porosity of soils. Also it can be observed that the relative decrease of porosity for the non-woven needle-punched fabrics is lower than the thickness's decrease. This is accordance with Kolb (2) that calculated, from a theoretical analysis, the minimum porosity of non-woven fabrics (to be egual to 45 %).

Because of the importance to understand the hydraulic behavior of geotextiles of different structures under compression, a study was performed to measure permeabilities using two permeameters designed at IRIGM (Grenoble) : normal and lateral permeabilities, K_N and K_p, under pressures ranging from 0 to 2000 kPa. These results are presented in this paper and are completed by a morphologic analysis of the compressed fabrics using the Image Analyser technique developped at Ecole Polytechnique of Montréal. The aim is to develop a correlation between the permeability behavior of fabrics under compression and characteristic structural parameters.

I. PERMEABILITY BEHAVIOR UNDER COMPRESSION

A. TEST APPARATUS

Normal permeability

A permeameter designed to measure the normal permeability, K_N, is shematicaly presented on figure 2. It is constituted of a large number of accessories and measuring instruments. The essential item is an inox cylinder containing a pile of geotextile's samples sanwdwiched between two distributors. A piston is free to move inside the cylinder to compress the installed samples to the compression level.

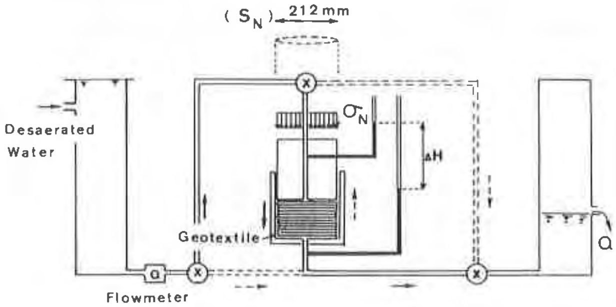

Fig. 2 : Permeameter (N 212) to measure normal permeability of geotextiles.

City water is desaerated in a vacuum tank and then stored in a second tank connected to the cylinder. Rotameters and control valves are installed to regulate the fluid's flow (permanent regime) while manometers are recording the head loss (ΔH) through the samples such that the head loss of the apparatus are not taken in account. Instruments are installed to measure static stress applied on samples (σ_N), the thickness of the pile of N samples (Nb), the water temperature (θ) and the constant fluid flow rate (Q_θ). Each sample of 212 mm in diameter are installed in the cylinder (S_N= 35300 mm^2) to a maximum height of 150 mm. We believe that the total area offer to flow, N S_N, is very representative.

A mechanical set up enables to apply pressure on the samples in a range of values from 0 to 2000 kPa and the flow of water through the samples, Q/S_N, are fixed to values small enough to insure flow conditions at which Darcy's law can be applied (3). Under these fixed water flow rates, the Darcy's velocity can be defined as $V_D = Q/S$ and the permeability coefficient can be calculated from the following equation

$$Q/S_N = K_N . \Delta H/Nb \qquad (2)$$

where Q is the water flow rate at 20°C and can be calculated from the measured flow rate, Q_θ, at θ°C.

Permeability in the plane

A second permeameter was used to measure the permeability within the plane of the fabric (see figure 3). This item is replacing the cylinder in the already described system. The test is performed with a pile of N squared samples, 200 mm x 200 mm, such that two measurements can be performed on the samples in the warp and fill directions (L_1 and L_2). The permeability coefficient, K_p, can be found from the following expression

$$Q/S_p = K_p . \Delta H/L_1 \qquad (3)$$

where S_p = Nb.L_2

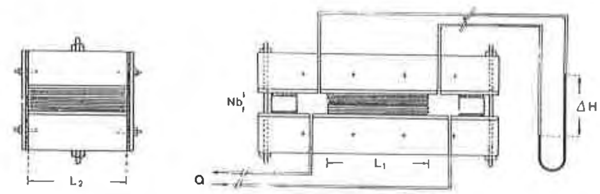

Fig. 3 : Permeameter to measure permeability in the plane of geotextiles.

B. THEORY

Capillary tubings

In this approach, an analysis identical to the well known analysis of granular media (1) will be used for the flow through geotextiles. The velocity, V_p, inside a straight capillary tube of diameter, d, and of length, L, of a fluid when under laminar flow regime (small Reynolds'number) can be expressed as

$$V_p = 1/32.g/\nu.d^2 i \qquad (4)$$

where i = ΔH/L is the hydraulic gradient and ν is the cinematic viscosity.
The flow of a fluid through a granular medium is analog through flow inside capillary tubings as first pointed out by Kozeny (4). The well known Poiseuille's law, equation (4), was then applied to geotextile even through the cross section as well as the diameter of the intersticial capillaries are not constant.

If L is defined as the distance between two, normal sections, S, of a fabric relatively to the direction of the fluid flow, then the average effective length of a pore size between the fibres should be greater than the distance between these two sections (L_e > L). Consequently the effective hydraulic gradient must be defined as

$$i_\varepsilon = \Delta H/L_e \qquad (5)$$

Following Poiseuille's law but substituting for the effective length, equation (4) becomes

$$V_p = 1/32.g/\nu.d^2.\Delta H/L_e \qquad (6)$$

Unfortunatly the cross section of intersticial pores is not circular such that a correcting factor, r, can be used and an average diameter, $\bar{d}_p$, must be utilized

$$V_p = r/32.g/\nu.(\bar{d}_p)^2 \Delta H/L_e \qquad (7)$$

Because of the difficulty to measure the mean diameter of pores, Kozeny substituted that value by the hydraulic diameter, $\bar{d}_p = d_H$, for the entire granular medium.

$$d_H = 4/A_s . n/(1-n) \qquad (8)$$

where A_s is the specific area defined as

$$A_s = 4/D \text{ for a uni fibre fabric} \qquad (9)$$

for a multi fibres fabric

$$A_s = 4\,\frac{\sum_i \frac{P_i}{\rho_s^i D_i}}{\sum_i \frac{P_i}{\rho_s^i}} = \frac{4}{D_e} \qquad (10)$$

where D_e is the equivalent diameter, P_i is the percent weight of fibres of diameter D_i and density ρ_s^i and $d_H = D_e.n/1-n$.

Then the velocity of the fluid in a direction parallel to the mean flow path between two fabric sections separate by a distance L is

$$V_n = V_p.\ L/L_e \qquad (11)$$

and $V_n = r/32.g/\nu.(\bar{d}_p)^2.(L/L_e)^2.\Delta H/L$ (12)

where the tortuosity is defined as $t = (L/L_e)^2$

Knowing that $V_D = Q/S = n.\ V_n$ (Darcy Velocity)

then equation (12) can be expressed as

$$Q/S = n.rt/32.g/\nu(\bar{d}p)^2.i \qquad (13)$$

Using Darcy's law, the permeability coefficent can be expressed as

$$K = n.rt/32.g/\nu\ (\bar{d}p)^2 \qquad (14)$$

Following Carman suggestion (4), rt = 2/5 for granular media

$$K = g/\nu.n/80.(\bar{d}p)^2 \qquad (15)$$

or $K = g/\nu.D_e{}^2/80.n^3/(1-n)^2$ (figure 4) (16)

More, Lord (5) showed, experimentally, that the expression representing flow of fluid through compressed plugs of fibres that had porosities n > 0,75, is

$$K = \frac{g}{\nu}\cdot\frac{D_e^2}{17,72}\cdot\frac{n^5}{(1-n)^{1,32}}$$

Single cylinder

Another approach to analyse flow of fluid through synthetic fabric consists in studying the flow around a single cylinder or fibre. This can be applied only if the mean distance between the fibres is large enough such that the fibres are not influencing the flow around each fibre. In another study, Rollin and al (6) have considered the case of bundle of cylinders.

For a flat cross section of geotextile S_N of a mass per unit area, m_s, the total length of a fibre, l, of diameter D and polymer density, ρ_s, is

$$l = m_s.S_N/\rho_s.4/\pi D^2 \qquad (18)$$

For the fibres all enligned in the plane of the fabric and for flow of a fluid normal to that plane

$$V_D = Q/S_N$$

Using the drag coefficient, C_D, around a single cylinder, the drag force, F_D, can be expressed as

$$F_D = C_D.\rho_w/2.(V_D)^2.(l.D) \qquad (19)$$

Knowing that this drag force can be expressed as

$$F_D = \rho_w.g.\Delta H\ S_N \qquad (20)$$

The following expression for the normal permeability can be found

$$K_N = \pi/2.\rho_s.g/m_s\ .\ b.D/C_D V_D \qquad (21)$$

or

$$K_N = \frac{g}{\nu}\cdot\frac{\pi}{2}\cdot\frac{D^2}{1-n}\cdot\frac{1}{C_D R_e^g} \qquad (22)$$

where $R_e^g = V_D.D/\nu$

Obtained data are presented on figure 4 for flow regime equivalent to $R_e^g < 1$ $(C_D(R_e^g) = 10)$. It can be observed that the determined permeability coefficients using the capillary tubings and single fibre are in agreement only for the highest porosity values.

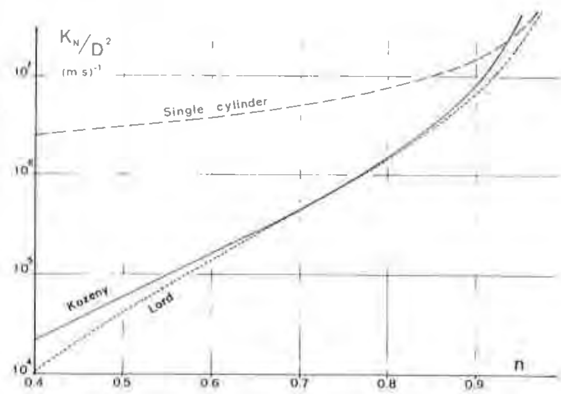

Fig. 4 : Proposed laws of permeability of geotextiles.

C. EXPERIMENTAL

Many different structured geotextiles have been analysed and their normal and lateral permeabilities measured : needle-punched continuous polyester fibres (BD = Bidim ; SDC = Sodoca), needle-punched short polyester fibres (SM : Sommer ; TXL = Texel) and spunbonded fabrics (TP = Typar ; TER = Terram ; LTR = Lutravil) and also woven fabrics (tx = TRI X ; ts = ts26) and multilayers. The mass per unit area of each fabric can be found from the utilized trade name as for example TP = 270 is a Typar product of 270 g/m².

The obtained results presented on figures 5 and 6 are compared with Kozeny's analysis expressed as

$$K/D_e^2 = \frac{1}{80}\cdot\frac{g}{\nu}\cdot\frac{n^3}{(1-n)^2} \qquad (23)$$

to delete the fibre's diameter influence.

Fibre's diameter :

On figure 5 it can be observe that the permeability coefficient varies as a function of $D_e{}^2$ for needle-punched geotextiles of equivalent mass per unit area but using different fibre's diameters.

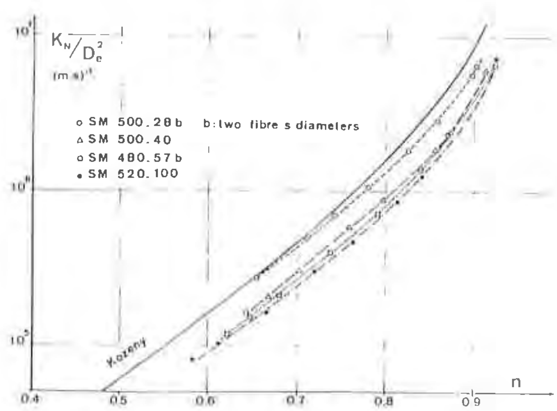

Fig. 5 : Influence of fibres diameter on geotextiles permeability.

Fibre's structure :

Also it can be observed on figure 6 that fabrics with identical structure followed the same analytical model for identical porosity. Needle-punched fabrics permeability measurements are closed to the predicted curve from Kozeny analysis while the woven fabrics are less permeable and the spunbonded more permeable (approximately 2,5 greater than for needle-punched fabrics) for fixed porosity and diameter.

The difference between the spunbonded and the needle-punched fabrics can be explained from the analysis of their structures as already pictured by Rollin et al (7). In fact most of the fibres of spunbonded product are gathered together by group of two or three fibres such that the equivalent diameter D_e must be greater than the used fibre's diameter (photo 1).

Finally it can be observed, from the presented data, that the mass for unit area (refer to TXL-1500 and BD-280) and the fibre's polymer (refer to BD and SDC fabrics) do not influenced the proposed analytical models.

The analysis of needle-punched fabrics manufacture with varying the needle's penetration and also the speed of the machine indicated a change in the thickness and also of the porosity but the measured permeabilites of theses fabrics can be correlated using the same proposed models (1).

Anisotropy :

To learn about the isotropy of fabrics, one should compare on figures 7 and 8 the measured normal and in the plane permeabilities under compression. It can be observed that the permeability in the plane coefficients are greater than the normal permeability coefficients even under large compression levels. On the other hand, the lateral permeability coefficients are decreasing faster with pressure than the normal coefficients indicating an inverse anisotropy when the compression level is increasing.

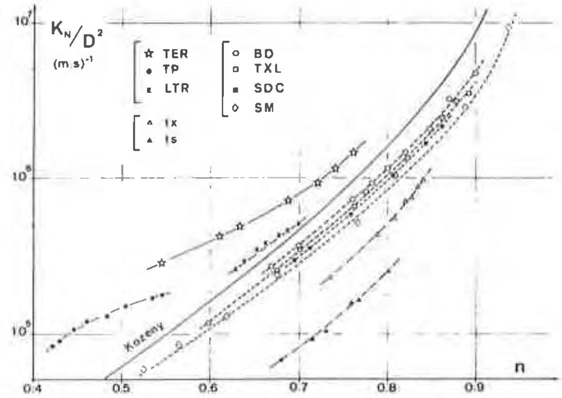

Fig. 6 : Permeability variations with the porosity and the structures of the geotextile

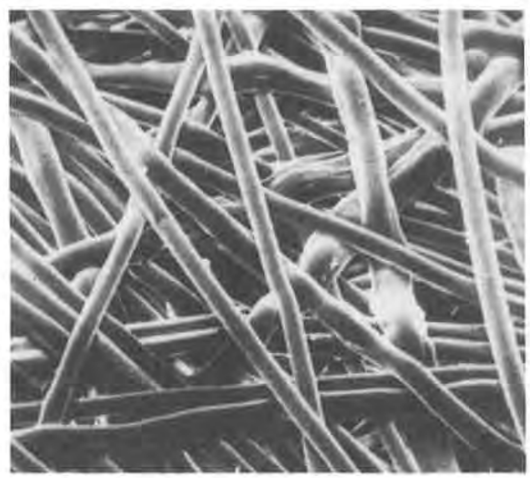

Photo 1 : Macro-photo of a TP270 sample (I.T.F.)

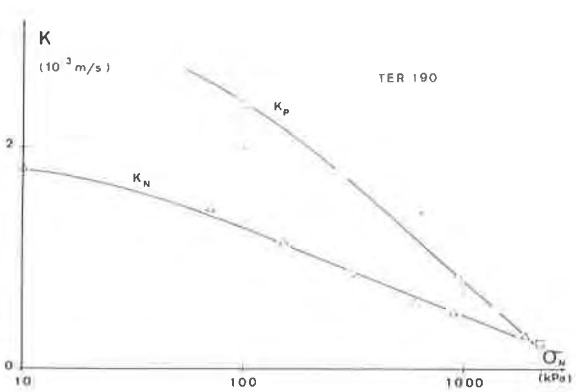

Fig. 7 : Anisotropy of permeability (spunbonded nonwoven TER 190).

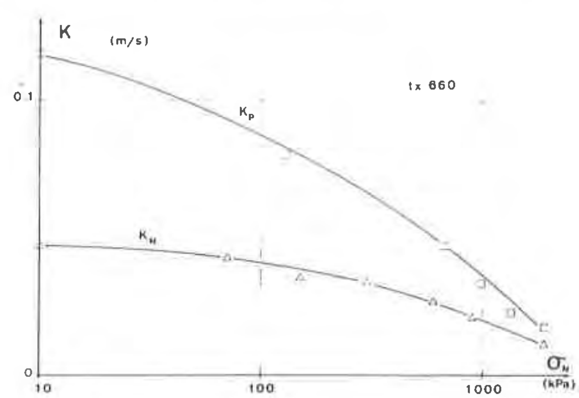

Fig. 8 : Anisotropy of permeability (woven tx 660).

Multilayer's fabric : (fig. 9 and 10) :

A study of the influence on the multilayers' fabrics was done by comparing permeability measurements on individual layer and also on the composite fabric. Comparing results obtained on individual GSM-700 draining layer, on GSM 700 with one filtering layer GSM 300 and also of a fabric constituted of a GSM-700 layer sandwished in between two GSM-300 layers it was found that under manufacturing process each layer's structure is altered by the needle-punching action producing the multi layers's fabric. The correlations to use, if each layer is not altered, are :

transmittivity $K_p \left(\sum_1^3 bi\right) = \sum_1^3 \left(K_p^i\, bi\right)$ (24)

permittivity $1/\left(K_N/\sum_1^3 bi\right) = \sum_1^3 \left(1/K_N/bi\right)$ (25)

The result obtained by adding layers is satisfactory for the permittivity but wrong for predicting transmissivity.

The permittivity of the multilayers'fabric is a function of the layers with less permittivity but the transmittivity of the fabric is controled by the layers with greater values. These latest layers are more affected by the needle-punched action of getting together the layers.

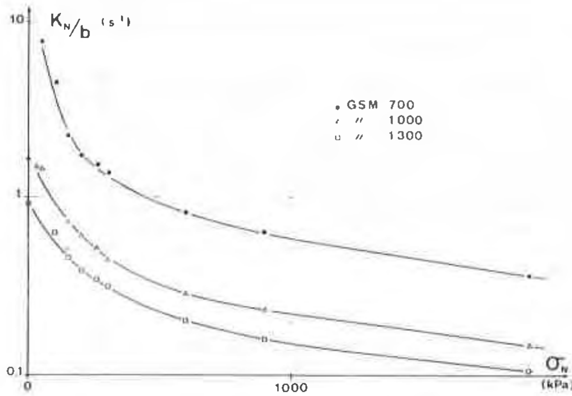

Fig. 9 : Permittivity of multilayer geotextiles under compression

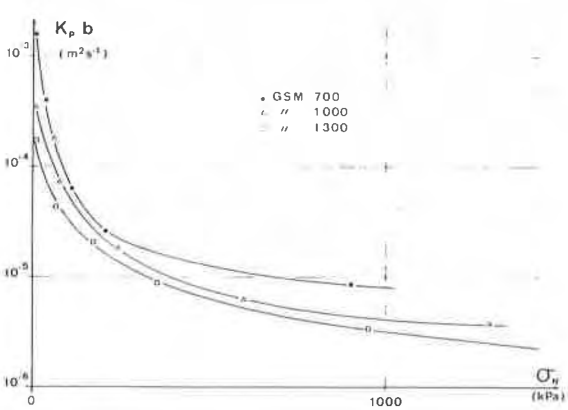

Fig. 10 : Transmissivity of multilayer geotextiles under compression

Summary

The measured normal and lateral permeabilities of geotextiles under compression are of interest to engineers that need to estimate their permeability behavior when installed under a civil engineering work. The permittivity, K_N/b, will vary only slightly with pressure compare to a greater decrease in the transmissivity, Kp.b. But it should be keep in mind that the presence of soil particles are surely influencing this behavior in actual applications.

A second interest is to learn about geotextiles' structure under compression and so to learn about their filtration behavior. Using the capillary's tubings analysis, an average pore diameter can be estimated (15) by the following expression

$$\overline{d_p^*} = \sqrt{\nu/g.80/n.K} \qquad (26)$$

In a following step, the morphologic analysis of the fabrics was performed and the measured mean distance between fibres compared to estimated mean pore diameter calculated from permeability coefficients using equation (26).

II. STRUCTURE ANALYSIS

A. IMPREGNATION TECHNIQUE

In order to analyse the structure of choosen geotextiles, each sample must be encapsulated under compression in a resin block. The technique to encapsulate fabric's sample under atmospheric condition was developped by Masounave et al (8) and has already been used extensively. The encapsulation of a sample is more complicated such that a new apparatus was designed at E.P.M.

A sample, impregnated with a very fluid resin, is inserted into a Teflon coated cylinder. The bottom of cylinder and of the piston head are designed to insure evacuation of excess resin as the sample is compressed. A mechanical system insure to lock in place the piston during the cooling period and the resin's recipe is defined to insure transparency of the resin submitted to large static pressures.

Each sample was encapsulated and then analysed with an Image Analyser. The fibre's diameter, the porosity, the thickness and the histogram of the distance between fibres were measured under pressures varying from 0 to 1000 kPa

B. CHOICE OF SAMPLES

For each geotextile, six samples were choosen to be impregnated. As shown on figure 11 the mass per unit area of needle-punched fabrics obtained from 100 samples of 100 cm³ is varying from a sample to another. In fact the variances of histograms were found to be so large that before choosing the samples to be impregnated the mass per unit area, m_s, histograms were determined. Samples

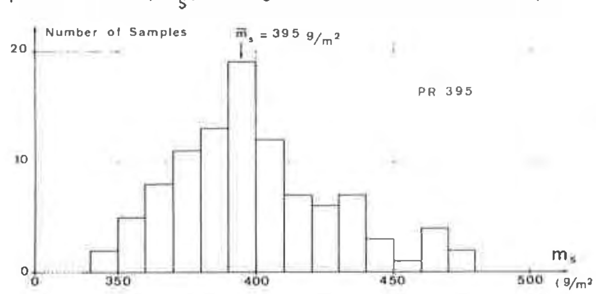

Fig. 11 : Choice of samples to structure analysis.

with identical mass per unit area and corresponding to the mean value were then choosen to insure satisfaction comparison of structure.

The analysis of spunbonded fabrics was achieved by encapsulating a pile of samples to overcome the small thickness of each sample. The analysis of a pile of samples in a manner similar to thick needle-punched non woven fabrics. This is a new technique that will permit to analyse their fabrics and perhaps defined structural parameters responsible for their hydraulic behavior.

C. STRUCTURAL PARAMETERS

For each encapsulated sample under compression the thickness, the porosity and the mean distance between the fibres were measured. As an example, the histograms of the distance between fibres of sample of PR 395 fabric are presented on figure 12. The mean distance between the fibres is greatly affected by the compression level varying from a value of 140 µm at 25 kPa to a value of 38 µm at 800 kPa. As shown on the pore histograms, large holes in the structure are being filled with fibres as the pressure is increased such that smaller and smaller holes are created. This is very important because it points out that under compression the filtration behavior must be different (clogging level, particle size retention...).

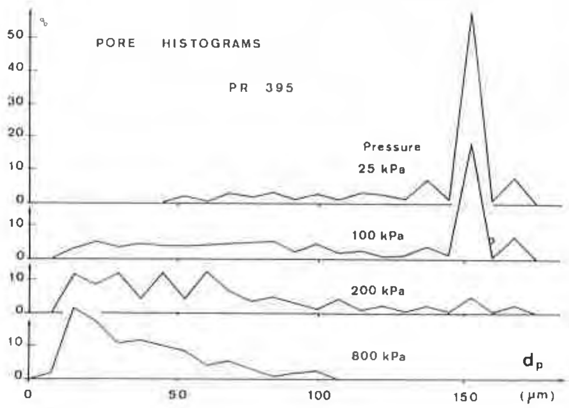

Fig. 12 : Variation of structure under compression by image analyser

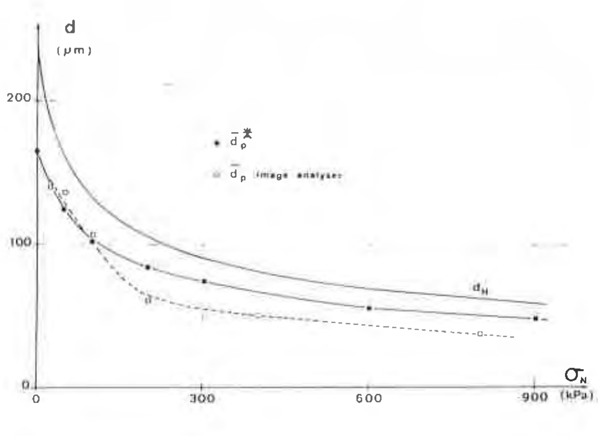

Fig. 13 : Characteristic diameter of pores under compression.

On the other hand the measured porosity of the same samples is equal to 86 % under 25 kPa and 72 % under 900 kPa. This is supporting the permeability measurements because even under large static pressures the porosity of needle-punched fabrics is very large such that under these conditions the permeability is not expected to vary significantly.

As shown on figure 13, estimated mean pore diameter by permeability method, $\bar{d}p^*$, using equation (26) are compared with the calculated hydraulic diameter and the measured mean distance between the fibres. The three curves are similar in shape indicating a possible correlation between these three parameters. Even though the actual calculated and measured values are different at higher compression levels, a study is presently under way to determine that correlation.

III. CONCLUSION

The analysis of structural parameters of geotextiles under compression coupled to normal and lateral permeabilities' measurements under identical static pressure supported the correlation between the level of compression and the decrease in the permeability coefficients, K_N and K_P, and the porosity of the geotextiles.

The relationship is very important because it enable to estimate the permittivity and transmissivity of fabric under compression by using only measured thickness and mass per unit area.

Finally it was found that, even though the permeability behavior of non-woven geotextiles are not greatly influenced by the compression level, the filtration behavior is expected to be greatly affected. The level of clogging and the soil retention are related to the size of the openings in a geotextile that are found to decrease by approximately three times under pressure varying from 0 to 800 kPa. In fact the structure of needle-punched fabrics is found to be altered in a range of pressure < 200 kPa.

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Hydraulic Function and Performance of Various Geotextiles in Drainage and Related Applications

Performance et rôle hydraulique des géotextiles dans le drainage et les applications similaires

Many types of damage are caused when soil particles are moved by water, e.g., drainage systems without proper protection against piping of the soil become silted, clogged, and ineffective; river banks, lake and coastal shores are eroded; structures are undermined and become unstable, etc. The importance of protecting soil from piping or scouring by water is well recognized, and well-graded aggregate filters have conventionally been used. Geotextiles have demonstrated they can replace well-graded aggregate filters in many applications.

The function of geotextiles in drains and related applications is to permit water to pass through without reducing its rate of flow, while preventing the soil from being moved by water. As the geotextile performs in a geotextile - soil-water system, drain design criteria and the functional requirements of geotextiles should be based upon conditions of realistic soil-water-fabric interaction.

Les déplacements des particules de sol causés par l'eau sont à l'origine de nombreuses catégories de dégâts. Par exemple, les systèmes d'écoulement, sans une protection régulière contre l'hydrauliquement du sol, s'envasent, se bouchent et fonctionnent inefficacement; les rives fluviales, lacustres et marines s'érodent; les fondations, dégravoyées par l'eau, deviennent instables, etc. L'importance de la protection du sol contre l'hydrauliquement est bien reconnue, et cette protection est normalement obtenue au moyen de filtres d'agrégat convenablement classés. On a trouvé que les géotextiles peuvent, dans nombreuses situations, remplacer tels filtres d'agrégat. La fonction des géotextiles dans les systèmes d'écoulement s'agit de laisser passer l'eau sans affaiblir sa vitesse d'écoulement, tout en empêchant le déplacement du sol. Le géotextile fonctionne dans un système géotextile - sol - eau; par conséquent, les critères du projet et les fonctions exigées des géotextiles doivent être fondées sur les conditions véritables de l'action réciproque du système tissu - sol-eau.

INTRODUCTION

Engineering fabrics or geotextiles were at first slowly accepted as their uses represented a rather new concept lacking documented long term field experience. Uses of geotextiles in construction projects were at first considered novelties limited to "non-critical" applications.

Specifications are sometimes developed from limited experience and less fundamental data; but as performance mechanisms and requirements for long-lasting, non-clogging drain systems, protected by geotextiles, are better understood, specifications for geotextiles can be set for ensuring their desired performance. The keys to proper functioning of various systems are their design and construction. Some new design criteria are suggested.

In drainage and related applications the importance of protecting soil from being moved by water has long been recognized. Conventionally, well-graded aggregate filter systems have been used to keep the soil from piping or being scoured. Well-graded aggregate filters are designed to be able to hold the soil in place, but they must permit the water to flow at an unreduced rate, thereby avoiding a buildup of excessive hydrostatic pressure. The design and construction of well-graded aggregate filters depend upon the quality of the soil they must protect and the degree of water action. Proper design criteria and construction will influence the performance of aggregate filters.

WELL-GRADED AGGREGATE FILTERS

The pore openings between the aggregate filter particles should be small enough to prevent passage of most of the particles of the protected soil. But the filter layer should be permeable enough to permit unreduced flow of water from the protected soil to the drain without becoming clogged. A well-graded aggregate filter may be visualized as an obstable course through which piping soil particles must pass before they can be carried by water into the drain. Since an aggregate filter is made of loose particles, it must sometimes be built in multiple stages to be resistant against disintegration by water action, such as with soil erosion protection of stream banks, or of lake and coastal shores subjected to heavy waves.

Taylor (1) showed that a small sphere would move through the opening enclosed by three perfect equal size spheres if the diameter of the larger spheres is greater than six and one half times that of the small sphere (Figure 1A).

In a more unstable configuration of four equal size perfect spheres enclosing the opening, a small sphere would move through the opening if the diameter of the larger spheres is greater than two and one third times that of the small sphere (Figure 1B). An average opening size of the two configurations would be between 4-5.

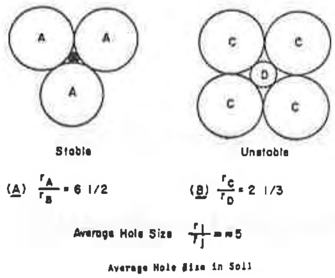


Figure 1

Bertram (2) established the filter design criteria:

$$\frac{D_{15}\ \text{(of filter material)}}{D_{85}\ \text{(of protected soil)}} < 8 \text{ to } 12$$

for protection against piping, and

$$\frac{D_{15}\ \text{(of filter material)}}{D_{15}\ \text{(of protected soil}} > 4 \text{ to } 5$$

for sufficient permeability to prevent buildup of hydrostatic pressures.

The U.S. Army Corps of Engineers (3), being more conservative, requires the following specifications be satisfied for protecting all soils, except for medium to highly plastic clays without sand or silt particles:

$$\frac{D_{15}\ \text{(15\% size of filter material)}}{D_{85}\ \text{(85\% size of protected soil)}} \leqslant 5$$

for protecting against piping, and

$$\frac{D_{50}\ \text{(50\% size of filter material)}}{D_{50}\ \text{(50\% size of protected soil)}} \leqslant 25$$

to ensure parallel grain-size curves of filters and protected layers. Multiple-stage filters are needed for medium to highly plastic clays.

Well-graded aggregate filters must often be hauled over long distances, and their quality, unless carefully monitored, might not be reliable. Their placement requires care because carelessly constructed filter systems could fail to function properly.

GEOTEXTILES

Geotextiles used for drainage and related applications should have functional criteria similar to those of well-graded aggregate filters. Fabrics should permit unreduced flow of water from the soil and simultaneously keep soil particles from piping or being scoured by water.

Often called filter fabrics, geotextiles for drainage and related uses actually do not perform as a true filter fabric. They function for an indefinitely long time without becoming blinded or clogged. True filters have limited functional lives as they eventually become blinded or clogged by continually accumulated particles. Geotextiles function properly by restraining the soil and keeping it from being moved by water. Actually, the soil body being restrained by the fabric holds soil particles in place.

Acceptance of geotextiles for civil engineering projects was at first slow as their use represented a relatively new concept. Hence, a few persons tried them only in "non-critical" applications. Over the years geotextiles have proven their good performance and as their uses grew rapidly, many suppliers entered the market with a variety of geotextiles having various fabric properties. In order to properly select a geotextile for a project, the fabric should satisfy the system's functional requirements with its structural and physical properties. A good understanding of the relationship between the relevant properties of geotextiles and their performance helps design engineers properly select and specify geotextiles for their projects. Key properties of geotextiles that are relevant and could influence their performance and functioning are:

- Structural features
- Ability to allow water movement without contributing to buildup of hydrostatic pressure, combined with ability to prevent soil movement by water.
- Adequate strength for withstanding normal, proper construction methods and for functioning through the service lives of projects.
- Dimensional stability and resistance to attack by microorganisms and chemicals to which the geotextiles are continuously exposed in the projects.

In a properly functioning drain constructed with a geotextile, water flowing from the soil into the drain can initially suspend and remove very fine surface soil particles through the fabric. This removal is not detrimental to the performance of the system. Suitable geotextiles in intimate contact with most natural soils will prevent them from being moved by flowing water.

PROPER FUNCTIONING OF GEOTEXTILES

In order for geotextiles to function properly in drainage and related applications, they should:

- Permit the passage of water without contributing to buildup of hydrostatic pressure.
- Prevent piping of soil particles by establishing a stable hydraulic condition in which the soil, kept in place by the more permeable fabric, maintains its structural integrity and prevents soil particles from moving through.

Actual field performance of drains constructed with geotextiles (4) confirmed the results of laboratory tests by various researchers (5). The researchers found that the percentage of soil actually trapped in meltbonded continuous filament nonwoven fabrics was quite small and did not vary significantly with soil type. The better the gradation of the soil, the smaller the amounts of fine particles that passed through the geotextile.

Results of tests conducted at Colorado State University Engineering Research Center showed that the gradations of the tested soils could determine whether the soil would control the hydraulic responses. It was determined that a soil layer satisfying the condition:

$$\frac{D_{85}}{D_{50}}, \frac{D_{50}}{D_{35}}, \frac{D_{35}}{D_{15}} < 5$$

would become a soil filter, and natural soil usually satisfies this condition.

When the soil body is not a filter, significant amounts of fine particles can be carried by water through the soil voids, and any fine opening system placed against such a soil would be made to function as a real filter that captures and accumulates the fine particles. When these systems perform as a true filter, they all will eventually be blinded by a cake of fine soil particles.

DESIGN CRITERIA

Initially little was known about uses of geotextiles as there was no long term experience with them. Decisions on how to select them were based upon results of limited trials and of laboratory evaluations. This sometimes led to incorrect or premature conclusions, e.g., a geotextile that has performed satisfactorily for certain conditions is automatically the best for every condition and should be used as the only standard for specifying geotextiles. Actually, each project should be individually studied, its performance requirements determined, and accordingly, its design criteria established to ensure successful functioning for its service life.

Sound engineering practices should be applied to each project. Careful design studies and proper construction methods help avoid mistakes and ensure satisfactory functioning of the system. Proper planning and designing should include site surveys. Soil conditions, soil quality, soil particle size distribution, and soil permeability and hydrology should be determined. In addition, one must establish functional requirements, and select the right materials for the project.

As more test and performance data are accumulated, they help engineers better understand what physical properties of geotextiles are relevant to the geotextile's performance. Such an understanding enables designers to properly select and specify geotextiles for their projects.

The U.S. Army Corps of Engineers is one of the few agencies to document results of hydraulic characteristic testing (6). Using soil types of known gradation, the Corps established guidelines for using geotextiles, indicating that selection of geotextiles should depend upon the D_{85} size of the protected soil, the EOS (equivalent opening size) and a gradient ratio. As these results were based upon tests conducted with woven filament fabrics, the design criteria for fabrics recommended by the Corps of Engineers would apply to woven but not to nonwoven fabrics, whose structures are quite different. Filtration tests were conducted for 60 hours, now considered too short a period as concluded from later tests at the University of Tennessee and at Colorado State University.

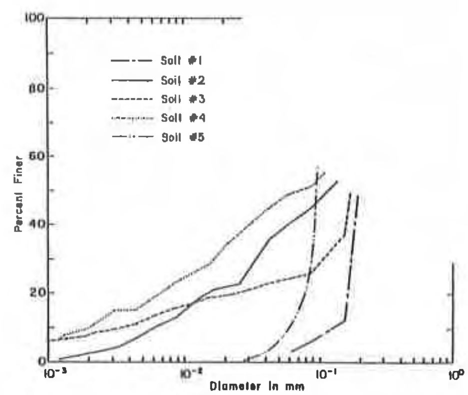
The University of Tennessee conducted tests for 21 to 28 days, or until the flow through the system became constant. Only one fabric was considered in tests using twenty soil types. The University of Tennessee researchers concluded that the design criteria of geotextiles must be the same as those established for conventional aggregate filters.

In order to broaden our understanding of the hydraulic function and performance of various geotextiles, laboratory tests were conducted at Colorado State University Engineering Research Center using five soil types with various fabrics in prolonged (800-1000 hours) permeameter tests.

The five soil types represent a variety of fine soils considered to be critical to geotextile usage by the U.S. Army Corps of Engineers and others. The soils are classified as sand, sandy loam (two), loam and loamy sand respectively, in the Triangular Soil Classification.

The soil particle size distributions for the five soils are shown in Figure 2. A different geotextile was placed within each permeameter.

Test results demonstrated that a soil layer having adequate particle size distribution can work as a filter to limit migration of fine particles within the soil. In the soil-fabric system, the soil serves as the filter and the fabric serves as a permeable constraint to prevent the soil from being moved through the fabric. As there are relatively few fine particles reaching the geotextile, the fabric will not be clogged.



Size Distribution for the Tested Soils

Figure 2

Results of laboratory tests at Colorado State University Engineering Research Center indicated that geotextiles would constrain particles if they satisfy the condition:

$$\frac{P_{95} \text{ (or EOS of fabric)}}{D_{85} \text{ (85\% size of protected soil)}} \leq 3$$

in which P_{95} is the pore diameter in the fabric of which 95 percent of the pores are finer, while earlier the U.S. Army Corps of Engineers (7) had suggested the ratio:

$$\frac{\text{EOS (of fabric)}}{D_{85} \text{ (of protected soil)}} < 1$$

Experimental results at Colorado State University indicated that this ratio was too restrictive as very little soil loss was observed even when the ratio

$$\frac{\text{EOS (of fabric)}}{D_{85} \text{ (of protected soil)}}$$

was varied from 0.04 to 3.82. Based on this work, a new ratio is, therefore, proposed:

$$\frac{\text{EOS (of fabric)}}{D_{85}\text{ (of protected soil)}} < 2$$

Some geotextiles have been observed to be more effective in their ability to restrain soil particles than indicated by their EOS.

Based upon analysis of laboratory test results at Colorado State University and actual field experiences with geotextiles, the following conclusions are drawn:

- Natural soils are generally well-graded and serve as filters limiting migration of soil fines. Suitable geotextiles used with such soils work as permeable soil constraints rather than as filters and do not become clogged.
- Self-filtering soils, which are less permeable than the geotextiles, control the hydraulic responses of the systems.
- When the soil body is not a soil filter, water passing through the soil can carry significant amounts of fine particles through soil voids. Any fine opening material used with such nonfiltering soils is made to work as a real filter that can trap and accumulate fine particles. An accumulated layer of fine particles on the filter can significantly lower its permeability and can cause hydrostatic buildup.
- In these laboratory tests it was observed that for 1000 hours of continuous testing, the fabrics used had no detectable effect upon the system's hydraulics. These fabrics included a variety of constructions; woven slit films, spunbonded products and needle-punched nonwovens. Absent other design criteria, the lightest suitable fabric should be selected for lowest system cost.
- The gradient ratio developed by the U.S. Army Corps of Engineers to indicate clogging of the soil/geotextile system should be analyzed for long-term (>500 hours) rather than short-term performance. In using short-term analysis, there could be system instabilities, especially with finer soils.
- The criteria established by the U.S. Army Corps of Engineers for granular soils containing 50 percent or less by weight silt should be increased to,

  $$\frac{D_{95}\text{ (or EOS of fabric)}}{D_{85}\text{ (of soil)}} < 2,$$

  and for soils containing more than 50 percent silt, P_{95} (EOS of fabric) < opening in U.S. Standard Sieve No. 70.

The old Corps criteria may apply to systems with coarse sands and large water velocities, but fine sands containing more than 30 percent silt and/or nonswelling clay will control the system and the standard would not apply. This was shown in tests using geotextiles with a wide range of opening sizes (EOS 20 to > 200) and soils with large concentration of fines. Based upon these test results, the U.S. Army Corps of Engineers' criteria appear too restrictive and conservative.

GEOTEXTILES IN DRAINAGE AND RELATED APPLICATIONS

Engineers have successfully utilized the above design criteria for soil-water-fabric systems in assorted drainage and related applications, below.

Drainage in Pavements

In pavement structures, water drainage is usually intermittent. The drains are designed for removing water that infiltrates during rains. This water should be removed at a rate at least equal to the infiltration rate. An effective passage of water from the pavement's subbase into the drain is essential for the proper functioning of the system.

Pavements of roads, airfields, parking areas, etc. with effective drains will have longer service lives and require far less repair and maintenance than those lacking drains (7).

To be most effective, drains of flexible and rigid pavements should be installed as part of the original structure. Installing them after the pavement had badly deteriorated may not be as effective. Badly damaged subbases of pavements can no longer permit the water to steadily flow into the drain. Instead, it is pumped by traffic pressure upon the pavement and jetted through cracks and voids, carrying with it fine soil particles from the subbase and subgrade.

The functionality of drains constructed with geotextiles has been proven in hundreds of miles of highways (8).

Drains Of Earth Dams

Performance requirements for these drains are different than for pavement drains. Drains of earth dams must continually remove seepage water under far greater hydrostatic heads. Protective systems for these drains must withstand these heads and effectively prevent piping of soil particles.

Engineers have recognized the following advantages of geotextiles over graded aggregate filters:

- Geotextiles have independent tensile strength and can better resist disintegration by water.
- Geotextiles are easier to handle and install.
- A single layer of geotextiles can perform as a multi-phase graded aggregate filter.
- Geotextiles have more reliable supply and quality control.
- Geotextiles are of much lighter weight and, therefore, require less people and equipment for handling, storing, and installing them.

Early examples of geotextile uses in dams are:

- Encapsulation of toe drain and upstream lining under riprap in 1970 (9).
- Downstream drainage collector and seepage cutoff (9).

Design engineers have used the soil-water-fabric design criteria discussed earlier for the design and construction of drains in various dams. A spunbonded polypropylene geotextile was selected for:

- Chimney and blanket drains of an earth dam for the water reservoir of a fossil fuel power plant in 1977.

- Underground French drains in the downstream slope of an earth dam of a large water reservoir and as lining under riprap of the dam's entire upstream slope and at the toe of its downstream slope. The dam was built in 1978.

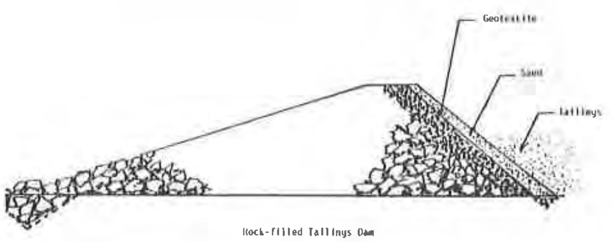
- Lining under sand of the upstream slope of a rock-filled tailings dam of a government copper mine in Chile (Figure 3) in 1981. The geotextile prevents the sand from being moved into the gravel layer of the dam.

- The blanket drain of a large starter dam for containing the tailings of a copper mine in 1981 (Figure 4). The 2km long dam will, over the years, be built to more than 90m high.

- Correcting drain systems of various dams by adding slope or toe drains to their downstream slopes without severely disturbing their existing structures.

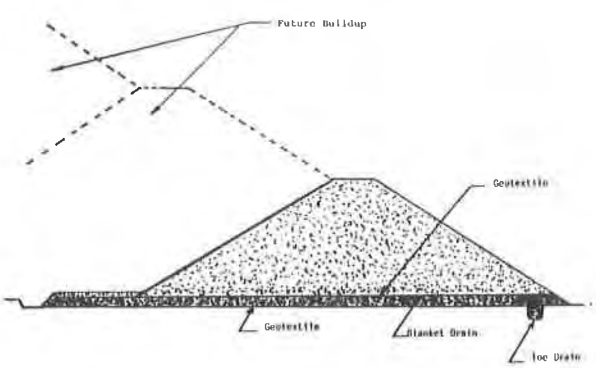
Vertical Drains

Synthetic vertical drains, also called wicks, are becoming widely accepted alternatives to vertical sand drains because they are considerably easier to install. They can better resist distortion from dynamic load stresses and reduce the risk of becoming discontinuous.



Rock-filled Tailings Dam

Figure 3
(Not to Scale)



Tailings Starter Dam
Figure 4
(Not to Scale)

Vertical drains are used to permit accelerated consolidation of wet soils by permitting water to be removed under the weight of the surcharge.

Vertical drains are expected to function under high hydrostatic pressures and should:

- Permit water to be removed from the soil at a relatively high rate without permitting soil to be removed with it.

- Not become ineffective due to distortion, clogging or degradation until the desired soil consolidation is achieved.

- Function effectively under relatively high pressures deep underground.

Intimate soil-fabric contact is virtually assured in synthetic vertical drains installed in wet soils. Their effectiveness depends upon the functionality of the sleeves around their compression-resistant plastic cores that provide passage for water squeezed into the drains. The sleeves must withstand rupturing and clogging and the cores must resist being crushed and distorted. Results of evaluations by a testing laboratory in Holland (10) led a manufacturer of synthetic vertical drains to select a spunbonded polypropylene geotextile for the sleeve of his product that has been used for various projects all over the world.

PROTECTING STREAM BANKS, LAKE AND COASTAL SHORES WITH GEOTEXTILES

The same soil-water-fabric design criteria apply as for drainage, but because water action in these applications is more severe than in drainage, there are additional functional requirements for their protective systems.

These systems should withstand, absorb and dissipate water forces. Maintaining intimate contact between the geotextile and the protected soil is essential to prevent piping and scouring of the soil.

To be effective, a riprap revetment should be heavy enough to withstand water velocity of the worst expected conditions. As the main forces of water assaulting the protective system impact from the outside, the riprap revetment is constructed of an outer belt of heavy, angular armor stones that will absorb and withstand these forces. The armor stones should interlock to better resist being moved and should have the lowest possible void volume between them to reduce the velocity of water. The voids can be reduced by filling them with smaller angular stones which interlock with the armor stones and with the fine stones of the interior zone. Such a united system would be more difficult to move than loose, individual armor stones.

An interior zone of crushed stone gravel (about 5-6 cm in diameter), or of small angular riprap (preferably smaller than 20 cm in diameter) should be placed directly on the geotextiles. This interior zone of interlocking stones, having a minimum of voids between them, will effectively dissipate the frontal assault of the water. Sometimes called a cushion blanket, it also:

- Ensures more even distribution of the weight of revetment stones on the geotextile's surface.

- Ensures better contact between the geotextile and protected soil, enabling the geotextile to better restrain the soil. This minimizes the risk of the geotextile pulsing and ballooning under water action, which can cause it to tear or make the revetment become unstable.

- Protects the geotextile against damage during placing of large, angular armor stones. This permits the use of less costly but equally effective lighter geotextiles without sacrificing the revetment's performance.

This revetment design (Figure 5) would be better than one with large, angular armor stones placed directly on the geotextile.

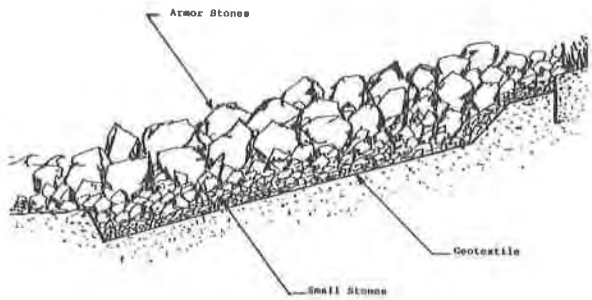
Designers should focus on the true function of geotextiles rather than become sidetracked by construction considerations. Geotextiles should keep the soil from being moved by both frontal assault and reverse flow of water.

Stones of different weights are required for different water velocities (11). The stones are more effectively used when interlocked and functioning as a united system. A good example is the use of relatively small stones kept in gabion baskets that can withstand water velocity far greater than the individual stones can withstand. Severe wave forces can damage revetments with large open areas exposed to the waves (12), even though test results had indicated the heavy individual units would be effective for the project.

In 1976-77 the New Orleans District of the U.S. Army Corps of Engineers conducted a series of tests. They evaluated the effectiveness of different geotextile structures for restraining fine Mississippi River bank soil exposed to river current and wave action generated by assorted river traffic. Bags made of different fabrics were tightly filled with the soil, and then stacked in the river under water. The effectiveness of the geotextiles was indicated by their ability to retain the soil while the bags are constantly exposed to water action of river currents and boat traffic. Bags made of a spunbonded polypropylene geotextile were found to be more effective than those of woven polypropylene because the spunbonded polypropylene geotextile structure contains a range of opening sizes that parallel more closely the particle size distribution of the protected soil. The same method might perhaps be used for evaluating geotextiles exposed to laboratory wave action.

The effectiveness of a spunbonded polypropylene geotextile for protecting soil against scouring is demonstrated in a large Dutch delta project in which millions of square meters of geotextile are used.

In order to ensure their successful performance, geotextiles should be selected with the same careful engineering, design and construction being practiced with conventional materials. Fabric performance is improved by modifying construction methods to achieve the design which will provide a properly functioning system at the lowest cost.



Revetment With Cushion Blanket
Figure 5

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Pressure Drop through Non-Woven Geotextiles: A New Analytical Model

La perte de charge au travers les géotextiles non-tissés: un nouveau modèle

Flow of water through non-woven geotextiles has been associated to the flow of a fluid through granular media such as bed of gravel, sand or soils. The analysis of the hydraulic gradient across capillaries has been used to represent the flow behavior across synthetic fabrics even though the analysis should be applied only to porous media with a porosity equal to or lower than 0.50. As shown, most of the thick non-woven geotextiles contain void fraction greater than 80% such that a new analytical model has been developped to represent the flow of a fluid through fabrics. This model considered the flow of water across a bundle of fibres uniformly distributed throughout the thickness of the fabric. It was found, using data obtained from 6 different permeameters with hydraulic head ranging from values of 450 to 1.5 mm of water and using 40 samples of non-woven geotextiles, that the flow regime was laminar in all the range of flow conditions encountered.

L'écoulement de l'eau au travers de géotextiles non-tissés a été associé à un écoulement au travers d'un milieu poreux tels que les sols. Bien que l'analyse de la perte de charge au travers d'un tube capillaire ne s'applique qu'à des milieux poreux de porosité plus faible que 0.50, elle a été utilisée pour représenter l'écoulement au travers des membranes synthétique. Cependant tel que démontré, les géotextiles non-tissés possèdent une porosité plus grande que 80% de telle sorte qu'un nouveau modèle analytique a été développé pour représenter l'écoulement de l'eau au travers un faisceau de fibres uniformément distribuées dans le géotextile. Les résultats obtenus à l'aide de 6 perméamètres, dont les gradients hydrauliques variaient de 450 à 1.5 mm, et utilisant 40 échantillons de géotextiles non-tissés ont permis de determiner que le régime d'écoulement sous ces conditions est laminaire.

INTRODUCTION

To compare different soils with another, the Darcy's law is used establishing that the permeability coefficient is a function of the type of granular material and its particle size for flow of water at 20°C. The quantity of water flowing through a unit area per unit of time of a layer of soil varies directly with the hydraulic gradient and inversely with the soil's layer thickness (8). This relationship can be applied only when the flow through a porous medium is in laminar regime as it is the case for water travelling through soils. The finer the particle size of a soil, the greater is the resistance to flow such that the permeability coefficient is a useful constant to compare kind of granular material with another.

In recent years, synthetic fabrics have been extensively used in geotechnical work. They offer a way to develop drains with high discharge capacity and good resistance to clogging to protect open-graded drainage aggregates from clogging by adjacent fine soil. Typically many products are non-woven fabrics consisting of a mass of fibres intertangled together. The unloaded thickness of a particular fabric is simply a reflection of the degree of fibre's entanglement that is achieved in manufacturing the fabric (7).

One should not be suprised to learn that even the densest, most compact, needle-punched products contain 75% or more void space in their structure. Usually under no static pressure, the porosity of non-woven geotextiles except spunbonded fabrics, is higher than 90%. More appreciable is the fact that even under a load of 800 kPa, the average porosity of a Polyester needle-punched fabric of 400 g/m^2 is approximately 70% (15).

These very large values of porosity cannot be compared to the porosity of soils that are know to be lower than 50%. One must then be careful in applying to geotextile the flow analysis used for granular materials.

Recently many experimental results have been interpreted to establish the flow regime existing in geotextiles' structure under a wide range of hydraulic heads(5, 12). The growing interest in correlating the hydraulic gradient with the flow parameters of test apparatus emerged from the difficulty of comparing results obtained from different laboratories as well as from an effort to normalize a permeability test method. Until now engineers have been applying the same analytical model for flow through soils and geotextiles, and many permeameters were designed and used (1,14).

In this paper, it will be shown that the approach used until now does not represent the mechanism encountered when water is flowing through non-woven geotextiles. A new approach for the fluid flow through geotextiles is presented and experimental data gathered to support the relationship between the hydraulic gradient and the established flow of a fluid across a bundle of fibres.

FLOW THROUGH SOILS

The flow of a fluid through a porous medium has been extensively studied using the model of flow through capillaries of complex shapes and lengths. The Hagen-Poiseuille formula representing flow through a tube is well known (5,3)

$$V_o = \Delta P\, R_h^2 \,/\, 2\,\mu L' \qquad (1)$$

where V_o is the mean velocity of the fluid in the tube, ΔP is the pressure drop along a tube lenght L', R_h is the hydraulic radius and μ is the fluid's viscosity.

For a granular medium constituted of soil particles, the mean pore diameter is defined using the equivalent diameter, D_e , related to the hydraulic radius defined as the ratio of the cross section available to flow to the wetted perimeter

$$D_e = 4\, R_h \qquad (2)$$

Using the porosity, n , and the specific surface, a_v , (the total particle surface to the volume of the particles) equation (2) becomes

$$D_e = n \,/\, a_v\, (1-n) \qquad (3)$$

Supposing that the granular medium is constituted of spherical particles of diameter D_s , then the equivalent diameter can be expressed in term of particle's diameter

$$D_e = \frac{2}{3} D_s\, n \,/\, (1-n) \qquad (4)$$

Because the average velocity in the interstices is not of general interest, the superficial velocity is defined

$$V = V_o\, n \qquad (5)$$

Substituting equations (5) and (4) into the Poiseuille's law, one obtains

$$V = \frac{\Delta P \; D_s^2 \; n^3}{L' \; 72\, \mu \; (1-n)^2} \qquad (6)$$

Using the defined friction factor, λ , and the Reynolds number, R_e , for flow in a tube, the following expressions are found

$$\lambda = \frac{g \;\Delta H \; D_p \; n^3}{V^2 \; L \; (1-n)} \qquad (7)$$

$$R_e = \frac{D_p \; V \; \rho}{\mu \; (1-n)} \qquad (8)$$

where D_p is the particle's diamater, L is the thickness of the soil's layer and ΔH is the hydraulic gradient in height of fluid.

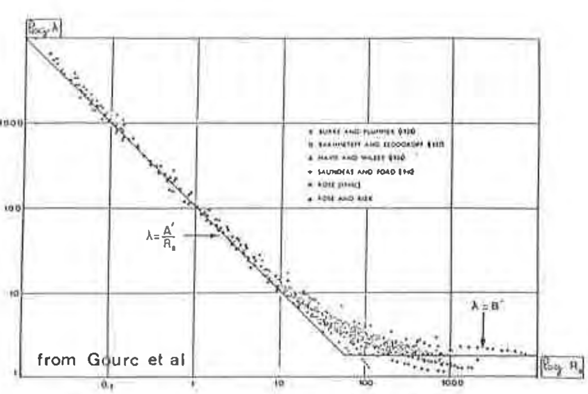

Fig. 1 Pressure drop coefficient for uniform granular media.

Data obtained by several researchers (5,13) for flow through porous media are presented on Figure 1. It can be observed that for $R_e < 10$, a line of slope -1 is obtained resulting in the following expression

$$\lambda = A' \,/\, R_e \qquad (9)$$

This region correspond to a laminar flow regime and can be expressed by the well known Blake-Kozeny equation. It is very important to stress that this analysis is valid for a porosity less than 0.5 .

For a soil, the region of flow for $R_e < 10$ corresponds to a laminar flow regime and Darcy's law can be applied for flow of water at 20°C

$$\frac{Q}{S} = K \frac{\Delta H}{L} \qquad (10)$$

where S is the flow area, K the permeability coefficient and Q the fluid flow.

For $R_e > 1000$, the friction factor is no more a function of the Reynolds number and the flow regime is turbulent. Finally a transitional flow regime exists for $10 < R_e < 1000$.

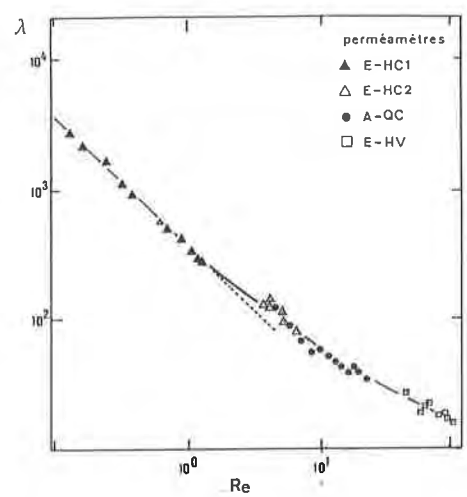

Fig. 2 Pressure drop coefficient for BD geotextile samples (Gourc et al (5))

On Figure 2, the calculated friction factor (5) for samples of Bidim C-34 fabric (BD) and obtained under a range of hydraulic gradients using four different permeameters are plotted versus their Reynolds numbers as defined by equations (7) and (8). It can be observed that, a curve similar in shape to the curve representing the flow across a layer of uniform granular media, is obtained with the difference that the critical Reynolds' number at which Darcy's law cannot be applied is approximately 1.

This would tend to indicate that the transitional flow regime is starting at lower flow rate in a non-woven geotextile than for a granular medium. This cannot be the case as the porosity of this non-woven geotextile is 93% compared to granular media with porosity values less than 50%.

FLOW THROUGH NON-WOVEN GEOTEXTILES

As shown on Figure 3 and as already discussed in a earlier paper (10), thick non-woven fabrics are constituted of a large number of fibres randomly distributed. From the analysis of obtained internal structures of geotextiles, the analytical model that should represent the flow of fluid through non-woven fabrics must be that of flow around a bundle of cylinders, as shown schematically on Figure 4.

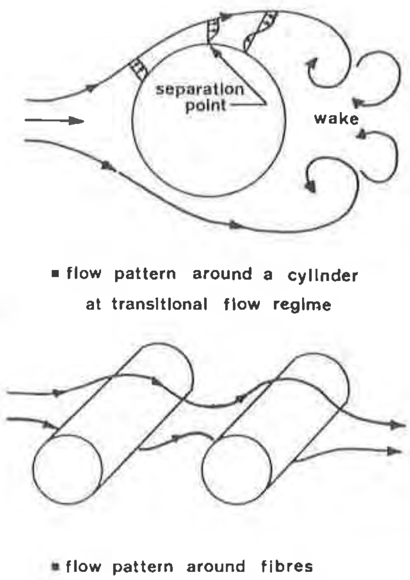

Fig.4 Schematic flow pattern around a cylinder and around two fibres

As the fluid is flowing around a fibre, a force, in the direction of flow, is exerted by the fluid on the solid and it is defined as "drag". A form drag is related to the pressure and a wall drag is related to the shear stress.

Even though the development of flow around an immersed cylinder is well known (6), unfortunatly the phenomena causing both wall and form drag in actual fluids are complicated and cannot in general be calculated. They are always determined by experiments (11).

The drag coefficient, C_D', is defined by the following equation

$$F_D = C_D' S_p K_e \tag{11}$$

where F_D is the drag force, S_p is a characteristic area and K_e is a cinetic energy term. For a cylinder of length "l" and of diameter "d", the characteristic area is taken as the projected area (lxd) and the cinetic energy term as

$$K_e = \frac{1}{2} \rho v_\infty^2 \tag{12}$$

where v_∞ is the velocity in the bulk of the fluid. Substituting these expressions in equation (12), the drag coefficient becomes

$$C_D = F_D / (lxd)(\frac{1}{2} \rho v_\infty^2) \tag{13}$$

From a dimensionless analysis, the drag coefficient is found to be a function of the Reynolds number defined as

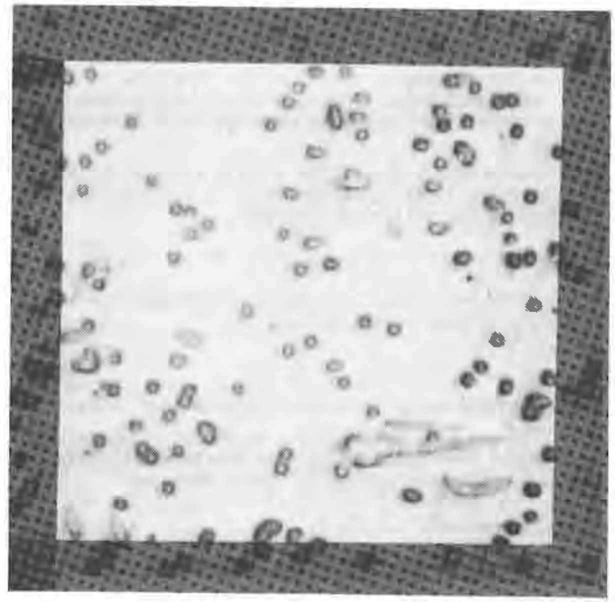

Fig. 3 Cross section of a needle-punched geotextile

$$R_e = \rho v_\infty d / \mu \tag{14}$$

Many experimental works have been conducted in the past to obtained data to correlate the drag coefficient around a single cylinder to the flow conditions. The curve representing collected data is presented on Fig.5.

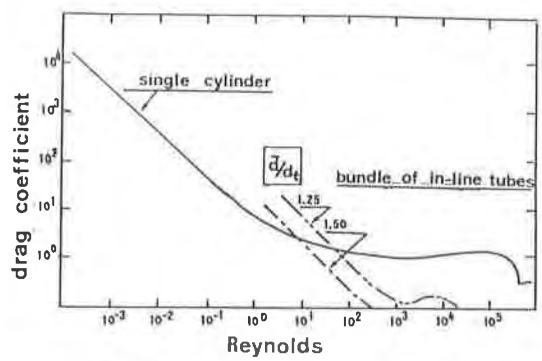

Fig. 5 Drag coefficient around a cylinder and a bundle of tubes.

It can be observed that for low Reynolds' numbers, $R_e < 1$, the data fall on a straight line such that the flow regime is laminar. It is usually called " creeping flow ". As the Reynolds' number is increased, a separation in the boundary layer on the cylinder surface occurs at a point just forward the equatorial plane and a wake, covering the entire rear hemisphere, is formed as schematically shown on Figure 4. A vacuum is then created at the rear of the cylinder resulting in a upward trend of the drag coefficient. As the flow rate is further increased, the separation point moves toward the rear of the body such that finally the wake is dissapearing resulting in a sudden decreased of the drag coefficient.

For the case of a bundle of cylinders or of fibres, the presence of the adjacent cylinders will affect the flow behavior around each cylinder. One can intitutively picture that as the distance between the cylinders is decreased, the higher should be the flow rate at which the separation point appears with the result that the transitional flow regime is expected to be shifted to higher R_e.

The pressure drop which a fluid flowing through a bundle of cylinders experiences(2) is conventionally expressed by the following equation

$$\Delta P = N C_D \rho V_0^2 / 2 \qquad (15)$$

where N is the number of rows of tube in the bundle, C_D is the drag coefficient defined as

$$C_D = \Delta P / N \rho V_0^2 \qquad (16)$$

and V_0 is the velocity inside the bundle of tubes.

The pressure drops across bundle of tubes in heat exchangers were measured (4) and related to Reynolds' numbers determined by the following equation

$$R_e = \rho V_0 d_t / \mu \qquad (17)$$

where d_t is the diameter of the tube.
The empirical curves obtained for in-line tubes' arrangement (2) are presented on Figures 5,7 and 10. As forecast a family of curves exists depending on the ratio of the distance between the tubes to the tube diameter. As the ratio is increased, the influence of the second row of tubes on the preceeding row is less such that the flow is more similar to flow around a single tube. In fact, one can observed on Figure 5, that the critical Reynolds' number at which the laminar flow regime does not exist anymore is greater for flow through a bundle of tubes than for the single cylinder. The smaller the distance between the tube, the greater is the critical Reynolds' number.

Applying the same development to geotextiles and supposing that the fibres arrangement is a serie of rows with square pitch, the following equations can be derived

$$R_e = \rho V_0 d_f / \mu \qquad (18)$$

$$C_D = g \Delta H / N V_0^2 \qquad (19)$$

where d_f is the fibre's diameter, ΔH is the hydraulic gradient, V_0 is the velocity inside the fabric and N is the the number of rows of fibres. This value can be found from the following equation

$$N = b / (\bar{d} + d_f) \qquad (20)$$

where $\bar{d}$ is the mean distance between the fibres and b is the fabric's thickness.

The value of the mean distance between the fibres can be measured using an Image Analyser (10) or can be estimated using the following equations and the curve presented on Figure 6. (9).

$$n = 1 - \frac{m^+}{b \; \rho_s} \qquad (21)$$

$$\nu = 4 (1-n) / \Pi \; d_f^2 \sqrt{3} \qquad (22)$$

where m^+ is the fabric mass per unit area, ρ_s is the polymer density, ν is the fibres' density per unit area of cross section of fabric.

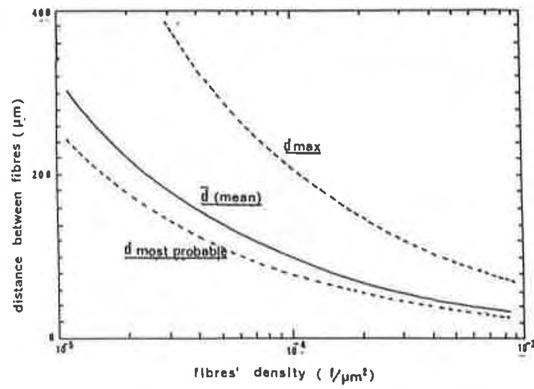

Fig. 6 Distance between fibres as a function of the fibres' density.

Using equations (21) and (22), the number of fibres per unit area of analysed cross section of the geotextile can be calculated. In a following step, the curve representing the mean distance between fibres as a function of the fibres' density is used and the number of rows of fibres can be calculated using equation (20).

EXPERIMENTAL

A) Structure of non-woven fabrics

To confirm that the porosity of thick non-woven geotextiles is greater than 75% and to obtain the structure' parameters of tested geotextiles, twelve non-woven fabrics were analysed using the Image Analyser technique (10). Photographs of cross section of one of these fabrics is shown on Figure 3. As already stated, a cross section of a non-woven fabric should be schematically represented as a porous medium constituted of a large number of fibres randomly distributed. Each dot, representing the cross section of a fibre, is surrounded by void (white area). The experimental testing procedure to obtain picture of cross section of geotextiles has been already discussed in earlier papers (10,14).

The measured structure parameters of the choosen commercial virgin fabrics are discussed. The geotextiles were fabricated with Polyester fibres of diameter equal to 25 μm with a resulting thickness ranging from 2.0 to 7.8 mm. The fibres' density varies appreciably from one fabric to another with results indicating a range of values from 26×10^{5} to 3.8×10^{5} fibre/μm^2.

But the most significant parameter for this study is the porosity that was measured for each fabric with values ranging from 80 to 94%. These values are supporting the statement that the Blake-Kozeny equation must not be applied to non-woven geotextiles because of their very high porosity.

The calculation technique using equations (21) and (22) and the curve on Figure 6 was used with a sample of BD fabric to obtain estimated values of the porosity and also of the average distance between the fibres. It was found that the difference between the estimated and the measured porosity was of 1% (92% compare to 91%) and that for the mean distance between fibres was 9% (133 μm compare to 146 μm). This procedure was then accepted and used to calculated the number of rows of fibres for the samples used at Grenoble and at Texel laboratories.

B) Permeability measurement

Experimental work was performed at two laboratories, Ecole Polytechnique de Montréal and Texel Inc., and data published by Gourc et al (5) were used. The data from Gourc et al were obtained with four types of permeameters using samples of BD fabric. In this study two other types of geotextiles were choosen to represent thick non-woven fabrics produced by different manufacturing processes: Texel's fabrics (TE) and Terrafix's fabrics (TR).

A total of twelve samples were analysed with the Image Analyser and permeability measurements through forty-one samples were performed and compared with the data obtained at Grenoble using twenty-two different hydraulic gradients. Also twenty-six fabrics were manufactured on the same machine using identical fibres with the only variable being the mass of fibres per unit area of fabric.

The data obtained from the six permeameters are presented on Figures 7, 8 and 9. The hydrostatic gradients covered a range from 450 to 1.5 mm of water with samples of fabrics' area ranging from 353 to 2.85 cm^2. Also flow through a pile of 15 samples was measured, with varying hydraulic head, in one of the permeameter used in Grenoble (HC-1). These data were choosen to represent a wide range of flow conditions to verify the correlation in a range covering most of the test apparatus used in soil's and producer's laboratories.

For each of the 63 samples, the drag coefficient and the Reynolds' number were calculated. The R_e range from values of 25 to 0.01 with corresponding drag coefficient ranging from values of 0.3 to 624.

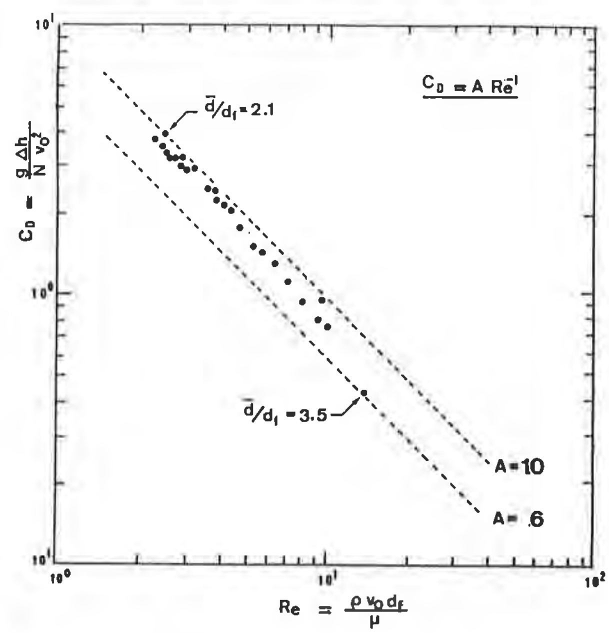

Fig. 7 Drag Coefficient for TE geotextiles

The permeability of the TE samples, varying in mass from a value of 250 g/m^2 to a value of 1450 g/m^2, were measured and the porosity as well as the mean distance between the fibres were estimated. The calculated Reynolds' numbers and the drag coefficients are reported on Figure 7. Because the ratio of the mean distance between the fibres to the fibre's diameter varies from values of 2.1 to 3.5, the data must not fall on a straight line of slope -1. Taking the 250 and 1450 g/m^2 samples as the lower and upper limits of the correlation between the drag coefficient and the Reynolds' number, the value of A in the following expression was found to be ranging from 10 to 6

$$C_D = A / R_e \tag{23}$$

This set of data indicates that as the ratio value, $\bar{d}/d_f$, becomes larger, the flow around each fibre is less disturbed by the presence of the surrounding fibres and consequently the experimental curve fall lower as shown on Figures 8 and 10.

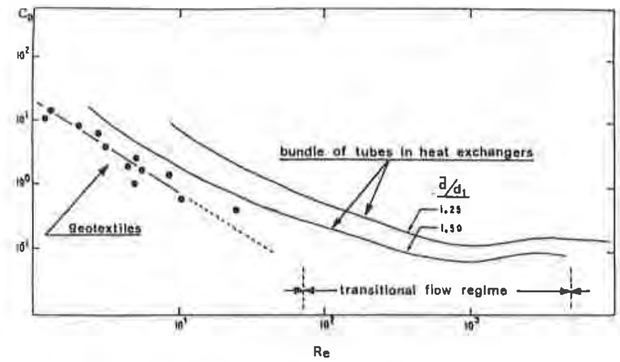

Fig. 8 Drag coefficient for BD,TE and TR geotextiles.

The data obtained for 12 different fabrics using a second permeameter are presented on Figure 8 and compared with two curves representing the pressure drop across a bundle of in-line tubes in heat exchangers. The upper curve was obtained with a ratio $\bar{d}/d_f$ of 1.25 while the lower curve is for a ratio of 1.50. The experimental values for geotextiles are lower suggesting that the ratio for geotextiles should be greater than a value of 1.5. This is the case for most of the analysed fabrics with a characteristic ratio value of 3.

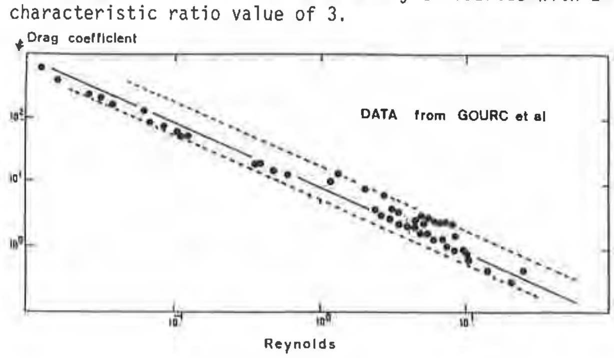

Fig.9 Drag coefficient for BD geotextiles

The data published by Gourc et al are presented on Figure 9. In their work, they have tested BD samples using four permeameters. The ratio of $\bar{d}/d_f$ was estimated at 2.96 and the experimental drag curve should be a straight line of slope -1 with a value of A close to 10. This is the case for most of the data except for the case where the permeability of the pile of samples was measured. The drag coefficient estimated are lower than expected. This can be a result of the overestimation of the real thickness of the pile that was obtained by multiplying by 15 the thickness of the unloaded sample.

As shown on Figure 10, the experimental values can be approximated by a straight line parallel to the curves obtained for bundle of tubes in heat exchangers. The flow rate at which the separation point appears on the fibres are higher from the presence of the other fibres such that a wake do not appear in the rear of the fibre unless the Reynolds' number is very high. Also the greater is the ratio $\overline{d}/d_f$ the greater is the distance between the fibres and the sooner will the transitional flow regime appeared.

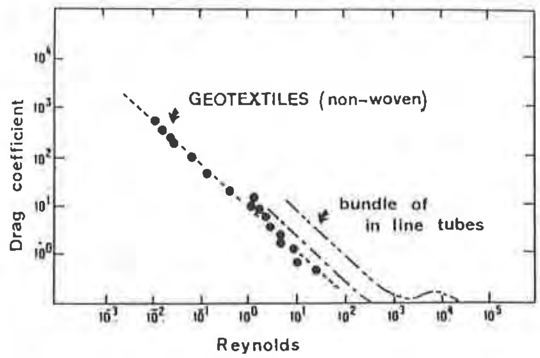

Fig. 10 Drag coefficient for non-woven geotextiles and bundles of in-line tubes.

C) Permeability correlation

From this study, it was confirmed that a relationship exists between the drag coefficient and the Reynolds number supporting the proposed flow model, flow through a bundle of fibres. Even more, from these correlations an expression for the permeability coefficient can be obtained

$$K = \frac{d_f \; (d_f + \overline{d} \;) \; n \rho \; g}{A} \tag{24}$$

The permeability coefficient is a function of the geotextile's structure and the fluid's properties. For water at 20°C, both the values of the density and the viscosity are equal to one and the permeability is only a function of the fabric structure. As an exemple, the estimated permeability of a TE sample, using equation (24), is 0.00258 m/s compared to a measured value of 0.00240 m/s.

This expression is also very useful in predicting the permeability coefficient under compression. Under a load, the porosity as well as the distance between the fibres will decrease resulting in a decrease in the value of the permeability coefficient. This is presently being investigated more thoroughly.

CONCLUSION

Permeability measurements through geotextiles, coupled to the analysis of their structure, were used to support the model of flow across a bundle of fibres. It was found that, the data obtained from six different designed test apparatus and using more than 60 samples of fabric can be best represented by a correlation analog to flow across a bundles of tubes in heat exchangers. The resulting empirical correlation is $C_D = A \;/\; R_e$ with values of A ranging from 6 to 10 depending on the ratio $\overline{d}/d_f$. For the case of most of the studied geotextiles, the mean distance between the fibres is three times the fibre's diameter. For Reynolds' number as high as 25, the flow regime is still laminar even though hydraulic gradients as high as 450 mm of water were used. This should be keep in mind in determining standard permeability test method.

Secondly, it was established that the estimated values of the porosity and the mean distance between the fibres of thick non-woven fabrics check within less than 10% of the measured values using the Image Analyser. The properties of fabrics can then be easily estimated by engineers using only the known manufacturing data.

Finally the permeability coefficient for flow of water at 20°C through geotextiles was found to be a function of the fabric's structure forecasting their behavior under compression. As a pressure will be applied on fabrics, their permeability coefficient will be affected, decreasing with greater loads. The change of structure and the flow behavior of non-woven geotextiles under compression is presently under investigation.

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Compressibility and Hydraulic Conductivity of Geotextiles

La compressibilité et la conductivité hydraulique de géotextiles

Presently available filter design criteria differentiate between specific geotextiles in terms of their mode of construction and thickness. In this paper, three compressible geotextiles are tested in an apparatus which measures compressibility and hydraulic conductivity over a range of confining stresses. Recommendations are given for the use of the data from these tests to modify existing design criteria and so take account directly of compressibility.

Les critères actuels de la conception de filtres differencient entre les géotextiles spécifiques en fonction de leur mode de construction et de leur épaisseur. Dans cet article on soumet trois géotextiles compressibles a des essais dans un appareil qui mesure la compressibilité et la conductivité hydraulique compressée. On donne des recommandations en ce qui concerne l'emploi des données de ces essais pour modifier les critères de conception actuels, et ainsi tenir compte direct de la compressibilité.

INTRODUCTION

The introduction of geotextiles as filter drains has involved the use of materials which are very thin, sometimes compressible, and with a structure which may contain a far greater range of pore sizes than would normally occur within conventional granular drains. The use of geotextiles thus necessitates a greater appreciation of the factors which influence filter drain performance and will generally require a different approach to be adopted for their design. A joint programme of research has been undertaken by the Transport and Road Research Laboratory and the University of Strathclyde to develop such an alternative approach.

In this paper, consideration is given to the main factors which influence the filter function of geotextiles and an apparatus is described for testing their compressibility and confined hydraulic conductivity. Results obtained from tests in this apparatus, carried out on three widely differing materials, are presented and an alternative approach to the design of geotextile filter drains is suggested which includes directly the influences of the compressibility of the geotextiles.

FACTORS INFLUENCING FILTER DRAIN PERFORMANCE

Properties of the Soil

The properties of soil which have most influence on the filter function of geotextiles are their particle size distribution, shape and structural arrangement. By virtue of the above properties, the size, shape and tortuosity of the voids and flow channels through which water must pass are governed. Moreover, soils are most commonly inhomogeneous and anisotropic possessing layering, fissures and other local variations in their structure which affect hydraulic conductivity. Thus although density is sometimes used as a measure of soil structure, this property at best can only represent the average conditions within a soil which in turn may frequently have little or no value, particularly when fluid flow is dominated by only a few porous channels. In view of this inherent variability it is necessary to present soil properties in terms of an average value together with upper and lower bound values.

Properties of Geotextiles

Many of the researchers determining geotextile filter design have based their criteria on the two principal geotextile properties of pore size and permeability, (1, 2, 3, 4, 5, 6). This approach is essentially an attempt to consider the geotextile as being equivalent to a thin layer of soil. However, one problem which arises with such an approach is that of making a reliable assessment of pore size. A further difficulty is that, because of such factors as geotextile compressibility, the use of a constant permeability coefficient is not strictly valid. Further consideration of the assessment of these properties is given in subsequent sections.

Conditions of Hydraulic Flow

The conditions of flow can have a major influence on the performance of both conventional and geotextile filter drains although such affects are likely to be most pronounced with the latter types. Flow can be steady-state, transient, cyclic or reversible. Moreover, the direction of the flow with respect to the plane of the geotextile can have important consequences on its performance. Thus the selection of the appropriate geotextile for particular conditions will involve consideration of these various factors.

PORE SIZE OPENING DATA AND THEIR USE IN FILTER CRITERIA

The size of the openings in many open weave geotextiles can be directly measured, but with closely woven and non-woven geotextiles, such measurements, are not possible. For this reason, various wet and dry sieving techniques have been developed, (1, 2, 6, 7). The data obtained from these various tests are not entirely consistent, but all can provide an approximate estimate of the largest pore sizes of geotextiles in a relatively unloaded, uncompressed state. Most of the techniques produce some scatter of results from different test specimens of the same geotextile. This scatter indicates that the data on pore sizes should be presented in terms of an average value together with upper and lower bound values as described previously for soil properties. Schober and Tiendl (3) suggest a method of comparing the various filter criteria based on pore size data. They assume that the grading curve for a soil may be represented by a straight line on the usual semi-log plot, with the slope of the line being determined from the uniformity coefficient of the soil. Using this approach to normalise the filter criteria referred to previously, produces the relation shown in Fig. 1. From this figure it can be seen that the various criteria fall into two distinct groups. The first relate to woven and thin non-woven geotextiles, the second to thick non-wovens. The consistent pattern of these various criteria appears at first sight rather convincing, however as they all involve similar assumptions concerning geotextile behaviour, this may induce the trend of the results to adopt the same pattern. Moreover, it is not made clear in any of these criteria what limitations apply to the classifications for both thick and thin geotextiles. Clearly, critical factors in these limitations will be the complexity of the pore structure and the size of the filaments comprising the geotextile relative to its thickness. The former property cannot be directly measured in a simple manner but it is suggested that such properties may be assessed indirectly by considering the compressibility of geotextiles. Giroud (8) suggests that the changes in pore space may be related to compressibility in the following manner:

$$\frac{O + d_f}{O' + d_f} = \frac{Tg}{Tg'} \quad \ldots\ldots\ldots\ldots\ldots\ldots 1$$

Where O is the average pore space between filaments corresponding to thickness Tg, O' is the average pore space between filaments corresponding to thickness Tg' and d_f is the diamter of the filaments.

Considering again Fig. 1, it can be seen that the presently available design criteria vary from that of Calhoun (1) dealing with incompressible wovens to that of Schober and Tiendl (3), dealing with very compressible thick non-wovens. If approximate upper and lower limits are placed on the various design criteria, as shown in Fig. 1, then the relationships between the filter criterion (β), which is based on unloaded pore size data, and the soil uniformity coefficient (U) can be expressed as follows:

$$\beta = 0.5\ (U + \eta) \quad \ldots\ldots\ldots\ldots\ldots\ldots 2$$

where η is a geotextile compressibility factor varying from a value of 1 for wovens to 7 for very compressible non-wovens. A need now exists to directly link this compressibility factor (η) with the measured compressibility of geotextiles.

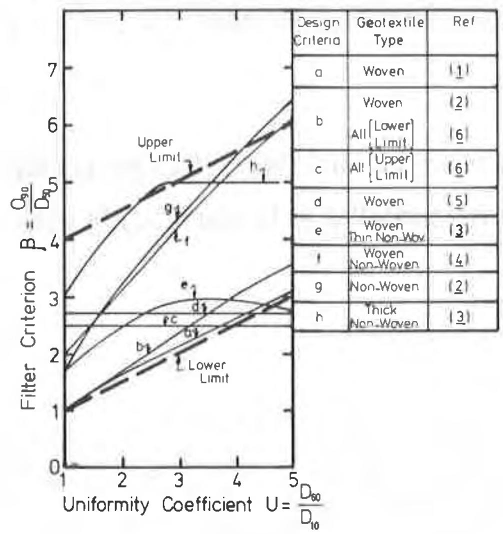


Fig. 1. COMPARISON OF AVAILABLE FILTER CRITERIA

REPRESENTATION OF GEOTEXTILE HYDRAULIC CONDUCTIVITY IN DESIGN CRITERIA

The most common approach is to specify the geotextile as requiring a Darcy coefficient of permeability which is some multiple of that of the soil. However, as stated previously, both the soil and the geotextile properties may vary with applied stress and it is inappropriate to attempt in general to assume a single unique value for the geotextile in order to define its relative properties. Also water flow through geotextiles is not laminar at all hydraulic gradients and confining stresses. Thus the simple proportionality relation between flow and hydraulic gradient based on Darcy's Law does not always apply. It is more appropriate therefore to assume a more general flow law for any confining stress, as follows:

$$i = bV^n \quad \ldots\ldots\ldots\ldots\ldots\ldots 3$$

where i is hydraulic gradient

V is the average flow velocity through the geotextile.

b, n are constants.

Note that when flow is laminar and n becomes unity, 1/b corresponds to Darcy's coefficient permeability (k).

A difficulty of using the more general law arises because of the variations in the values of b and n with confining stress in some geotextiles, particularly with non-wovens and composites which are compressible. To overcome this, it is suggested that the hydraulic conductivity be represented as follows:

Firstly at any confining stress,

$$\frac{i_2}{i_1} = \left(\frac{V_2}{V_1}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots 4$$

If i_1 is taken to be unity then Eqn. 4 may be rewritten in the more general form:

$$i = \left(\frac{V}{V_1}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots 5$$

where V_1 is the average flow velocity through the geotextile at a hydraulic gradient of unity.

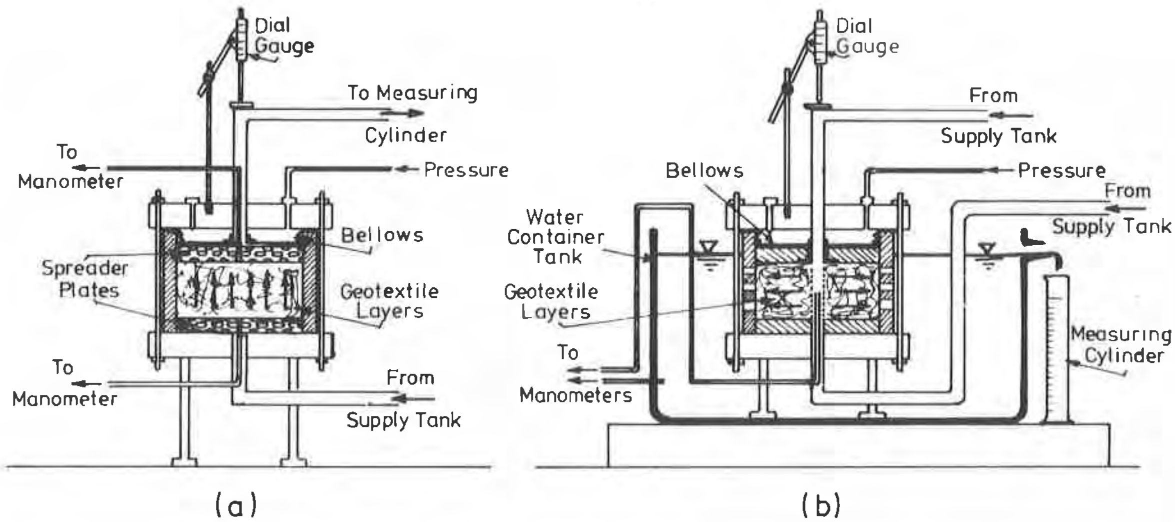


Fig. 2. LAYOUT OF TEST APPARATUS a) VERTICAL FLOW
b) RADIAL FLOW.

Now if tests are carried out at different confining stresses (σ_c) then the relationship between i and V can be calculated providing the relationships between V_1, n and the confining stress σ_c are known. Also the hydraulic gradient is related to thickness of the geotextile (Tg) as follows:

$$i = \frac{\Delta h}{Tg} \quad \ldots\ldots\ldots\ldots\ldots\ldots 6$$

where Δh is the applied hydraulic head across the geotextile.

With change in confining stress (σ_c) then the thickness of the geotextile (Tg) will vary. If the relationship between Tg and σ_c is known then it is possible to rewrite Eqn. 5 as follows:

$$\Delta h = Tg\left(\frac{V}{V_1}\right)^n \quad \ldots\ldots\ldots\ldots\ldots\ldots 7$$

where Tg is the thickness of the geotextile at the confining stress σ_c and corresponding to average flow velocity of V.

Inspection of Eqn. 7 indicates that if the variations of Tg, V_1 and n with confining stress (σ_c) are known then the head loss, either across or along the geotextile, can be calculated for any required average flow velocity V at any confining pressure. Since head loss is the most critical parameter in any filter design, it is suggested that this should be employed in preference to Darcy's coefficient of permeability.

APPARATUS AND PROCEDURES FOR THE MEASUREMENT OF COMPRESSIBILITY AND HYDRAULIC CONDUCTIVITY

The tests are carried out using modified consolidation cells of the type developed by Rowe and Barden (9). The cells are 152 mm internal diameter and can accommodate specimens of about 50 mm thickness. Two versions of the apparatus were developed to measure the relation between flow rate and compressibility, Fig. 2. In one version flow rates are measured across the geotextile while in the other the flow rates are measured along the geotextile in an outward radial direction. To ensure that both the flow of water and load applied to the specimen are uniformly distributed, spreader plates with 42 per cent contact area are placed on the top and bottom of the specimen, as shown in Fig. 2. The object of this test arrangement is to assess the properties of nonwoven and composite geotextiles when formed into multiple layers, however, combinations of soil and geotextile may also be used in the test apparatus. Woven geotextiles cannot be tested in this way as it cannot be ensured that their pores are aligned and that a greater degree of blocking of pore space is not occuring.

The principal modifications that were carried out on the standard Rowe cells were in relation to the inlet and outlet pipes to ensure sufficient flow and to allow the insertion of filter tips to act as piezometers. The piezometers are connected directly to external manometers to allow measurement of the applied hydraulic heads within the cell. The direct measurement of hydraulic head internallly distinguishes the design of this apparatus from those of similar design (6, 10) in which only externally applied heads were measured and assumed to be equal to the internally applied heads.

Great care was taken to control the temperature and condition of the water used in the tests. In all cases de-aired water at 20 °C was used with a dissolved oxygen content of less than 5 per cent. To reduce the suspended solid content of the water, two commercially available cartridge filters having a 1 μm opening size were inserted into the supply line between the de-aired water tank and the Rowe cells. In addition, the quantity of water flowing through the test specimens was strictly limited to generally less than 100 litres to avoid the build up of bacteria.

The test specimens were cut to the same diameter as the internal dimension of the Rowe cell. For radial flow, the specimens also had a 12 mm diameter central hole cut out. Pretreatment of the specimens involved soaking them in de-aired water for a period of one week prior to testing. They were then transferred to the Rowe cells under water. The heights of the unloaded specimens were then measured in the cells and a first load increment of 34 kN/m^2 applied. After 12 to 18 hours hydraulic flow rate tests were carried out at hydraulic gradients across the specimens of between

0.5 and 5.0. In each case the flow rate was established by measuring the time to pass 1 litre of water through the specimen or the quantity of flow passing in 8 hours, with three separate measurements made at each hydraulic gradient. The pressure on the specimen was then doubled and the flow rate tests repeated. This was continued at different pressures up to 483 kN/m^2, corresponding to the maximum available pressure supply in the laboratory.

RESULTS OF COMPRESSIBILITY AND HYDRAULIC CONDUCTIVITY TESTS

Three geotextiles were tested which are all commercially available and widely used in civil engineering works. The material chosen represent different geotextile types and are listed in Table 1 together with their basic characteristics.

The compressibility data obtained from each of the four types of geotextile are shown in Fig. 3. They are presented in terms of the relationship between the average thickness and void ration of individual layers against confining stress. The average thickness is calculated by dividing the total thickness of the test specimen by the number of layers within it. The very great difference in behaviour between the melt bonded non-woven geotextile and the needle punched non-woven and composite geotextiles is apparent between the various structural types.

The changes in average pore space (O) may be estimated from the data using Equation 1. Alternatively, compressibility coefficients may be derived from the thickness against confining stress plots thus enabling a classification of the geotextile compressibility to be made, as was suggested in relation to the filter criterion. Such classification could separate the geotextiles into various general groupings on a much more rational basis than the present "thick" or "thin" groupings. It could also allow new materials, particularly composite geotextiles to be more simply classified.

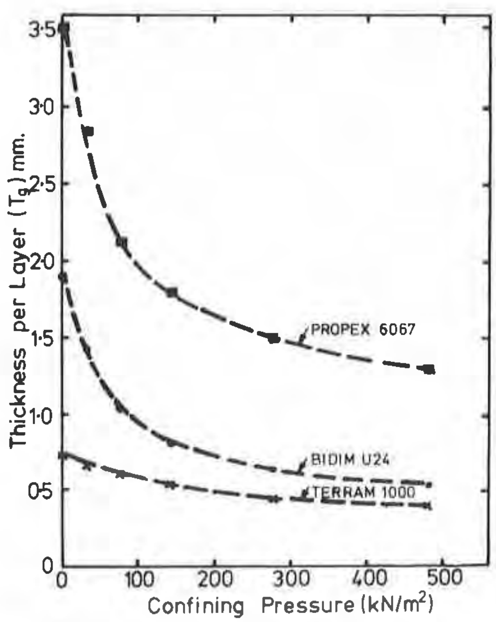


Fig. 3. COMPRESSIBILITY OF GEOTEXTILES TESTED.

Table 1. Basic Characteristics of Geotextiles Tested.

CHARACTERISTIC	TERRAM 1000	BIDIM U24	PROPEX 6067
Method of Construction	Non-woven Melt bonded filaments	Non-woven needle punched filaments	Composite Woven and needle punched
Polymer(s) Composition	67% Polypropylene 33% Polyethylene	100% Polyester	100% Polypropylene
Specific Gravity	0.9	1.39	0.91
Weight/Unit Area (g/m^2)	140	210	650
O_{90} (microns)	110	125	246
Nominal Thickness (mm)	0.7	1.9	3.5

Much work remains to be done but the technique shows considerable promise and is likely to lead to a more general approach to geotextile filter design which will allow the influence of compression to be assessed.

The data on hydraulic conductivity relating to flow across and along each of the three geotextiles studied are shown in Figs.4 and 5 respectively. The average flow velocities across the geotectiles (V_{1P}) were calculated by dividing the quantity of flow per unit time by the area of the specimen. The average radial flow velocities along the geotextile (V_{1R}) were calculated on the basis of the following equation:

$$V_{1R} = \frac{Q \ln R/r}{2\,To(R - r)} \quad \ldots\ldots\ldots\ldots\ldots\ldots 8$$

where Q is the quantity of flow per unit time, R, r are the external and internal radii of the specimens respectively To is the overall thickness of the test specimen at any confining stress.

The values of the coefficient n and average flow velocities across and radially along the geotextiles at hydraulic gradients of unity (V_{1P} and V_{1R}) are shown in Figs, 6 and 7 respectively. These once again clearly

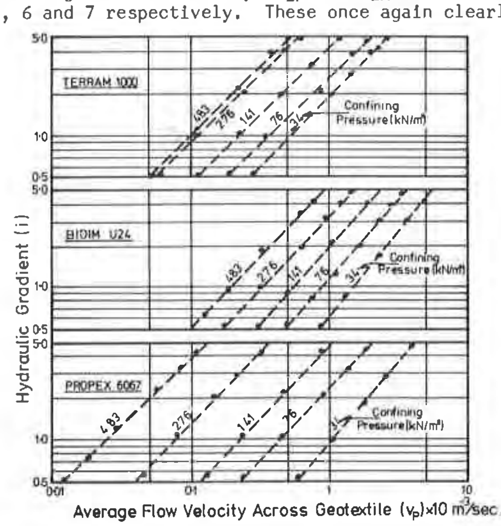


Fig. 4. RELATIONSHIP BETWEEN FLOW ACROSS GEOTEXTILES AND HYDRAULIC GRADIENT.

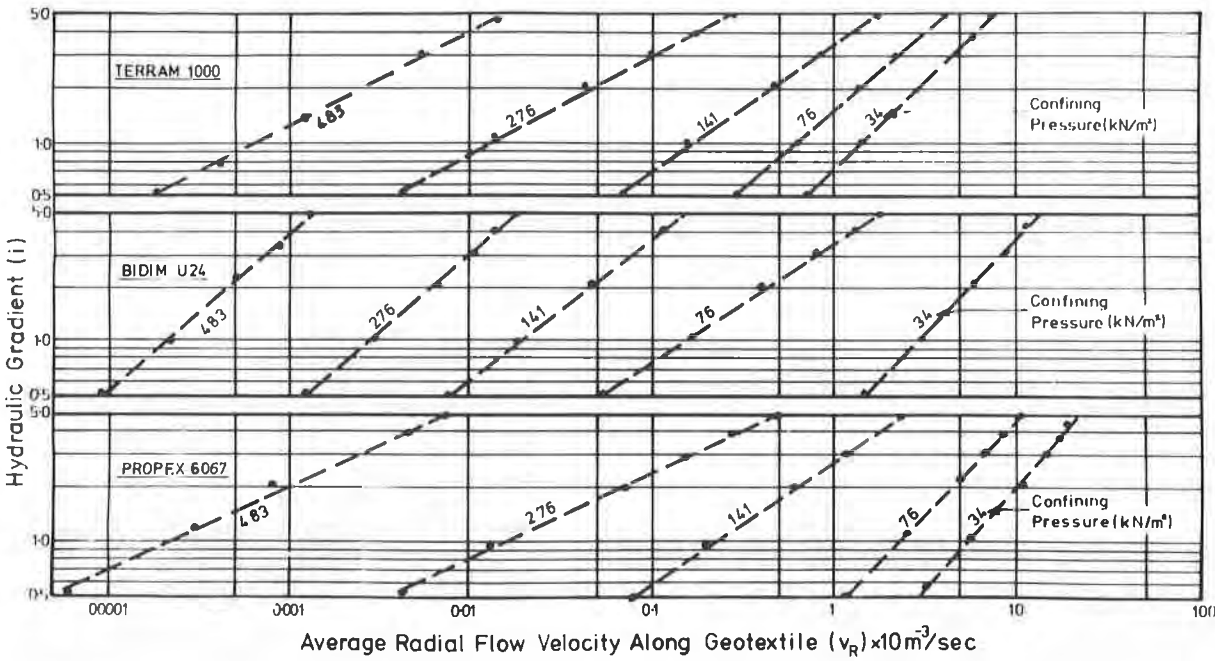


Fig. 5. RELATIONSHIP BETWEEN FLOW RADIALLY ALONG GEOTEXTILES AND HYDRAULIC GRADIENT.

Fig. 6. VARIATION IN V_{IP} and n WITH CONFINING STRESS.

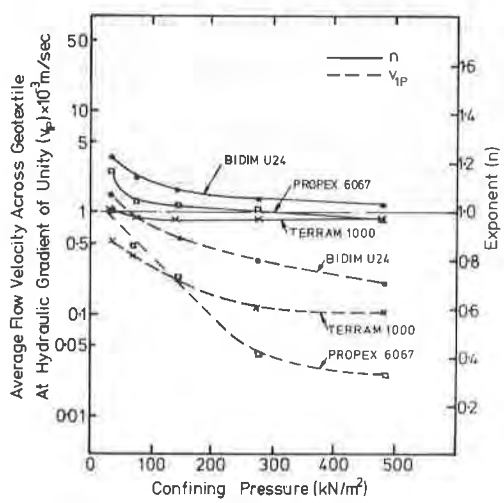
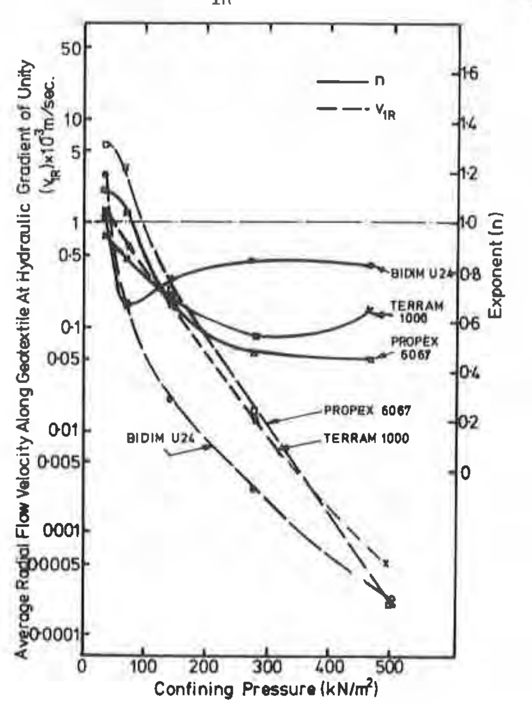


Fig. 7. VARIATION IN V_{IR} and n WITH CONFINING STRESS.



distinguish between the various types of geotextile and show that the relationship between flow capacity and confining stress is unique to each mode of construction and direction of flow. It is suggested that a measure of a geotextile compressibility would largely reflect the structural changes which are controlling its hydraulic conductivity and that this should be used to characterise their behaviour.

All the flow data indicate that n varies with confining stress for a particular geotextile subject to flow in a particular direction and confirm that Darcy's Law does not hold for these materials. The results also show that the flow velocities at hydraulic gradients of unity (V_{1P} and V_{1R}) reduce with increase in confining stress. It is suggested that it is more appropriate to characterise geotextiles by these confined flow data than by the single unloaded flow value that is presently used, since the latter is a maximum flow condition which does not relate to operational flow conditions occurring when the geotextile is compressed in-soil.

It should be noted that the value of flow at any given confining stress as obtained from these tests will not necessarily be a lower bound value since in-soil there will be clogging and blocking of the pores which are likely to further reduce the hydraulic conductivity of the geotextiles.

The values obtained from the tests with flow across the compressed geotextile can be used directly to estimate head losses (Δh) across them in preliminary design. The radial flow test data is specific to the geometry of the test specimens and cannot be generally applied to calculate head losses along geotextiles. To overcome this and to obviate the problems of side leakage, difficulty of cutting specimens as well as allow directional in-plane testing, a new version of the apparatus is being developed. It tests square specimens in undirectional flow; test data from this can be directly applied to calculate head losses in the same manner as the data from tests which flow across the geotextile. Also a single layer of geotextile may be placed in-soil and tested in the apparatus. The head losses across and along the system can be measured and related to flow in exactly the same way as is used in the present test set up. In this way, blocking and clogging effects may be measured. The new version of the apparatus will in many ways be similar to that previously developed by Fierz et al (11).

CONCLUSIONS

1. From a consideration of existing filter criteria and test data obtained from a specially modified Rowe cell apparatus, it is apparent that non-woven and composite geotextiles require a more open structure than simple woven materials for the same filter drain performance.

2. The presently available filter criteria can be simplified and modified to include the influence of geotextile compressibility.

3. The hydraulic conductivity of geotextiles should not be related to the permeability of soils using Darcy's coefficient of permeability. It is suggested that the head loss through the geotextile should be the limiting factor and this can be easily computed from a more general flow law and a knowledge of the influences of material compressibility on flow.

4. Overall the study has demonstrated that compressibility can significantly influence the filter function of geotextiles and is, moreover, an important parameter for characterising their behaviour.

ACKNOWLEDGEMENTS

This paper is published by kind permission of the Director, Transport and Road Research Laboratory.

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Transmissivity of Geotextiles and Geotextile/Soil Systems
Transmissibilité transversale de géotextiles et systèmes géotextile/sols

The necessity of determining in-plane permeability or transmissivity is critical for all applications of in-plane water flow using geotextiles. This paper describes a test method and apparatus which accurately models in-situ behavior, uses relatively large samples (61.0 cm x 30.5 cm), has the capability of testing under high normal pressures and can examine the performance of the fabric sandwiched between soils as occurs in the natural situation. It was found that transmissivity decreases exponentially with normal pressure but, for the fabrics tested in this study, is never completely cut off. Pressures up to 96 kPa (equivalent to approximately 10 m of water) were evaluated. Typically, the transmissivity of a 600 gm/sq. m weight geotextile is equivalent in its hydraulic capability to 2.5 cm of sand. The presence of clay soil on both sides of the fabric somewhat limits its performance. It is concluded that bulky geotextiles of the type evaluated in this study are well suited to transmit water in the in-plane direction.

La nécessité de déterminer la transmissibilité transversale est importante dans toutes les applications de l'ecoulement de nappes d'eau transversales utilisant les geotextiles. Cet article décrit une méthode et l'appareil qui (1) très précisément représentent le comportement "in-situ", (2) utilise relativement de grands échantillons (61,0 cm x 30,5 cm), (3) a la capacité de tester sous hautes pressions normales, et (4) peut examiner la performance du tissu pressé entre des sols comme cela se présente dans la situation naturelle. Il a été trouvé que la transmissibilité diminue exponentiellement avec une pression normale mais, pour les tissus experimentés dans cette étude, n'est jamais complètement coupée. Des pressions jusqu'à 96 kPa furent évaluées. Typiquement, la transmissibilité d'un poids de 600gm/m^2 de geotextile est équivalante dans sa capacité hydraulique à 2,5cm de sable. La présence d'argile des deux côtés de tissu diminue un peu sa performance. En conclusion, les géotextiles volumineux du genre évalués dans cette étude sont bien équipés pour transmettre de l'eau.

INTRODUCTION

The essential hydraulic aspects of geotextiles are their ability to pass water perpendicular to their plane and, for some, their ability to move water within their plane. This latter aspect of in-plane permeability of geotextiles, first noted by Leflaive and Puig (1), is the focus of this paper. The uses for geotextiles to transmit water in this manner are necessary in a number of practical situations. These situations include the following, which underscore the importance of the subject. See Giroud (2) for further details.

- interceptors for road drains
- drain wells to decrease consolidation times
- eliminate hydrostatic pressure behind retaining walls
- dissipate seepage forces in earth and rock slopes
- as chimney drains in earth dams
- as drainage galleries in earth dams
- dissipate pore water pressures in embankments and fills
- dissipate pore water pressures in fabric retaining walls
- dissipate pore water pressures in encapsulated soil systems
- transmit water from beneath railroad ballast
- guide subsurface water away from frost action areas
- break capillary zones in frost heave areas
- dissipate gas pressures beneath pond liners

The particular property of the geotextiles which will be measured in this study is called transmissivity (θ), which is the in-plane permeability multiplied by the fabric thickness and carries the units of area per unit time. The reason for including the thickness in the value θ is that the pressures exerted on the fabric cause a fabric compression, which can be quite large, and the term transmissivity is a method of including this characteristic. In most cases "θ" will be plotted against the stress "σ" exerted against the fabric thereby simulating in-situ conditions as close as possible.

The paper will describe the experimental apparatus which was used and then present a number of test results using fabric alone and in combination with soil over a wide range of normal stresses. The conclusions will include a summary of the entire project to date.

EXPERIMENTAL PROCEDURE

The number and variety of physical configurations that one can devise to measure transmissivity are many, e.g., see reference (3) for the Swiss approach to these measurements. All, however, are based upon Darcy's Law with a constant, or falling head system and are somewhat modeled after laboratory permeability measurements of soil. One added, and necessary, feature is that the fabric must be subjected to a measurable normal stress.

The system we have devised is shown in the photograph and diagrams of Figure 1. Here, the lower base plate measuring 77.5 cm x 30.5 cm supports the fabric, which in turn supports the upper assembly consisting of an inverted aluminum channel connected to a plastic water container. The upper aluminum channel, which supports the load used to develop normal stress in the fabric, measures 61.0 cm x 30.5 cm. After the load is placed, the assembly is bolted together so that water flow can only occur horizontally in the fabric between the aluminum plates. Water is supplied via the plastic tank at constant heads of 7.6, 15.3, 22.9 or 29.3 cm. At each value of normal stress, a series of four tests are performed at each of the available heads. The water which passes in the plane of the fabric is collected at the downstream end of the apparatus in a trough and measured per unit time. Calculations then proceed using the following relationships:

$$q = k_p \; i \; A$$

$$q = k_p \frac{h}{L} (w \; t)$$

$$\frac{q}{w} = (k_p \; t) \frac{h}{L}$$

$$\theta = (q/w)/(h/L)$$

where

q = flow rate (m^3/s)
k_p = in-plane permeability (m/s)
h = head forcing flow (m)
L = length of fabric (m)
w = width of fabric (m)
t = thickness of fabric (m)
$\theta = k_p t$ = transmissivity (m^2/s).

Graphically, the procedure reduces to plotting (q/w) versus (h/L) values and measuring the resulting slope for the desired value of transmissivity (θ). This value is then plotted against the normal stress on the fabric (or soil/fabric/soil) system and behavioral trends are observed.

TESTS RESULTS

In all of the test results reported in this paper, the geotextile used was a needle punched, nonwoven, monofiliment polypropylene fabric (in various thicknesses) marketed by Crown Zellerbach under the trade name "Fibretex" (4).

The results are categorized into experiments which evaluate the effect of initial fabric thickness, high normal stresses, tests on the transmissivity of sand for comparison purposes and then a series of tests with soil on each side of the fabric. Additional detail is contained in reference (5).

(a) Effect of Normal Stress and Initial Fabric Thickness

This initial series of tests was aimed at determining the basic question of how much water a specific geotextile will transmit under varying amounts of pressure, i.e., normal stress. The preliminary step of measuring flow rate versus the head forcing flow is shown in Figure 2. This gives an indication of the amount of data scatter which, as seen, is relatively small. The slope of these lines is the transmissivity which results in Figure 3 when plotted against stress. Here it is seen that stresses up to 93 kPa (approximately 10 m of water pressure) were imposed and a near constant behavior in θ resulted at stresses beyond 30 kPa. The resulting flow as a function of initial fabric thickness was predictable in that the thicker fabrics resulted in proportionally higher transmissivities at all stress levels.

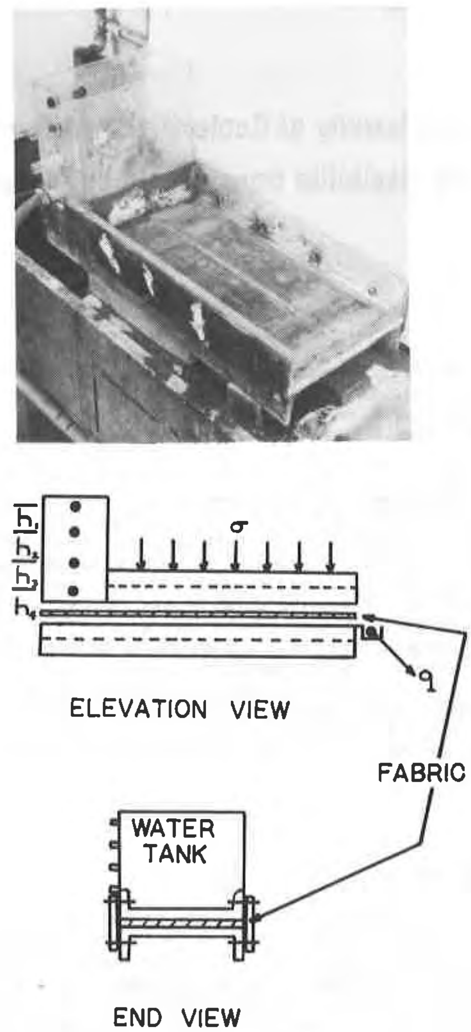

Fig. 1. - Photograph and Cross Section Diagrams of Transmissivity Measuring Device Used in This Study.

(b) Fabric Transmissivity Compared to Sand

In many of the applications mentioned in the introduction, fabrics are being used in place of sand. Thus it seems appropriate to compare the transmissivity of various thicknesses of sand to that of a geotextile. For this series of tests a medium graded sand (subrounded shape, effective size of 0.17 mm, coefficient of uniformity of 3.5, and classified as a SP soil) was used and tested at initial thicknesses of 1.3, 2.5 and 5.1 cm. The response is shown in Figure 4. Superimposed on the sand behavior curves is the 9.6 mm thick fabric response. Here it can be seen that fabrics are noticeably more compressible than sand and for typical highway drainage situations, say at 10 kPa stress, the 9.6 mm thick fabric is approximately equivalent to 5.1 cm of sand.

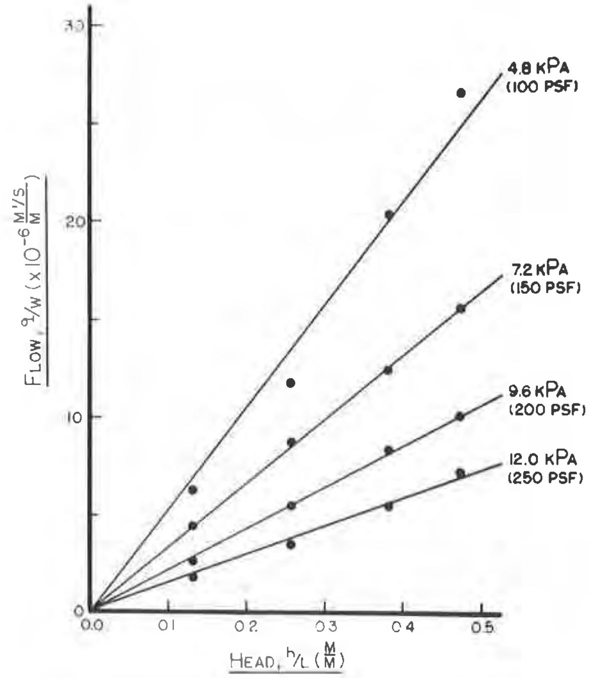

Fig. 2. - In-plane Water Flow versus Head Forcing Flow at Various Applied Stresses on Fabric (Note: the Slope of These Lines is the Transmissivity, θ).

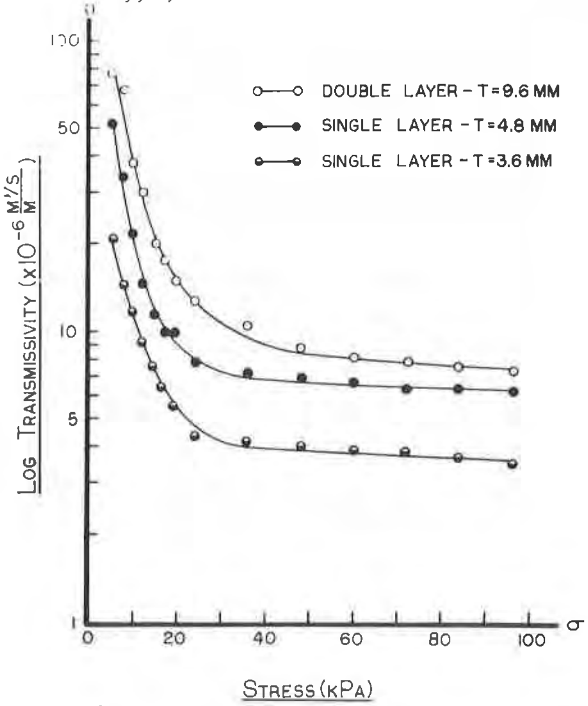

Fig. 3. - Transmissivity versus Applied Stress for Various Thicknesses of Nonwoven Needled Polypropylene Fabric.

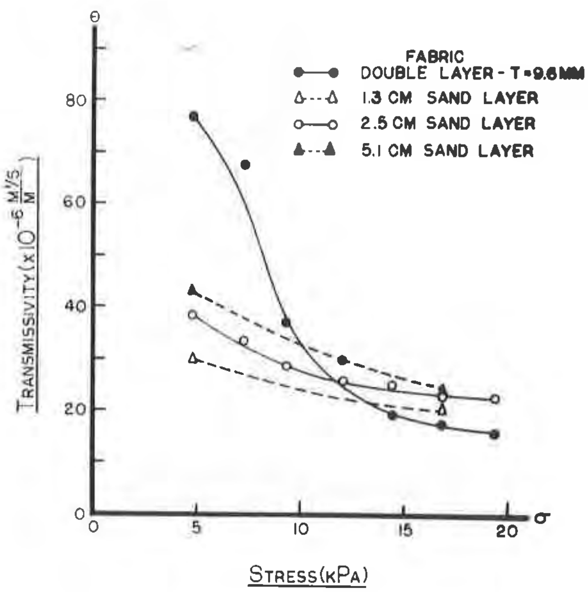

Fig. 4. - Transmissivity versus Applied Stress for Various Thicknesses of Sand Compared to Transmissivity of 9.6 mm Thick Fabric.

(c) Soil/Fabric/Soil Tests

The natural setting for fabrics is obviously with soil on each side of it and in order to assess the transmissivity under this condition a series of soil/fabric/soil tests were performed. A 0.7 cm thickness of soil was placed on the base of the test apparatus, then the fabric, and finally a second layer of soil 0.7 cm thick was placed on top of the fabric. The system was then assembled in the conventional manner. For these tests four different fine grained soils were blended with resulting particle size distribution curves shown in Figure 5. The transmissivity test results are given in Figure 6 for the fabric by itself, then for the four different blended soil/fabric/soil systems. Seen in these curves is the loss of flow capacity with increased amounts of clay soil content. There appears to be only a minor loss of transmissivity up to 15% clay content, but for higher clay contents, the loss is substantial. In these latter tests there was a noticeable infiltration of the soil into the fabric.

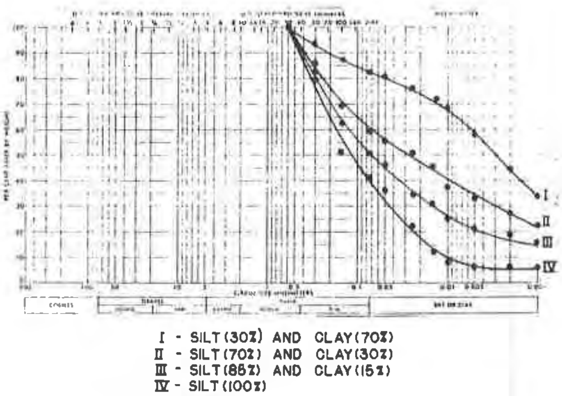

Fig. 5. - Particle Size Distributions of Soils Used in Soil/Fabric/Soil Portion of This Study - see Data of Figure 6.

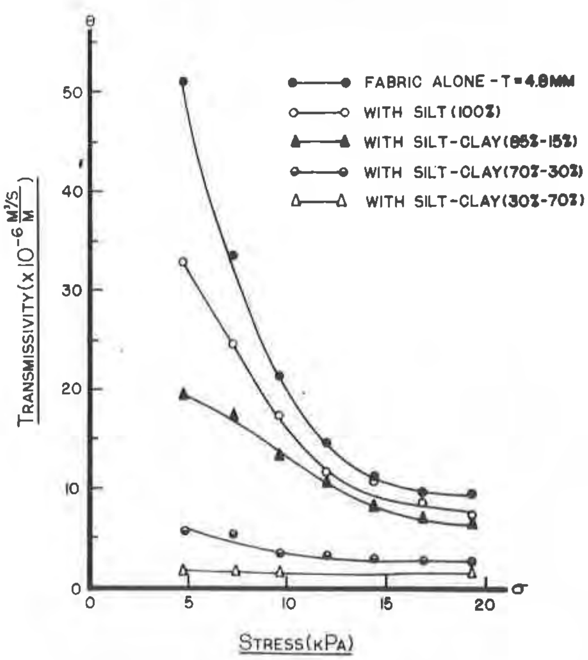

Fig. 6. - Transmissivity versus Applied Stress for Soil/Fabric/Soil Systems Compared to Fabric by Itself.

SUMMARY AND CONCLUSIONS

Using a laboratory test setup which closely simulates in-plane water flow in geotextiles in the field, the transmissivity (in-plane permeability times thickness) of a nonwoven, needle punched polypropylene fabric was evaluated. Major findings of the study were as follows:

- transmissivity decreases to a constant value with increasing normal stress applied to the fabric,
- the transmissivity is never cut off completely even up to stresses of 93 kPa,
- transmissivity increases with increasing initial fabric thickness,
- at normal stresses of 10 kPa, the transmissivity of a 9.6 mm initial thickness geotextile is equivalent to approximately 5.1 cm of sand,
- soil on both sides of a geotextile decreases its transmissivity, but only soils with very high clay contents (>30%) are considered to be detrimental.

In conclusion, it is felt that the use of bulky geotextiles is a logical and effective means of conveying water in a wide variety of construction applications. Many of these applications were previewed in the introductory section.

ACKNOWLEDGEMENTS

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Physical Characteristics of Geotextiles Definition Dimensions

Charactéristiques physiques des géotextiles Valeurs de définition

The paper brings forth a point of view on the geotextiles physical characteristics definition taking into account the elements with a specific textile nature that help establishing the value of these parameters. Considering the non-woven geotextiles strong compressibility the inclusion of this parameters, expressed by varying the non-woven fabrics thickness under loading, as a possible mean to be used in defining the characteristics comprised in the present paper is necessary. The final part exemplifies some of the stated characteristics for the MADRIL geotextiles made in Romania.

The paper conclusions underline the fact that the geotextiles specific anisotropy imposes the selection of the computing elements values on the basis of statistically processed laboratory tests.

L'article soumet à l'attention un point de vue sur la définition des caractéristiques physiques des géotextiles en tenant compte des éléments à spécifique textile qui concourent à l'établissement des valeurs des paramètres respectifs. En tenant compte de la compressibilité accentuée des géotextiles non-tissés on impose l'inclusion de ce paramètre aussi par la variation de l'épaisseur des géotextiles non-tissés sous charge, comme hypothèse pour définir les caractéristiques physiques renfermées dans l'article. Dans la partie finale on illustre par des exemples, pour les géotextiles de production roumaine MADRIL, quelques caractéristiques de celles énoncées déjà.

Dans les conclusions de cet article, on met en évidence le fait que l'anisotropie spécifique aux géotextiles impose le choix des valeurs des éléments de calcul à base de détermination de laboratoire expliquées du point de vue de la statistique.

1. INTRODUCTION

The use of textile materials in geotechnique imposes, besides defining them by means of values accepted for the textile testing, the establishment of notions that should offer the possibility of creating some correspondences between the two types of porous materials: geotextiles and soils.

Starting from this idea the authors point of view concerning the definition of geotextiles as building materials elements is presented taking into consideration the component elements (textile fibres and yarns) characteristics.

2. DEFINITION HYPOTHESIS

- geotextiles (woven or non-woven textiles) made of one or more than one polymers having strict manufacturing formulas;
- the circular fibres or yarn section;
- the initial thickness t_i considered σ_i = 0.5 kPa for non-woven and at σ_i = 1.0 kPa for woven fabrics;
- the reference unit 1 cm^2 or 1 cm^3;
- having in view that thickness represents an important characteristic especially with non-woven geotextiles the registering of thickness variation as hypothesis in defining the physical characteristics of this geotextile type is to be considered.

Figure 1 presents the thickness variation curve form for non-woven geotextiles made by needlepunching, the curve being drawn on the basis of a series of laboratory determinations performed in accordance with the STAS methodology (1).

Thus, three domains can be detected in this parameter variation, as it follows:

- the A domain (low loads) in which the thickness variation observes a law of the form

$$T_A = tg\,\alpha_1 \log\sigma + n_1 \Bigg|_{\sigma_i = 0.5\ kPa}^{\sigma_1} \qquad (1)$$

- the B domain (average loads - usually having the values 3 kPa $< \sigma <$ 300 kPa) in which the variation of T observes a hyperbolic law of the form

$$T_B = \frac{k^2}{\log\sigma}\Bigg|_{\sigma_1}^{\sigma_2} \qquad (2)$$

- the C domain (high loads) in which the thickness variation observes a law of the form

$$T_C = tg\,\alpha_2 \log\sigma + n_2 \Bigg|_{\sigma_2}^{\sigma_{max}} \qquad (3)$$

where, σ_{max} represents the loading for which $T=f(\sigma)$= = constant.

The values of the terms α_1, n_1, k, α_2, n_2 from the expressions (1), (2) and (3) can be determined by

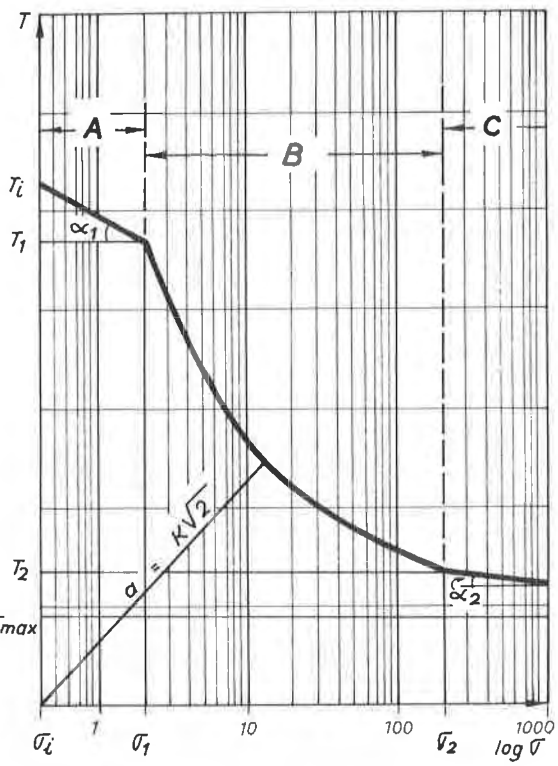

Fig.1. The variation of the non-woven geotextile thickness depending on the σ loading.

laboratory tests depending on the geotextile mass, the fibres characteristics and the technological parameters of the textile fabrication.

3. PHYSICAL CHARACTERISTICS

3.1. Nominal Volume (V_u)

The nominal volume represents the volume of one cm^2 of geotextile when the σ_i loading is applied to it

$$V_u = T_i \qquad (4)$$

T_i being the thickness at the initial loading.

For the case when $\sigma \neq \sigma_i$ the expression (4) becomes

$$V_\sigma = T_\sigma \qquad (4')$$

where T_σ is the geotextile thickness in the case of a certain "σ" loading (for the non-woven geotextiles, see figure 1).

3.2. Nominal Mass (μ_s)

The nominal mass represents the mass of one cm^2 of geotextile

$$\mu_s = 10^{-4}\ M \qquad (g/cm^3) \qquad (5)$$

M being the mass per unit area expressed in g/m^2.

3.3. Specific Mass (μ_{sp})

The specific mass represents the mass of one cm^3 of geotextile when the σ_i loading is applied to it

$$\mu_{sp} = \frac{\mu_s}{T_i} \qquad (g/cm^3 \qquad (6)$$

Note: Although the specific mass thus defined is in fact the geotextile specific weight the authors will keep the name of "specific mass" so that no confusions will be possible between it and the polymer specific weight (γ_p).

3.4. Nominal Volume of Fibres or Yarns (v_f)

The nominal volume of fibres or yarns represents the volume of the fibres or yarns having a mass equal to the geotextile nominal mass

$$v_f = \frac{\mu_s}{\gamma_p} \qquad (cm^3) \qquad (7)$$

γ_p being the specific weight of the polymer the fibre or the yarn is made of.

3.5. Specific Volume of Fibres or Yarns (V_f)

The specific volume of fibres or yarns represents the volume of the fibres or yarns comprised in one cm^3 of geotextile when the σ_i loading is applied

$$V_f = \frac{\mu_{sp}}{\gamma_p} \qquad (cm^3) \qquad (8)$$

For $\sigma \neq \sigma_i$, V_f becomes $V_{f\sigma}$ (the fibres or yarns volume in the case of a certain loading)

$$V_{f\sigma} = \frac{\mu_s}{T_\sigma \gamma_p} \qquad (cm^3) \qquad (9)$$

3.6. Nominal Length of Fibres or Yarns (l_f)

The nominal length of fibres or yarns represents the fibre or yarn length having a mass equal to the geotextile nominal mass

$$l_f = 10^5 \frac{\mu_s}{\lambda} \qquad (cm^{-1}) \qquad (10)$$

λ being the fibres or yarns fineness (linear density of fibres or yarns) expressed in Tex (3).

3.7. Specific Length of Fibres or Yarns (L_f)

The specific length of fibres or yarns is the fibre or yarn length having a mass equal to the geotextile specific mass

$$L_f = 10^5 \frac{\mu_{sp}}{\lambda} \qquad (cm^{-2}) \qquad (11)$$

If for the non-woven geotextiles L_f is defined as the length of fibre comprised in a certain volume of the geotextile this characteristic has the meaning of denseness.

3.8. Specific Denseness (∂)

The specific denseness represents the length of fibre (l_f) comprised in the nominal volume (V_u)

$$\partial = L_f = 10^5 \frac{\mu_s}{T_i \lambda} \qquad (cm^{-2}) \qquad (12)$$

If $\sigma \neq \sigma_i$ then the denseness at a certain loading will be expressed as:

$$\partial_\sigma = 10^5 \frac{\mu_s}{T_\sigma \lambda} \qquad (cm^{-2}) \qquad (13)$$

For woven geotextiles the denseness (∂_w) can be expressed according to the textile regulations (2), namely:

$$\partial_w = 10 \frac{N}{l} \qquad (yarns/10\ cm) \qquad (14)$$

where: N = number of yarns; l = length of the sample (in cm).

In the case of woven fabrics ∂_w has two values, one for each of the two winding directions: warp and filling.

3.9. Nominal Side Surface of Fibres (A_l) - for Non-woven Fabrics

The nominal side surface of fibres represents the surface of fibres having a length equal to the nominal length

$$A_l = \pi d_f \times l_f$$

where d_f = the diameter of fibres

$$d_f = 2\sqrt{\frac{\lambda}{10^5 \pi \gamma_p}} \quad (cm)$$

$$A_l = 2\mu_s \sqrt{\frac{10^5 \pi}{\lambda \gamma_p}} \quad (cm^2) \qquad (15)$$

3.10. Specific Surface of the Geotextile (A_g) - for Non-woven Fabrics

The geotextile specific surface represents the side surface of the fibres comprised in one cm^3 of fabric when the σ_i loading is applied to it

$$A_g = 2\mu_{sp} \sqrt{\frac{10^5 \pi}{\lambda \gamma_p}} \quad (cm^{-1}) \qquad (16)$$

For $\sigma \neq \sigma_i$ A_g becomes A_σ

$$A_\sigma = 2\frac{\mu_s}{T_\sigma} \sqrt{\frac{10^5 \pi}{\lambda \gamma_p}} \quad (cm^{-1}) \qquad (17)$$

In the case when the specific surface A_g is related to the fibres specific volume (V_f) the specific volumetric surface of the geotextile (A_f) is obtained

$$A_f = \frac{4}{d_f} \qquad (18)$$

In the case when the specific surface A_g is related to the fibres specific weight the specific surface of the geotextile mass ($A\mu$) is obtained

$$A\mu = 2\sqrt{\frac{10^5 \pi}{\lambda \gamma_p}} \quad (cm^2/g) \qquad (19)$$

$$A_f = \gamma_p \cdot A\mu \qquad (20)$$

3.11. Fineness Index of the Non-woven Geotextiles (F)

The fineness index of the non-woven geotextiles is defined as the inverse of the section of fibres the non-woven fabric is made of

$$F = 10^5 \frac{\gamma_p}{\lambda} \quad (cm^{-2}) \qquad (21)$$

The denseness dependence on fineness index is expressed by

$$\partial = V_f \cdot F \qquad (22)$$

3.12. Specific Porosity of Non-woven Geotextile (n_i)

The specific porosity represents the ratio between the pores volume of a non-woven fabric and the fabric volume when σ_i loading is applied.

$$n_i = (1 - \frac{\mu_{sp}}{\gamma_p})\ 100 \quad (\%) \qquad (23)$$

For a certain $\sigma \neq \sigma_i$

$$n = (1 - \frac{\mu_s}{T_\sigma \gamma_p})\ 100 \quad (\%) \qquad (24)$$

The porosity dependence on denseness is expressed by

$$n = (1 - \frac{\lambda\ \partial_\sigma}{10^5 \gamma_p})\ 100 \quad (\%) \qquad (25)$$

3.13. Specific Void Ratio (e_i) - for Non-woven Geotextiles

The specific void ratio represents the ratio between the pores volume of a non-woven fabric and the fabric fibres volume when the σ_i loading is applied

$$e_i = (\frac{\gamma_p}{\mu_{sp}} - 1) \qquad (26)$$

For a certain $\sigma \neq \sigma_i$

$$e = (\frac{T_\sigma \gamma_p}{\mu_s} - 1) \qquad (27)$$

and

$$\frac{n(\%)}{100} = \frac{e}{1 + e} \qquad (28)$$

3.14. Openings Dimensions of the Woven Geotextiles

In the case of woven fabrics the porosity problem is dealt with differently from the non-woven fabrics one.

The openings are not to be found in the material mass but are formed by the warp yarns intersection with the filling yarns.

Analysing the regular case of the woven fabrics with a cloth texture the fact can be admitted that the openings are of a rectangular form. If: N_w = number of warp yarns; N_f = number of filling yarns; b_w = the warp-wise length of sample; b_f = the fillingwise length of sample; λ_w = the warp yarns fineness; λ_f = the filling yarns fineness; w = the distance between two warp yarns; f = the distance between two filling yarns

$$w = \frac{b_f}{N_w} - 2\sqrt{\frac{\lambda_w}{10^5 \pi \gamma_p}} \quad (cm) \qquad (29)$$

$$f = \frac{b_w}{N_f} - 2\sqrt{\frac{\lambda_f}{10^5 \pi \gamma_p}} \quad (cm) \qquad (30)$$

From the values thus calculated for w and f the lowest one is to be considered as characteristic dimension of the openings.

The characteristic dimension of woven fabrics openings can as well be defined by the characteristic surface of the openings (Ω)

$$\Omega = w \cdot f \quad (cm^2) \qquad (31)$$

using the values w and f from the relations (29) and (30).

3.15. Openings Quantum (Porosity) of Woven Geotextiles (n_w)

The woven geotextiles porosity represents the ratio between the openings surface and the surface of the woven material

$$n_w = \frac{\sum \Omega}{b_w \cdot b_f} 100 \qquad (\%) \qquad (32)$$

or

$$n_w = \frac{(N_w - 1)(N_f - 1) w \cdot f}{b_w \cdot b_f} 100 \qquad (\%) \qquad (33)$$

3.16. Woven Geotextiles Openings Ratio (e_w)

The woven geotextiles openings ratio represents the ratio between the openings surface and the surface occupied by the fibres that make up the woven fabric

$$e_w = \frac{\sqrt{10^5 \pi \gamma_p}}{2} \frac{(N_w - 1)(N_f - 1) w \cdot f}{b_w N_w \sqrt{\lambda_w} + b_f N_f \sqrt{\lambda_f}} \qquad (34)$$

$$e_w = \frac{\sqrt{10^5 \pi \gamma_p}}{200} \frac{n_w (\%) b_w \cdot b_f}{b_w N_w \sqrt{\lambda_w} + b_f N_f \sqrt{\lambda_f}} \qquad ((35)$$

4. PHYSICAL CHARACTERISTICS OF MADRIL(R) GEOTEXTILES

MADRIL(R) geotextiles are non-woven fabrics made of polypropylene fibres of 0.66 Tex (MADRIL(R) M) or of 2.0 Tex (MADRIL(R) V) mechanically bonded by needle-punching.

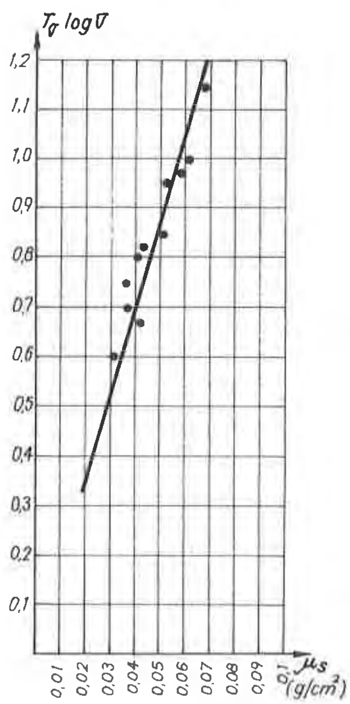

Fig.2. The dependence between the MADRIL(R) M geotextile thickness for different loadings σ and the mass (μ_s) (for σ in the domain $T_\sigma \log \sigma = k^2$).

Figure 2 shows the dependence existing between the MADRIL M geotextile thickness and its nominal mass (μ_s) in the case of the domain B (average loadings). The existence of a proportionality between the nominal mass and the product ($T_\sigma \times \log\sigma$) is to be noticed.

One can notice in figure 2 that

$$T_\sigma = \frac{tg\alpha}{\log\sigma} \mu_s \qquad (cm) \qquad (36)$$

Table 1 presents the MADRIL M geotextile thickness variation (T_σ) as a function of μ_s and σ.

Table 1. The thickness of the MADRIL(R) M geotextile for various loadings σ.

σ (KPa)	μ_s (g/cm²)					
	0.02	**0.03**	**0.04**	**0.05**	**0.06**	**0.07**
0.5	0.355	0.458	0.561	0.664	0.767	0.870
5.0	0.206	0.309	0.412	0.515	0.618	0.721
10.0	0.175	0.263	0.350	0.438	0.525	0.613
15.0	0.161	0.241	0.322	0.402	0.483	0.563
20.0	0.152	0.228	0.304	0.380	0.456	0.532
25.0	0.146	0.219	0.292	0.365	0.438	0.511
50.0	0.130	0.195	0.259	0.324	0.389	0.454
100.0	0.117	0.175	0.233	0.292	0.350	0.408

Table 2 presents some of the physical characteristics of the geotextile MADRIL(R) M calculated according to the relations presented in section 3.

Table 2. Physical characteristics of the MADRIL(R) M geotextile for different σ loadings.

$d_f = 31\ \mu m$	$\frac{f}{10^5} = 1.36\ cm^{-2}$		$A_f = 1310\ cm^{-1}$		$A_\mu = 1440\ cm^2/g$
	$\mu_s = 0.2 \div 0.7\ g/cm^2$				
σ (KPa)	∂_σ (cm^{-2})	n_σ (%)	ℓ	A_σ (cm^{-1})	$V_{f\sigma}$ (cm^3)
0.5	$1.13 \cdot 10^4$	91.72	11.08	$1.08 \cdot 10^2$	$8.30 \cdot 10^{-1}$
5	$1.46 \cdot 10^4$	89.30	8.35	$1.40 \cdot 10^2$	$1.07 \cdot 10^{-1}$
10	$1.71 \cdot 10^4$	87.47	6.98	$1.64 \cdot 10^2$	$1.25 \cdot 10^{-1}$
15	$1.87 \cdot 10^4$	86.30	6.30	$1.79 \cdot 10^2$	$1.37 \cdot 10^{-1}$
20	$1.97 \cdot 10^4$	85.56	5.93	$1.89 \cdot 10^2$	$1.45 \cdot 10^{-1}$
25	$2.06 \cdot 10^4$	84.91	5.63	$1.97 \cdot 10^2$	$1.50 \cdot 10^{-1}$
50	$2.32 \cdot 10^4$	83.00	4.88	$2.22 \cdot 10^2$	$1.70 \cdot 10^{-1}$
100	$2.57 \cdot 10^4$	81.17	4.31	$2.46 \cdot 10^2$	$1.88 \cdot 10^{-1}$

In this table the values written next to the reference loadings σ_l = 0.5 kPa represent the specific characteristics value of the MADRIL M geotextiles (∂, n_i, e_i, A_g and V_f) for μ_s = 0.05 g/cm^2.

5. CONCLUSIONS

The defining of the main physical characteristics of geotextiles, some of them being well known both to the textile specialists and to the geotechnicians, aims at bringing forth the elements specific for the textile materials which determine the value of these characteristics.

The introduction of the non-woven geotextiles thickness variation depending on the normal loading on the textile material plane as a calculation hypothesis offers the possibility of a real evaluation of the stated characteristics variability.

It is useful to underline the fact that the relations of dependence defined in the present paper have a general character, having the possibility to be particularized depending on the laboratory results obtained by means of direct measurements on various types of geotextiles. Moreover for a more precise evaluation of the characteristics values of a certain type of geotextile it is absolutely necessary to perform a detailed laboratory study imposed by the well known anisotropy of these materials.

In this context it is desirable that the various parameters values that contribute to the establishment of the physical characteristics values should be selected as a result of a statistical analysis.

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The Use and Behavior of Geotextiles in Underdrainage Systems of Gold Mine Tailings Dam in South Africa

L'emploi et la réaction des géotextiles envers les systèmes pour les sous—égouts des residus des mines d'or en Afrique du Sud

The paper describes the design and function of underdrainage systems in gold tailings dams.

The cost effectiveness, practical implications and potential problems associated with the incorporation of geofabrics in these underdrains are discussed.

Reference is made to the observed performance of geofabric underdrains in tailings dams which have been operational for the past six years.

The paper concludes that geofabrics can be successfully used in this application and provide both economic and constructional benefits when compared to natural drainage media.

Le papier décrit le dessin et la fonction d'un système de sousdrainage dans un barrage de retenue des tailings d'or.

On discute l'efficacité du coût, les implications pratiques et les problèmes potentiels liées à l'incorporation des géotextiles dans ces sousdrains.

On fait mention du fonctionnement des géotextiles observé sous des drains dans les barrages de retenue des tailings qui ont été en plein activité dupuis six ans.

Le papier dit en terminant que des géotextiles peuvent être employé avec succès dans cette application et qu'ils donnent en comparaison avec des moyens de drainage naturel des avantages economiques et de construction.

1 INTRODUCTION

For the past six years, the Authors of this paper have been involved in the design of tailings dams for Gold Mines located in the north-eastern area of the Orange Free State Province of South Africa. Increasing land costs in the Orange Free State Goldfields, as the area is commonly called, have contributed to the need for depositing high tonnages of gold tailings on relatively small land areas.

The flat nature of the local topography and engineering characteristics of the gold tailings material enable the tailings dams to be constructed employing what is known as the 'ring dyke' method. This method allows for point deposition at predetermined 'delivery stations' around the dam perimeter. Tailings is deposited in a slurry form with a 1:1 water:solids ratio by mass. During the day shift the tailings is deposited within the ring dyke, a system of adjoining paddocks around the dam perimeter. The coarse fraction of the tailings is allowed to settle out within the ring dyke between delivery stations and the fine fraction is decanted in a slurry form into the dam basin, thereby allowing the ring dyke to be raised using the coarser fraction of the tailings only. When building of the ring dyke is not being carried out, typically during the night, the tailings is deposited directly into the dam basin. Perimeter freeboard is achieved by ensuring that the rate of rise of the ring dyke exceeds that of the tailings within the dam basin.

The existing trend of increased deposition tonnages on relatively small land areas has two major implications, viz:

- a substantial increase in the quantity of water deposited per hectare, and
- a substantial increase in the rate of rise in vertical elevation of the tailings surface.

To ensure the stability of the ring dyke, the need for an underdrainage system beneath the tailings dams has become necessary in order to:

- accelerate the rate of consolidation and consequent gain in strength of the material deposited within the ring dyke, thus allowing the outer walls to be constructed at a relatively higher rate of rise
- accelerate the rate of dissipation of water pressure against the ring dyke by drawing down the phreatic surface within the tailings mass, thereby improving the stability of the outer walls

The paper describes the design philosophy adopted and discusses the detailed design, economic and constructional considerations which have led to the successful use of geotextiles in underdrainage systems of gold tailings dams in the Orange Free State Goldfields.

2 DESIGN PHILOSOPHY

The underdrainage system is designed to cater for both above surface and subsurface flow components. A brief description of these components are:

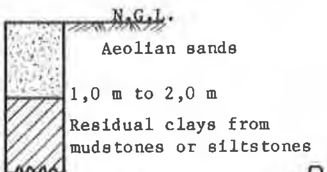
2.1 Above surface component

The above surface component of flow is that which is generated by the development of a phreatic surface within the tailings mass. Conventional flow net theory is applied to determine the order of magnitude of this component, thus providing a means of designing an adequate drain section. The most commonly used drain section for drawing down the vertical flow component is a rectangular 'blanket' section with the width being much greater than the height (w ≃ 16h) to allow for a large area of interface.

The location of the blanket section is of major importance with respect to the stability of the ring dyke and is determined by considering factors such as the permeability of the tailings, the final height of the tailings dam and the location of the pool of surface water formed within the dam basin.

2.2 Subsurface component

A typical geological profile of the surface in situ soils underlying the Free State Goldfields tailings dams is as follows:



The permeability of the surface aeolian sands is greater than that of the underlying clay resulting in a tendency for the majority of seepage water to flow in a horizontal direction within the sand layer. In order to minimize seepage outside the boundaries of the tailings dam, a vertical cut-off trench drain section is considered necessary in the underdrainage system. The depth of the cut-off drain section is determined according to the depth of the aeolian sand layer. The optimum location for the cut-off drain section is around the extreme boundaries of the tailings impoundment, thus minimizing contamination of the surrounding environment with seepage water.

2.3 Conceptual design

In order to cater for both the above flow components and to simultaneously satisfy stability and environmental requirements, an underdrainage system as shown in Fig 1 has been adopted. Fig 1 conceptually shows the locations of the blanket drain section, toe/cut-off drain section and outlet drains in relation to the ring dyke.

The toe/cut-off drain section is situated immediately adjacent to the starter wall.

The main function of this drain is to minimize seepage through the starter wall and outer slimes walls during the initial establishment of the dam and to cut-off subsurface seepage in the upper aeolian sand horizons.

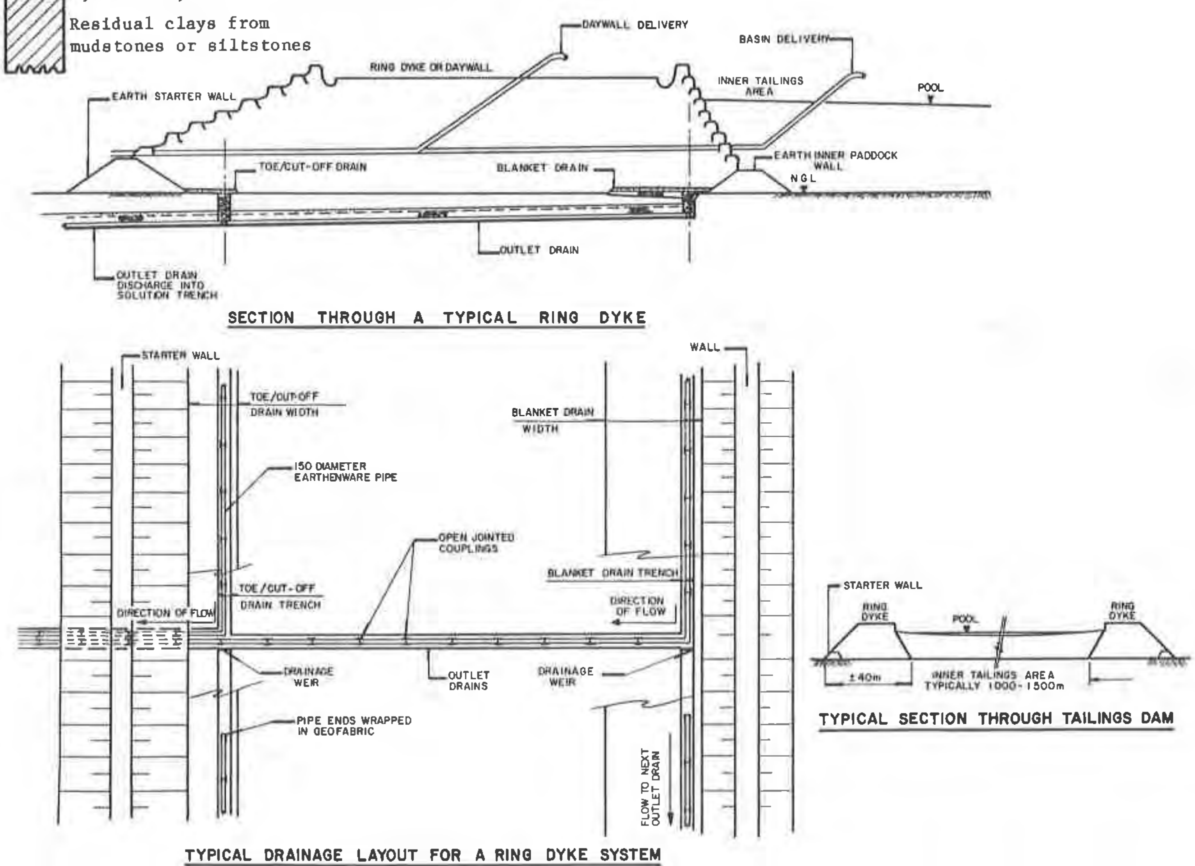


FIG 1

The toe drain thus controls the position of the phreatic line until the dam is high enough to permit the blanket drain to do so. Furthermore, during the entire life of the dam, the toe drain will be available as a back-up drainage system to the blanket drain should a section of the latter, at any stage, be rendered inoperational due to blinding or blockages.

The blanket drain section is situated well within the outer boundaries of the dam in order to control the position of the phreatic line after the development of a centrally located pool, thus minimizing seepage through the ring dyke and ensuring a stable perimeter wall.

The water flowing into the blanket and toe drains is transported out of the drainage system by means of a series of outlet drains interconnecting the blanket and toe drains. The frequency of outlet drain locations is determined according to the carrying capacity of the blanket and toe drains. The outlet drains discharge into the externally located solution trench which gravitates away from the tailings dam to a storage reservoir or sump for recycling back to the Mine.

3 DETAILED DESIGN AND ECONOMIC CONSIDERATIONS

3.1 The blanket drain

Fig 2 shows the typical cross-section adopted in the design of the blanket drain.

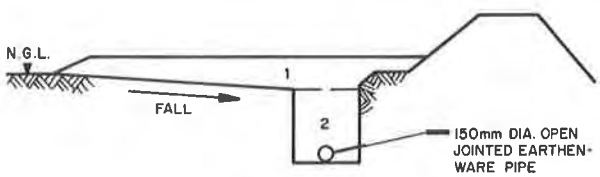


FIG 2

The drain has two components, viz:

- the blanket section for phreatic line drawdown, and
- the trench section which acts primarily as a means of transporting water to the outlet drain junctions, whilst simultaneously providing an additional area of attraction to subsurface seepage water.

In order to encourage flow, the natural ground below the blanket section is graded towards the trench section.

The open jointed drainage pipe is provided to increase the capacity of the drain where required, ie

- At drainage outlet junctions
- When, due to adverse topography, the frequency of outlet drains is insufficient to cater for the expected blanket drain flow.

Figs 3 and 4 show details of alternative drainage media considered, excluding the use of geofabric.

Figs 5 and 6 show details of alternative drainage media incorporating geofabric.

In determining the relative material and installation costs of the individual alternatives, the Authors have assumed that overall cross-sectional dimensions, viz width and thickness of blanket and depth of trench, are identical for each case. The relative implications of this assumption are noted in the text describing each alternative. The costs quoted in the text are based on current material supply and placing costs in the Orange Free State Goldfields.

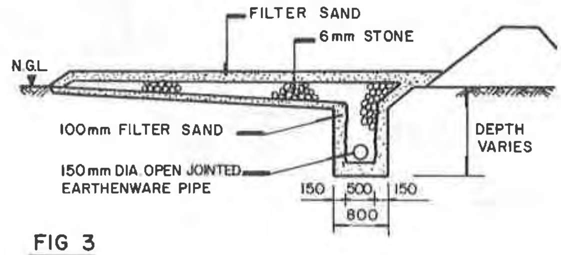


FIG 3

The drainage media in Fig 3 consists of filter sand and 6 mm stone. The filter sand is included to prevent migration of tailings and in situ soil into the 6 mm stone which is the main water transporting medium. The relatively low permeability of the 6 mm stone necessitates the installation of a larger quantity of trench pipes and/or a greater number of outlet drains than required by the other alternatives. The installation of this drain requires back shuttering and careful supervision in order to eliminate mixing of the different media. Installation costs are thus relatively high.

The estimated cost for supply and installation of materials is $45,00* per linear metre.

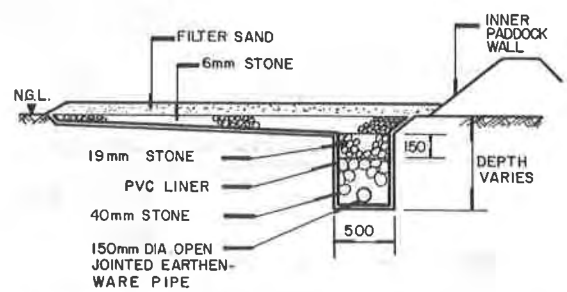


FIG 4

The drainage media in Fig 4 consists of filter sand, 6 mm, 19 mm and 40 mm stone and PVC liner. The filter sand prevents migration of the tailings into the stone, whereas the PVC liner is included to prevent migration of the in situ soil into the stone. The inclusion of the PVC liner, however, reduces the available area of attraction by creating an impervious layer adjacent to both the blanket and trench sections of the drain. The installation of this drain is relatively easy although careful attention and ground surface preparation is required prior to placing the PVC liner. The estimated cost for supply and installation of materials is $40,00 per linear metre.

* Costs are calculated in South African Rands and the U S Dollar equivalent quoted.

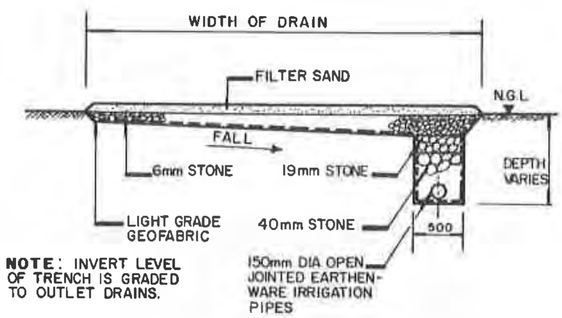


FIG 5

The drainage media in Fig 5 consists of filter sand, 6 mm, 19 mm and 40 mm stone, and geofabric. The filter sand again prevents migration of the tailings material into the stone, whereas the geofabric provides an effective filter between in situ soil and stone, preventing migration of the soil but allowing through flow of seepage water. Good supervision is required during installation to prevent mixing of the different media. Surface preparation of in situ soil adjacent to the geofabric is important to ensure the removal of loose material which may clog the geofabric and prevent through flow of seepage water. The estimated cost for supply and installation of materials is $30,00 per linear metre.

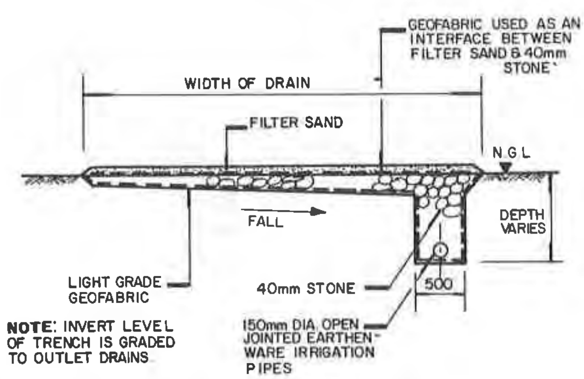
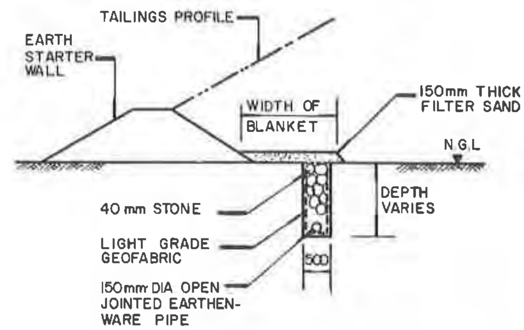


FIG 6

The drainage media in Fig 6 consists of filter sand, 40 mm stone and geofabric. The filter sand is required to exclude the possibility of fine tailings either clogging or migrating through the geofabric. The geofabric beneath the blanket section and lining the trench serves the same purpose as in Alternative 3. Stringent quality control is required during installation to ensure that proper lapping and stitching of the geofabric is carried out.

The relatively high permeability of the 40 mm stone allows for a lesser quantity of trench piping and/or a reduced number of outlet drains than required by the other alternatives. The estimated cost for supply and installation of materials is $35,00 per linear metre.

3.2 The toe/cut-off drain



The toe/cut-off drain has two components, viz:

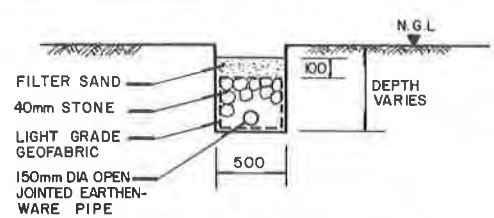
- The vertical trench section, which is lined with geofabric and backfilled with 40 mm stone. The geofabric is then stitched so as to fully enclose the stone. Apart from fulfilling its function as a cut-off to subsurface seepage water, the trench section serves as a means of transporting seepage water to the outlet drains. The open jointed drainage pipe is provided to increase the capacity of the drain as in the case of the blanket drain.

- The horizontal sand blanket serves two functions, viz:

  - to draw down the phreatic line in the ring dyke, particularly during initial stages of deposition.

  - to eliminate the possibility of tailings material either clogging or migrating through the geofabric at the 40 mm stone interface.

3.3 The outlet drain



The outlet drain consists of a trench section lined with geofabric, equipped with an open jointed pipe, and backfilled with 40 mm stone. A sand layer is placed above the geofabric to eliminate the possibility of the fine tailings material passing through the geofabric. This sand layer also protects the geofabric from ultra violet light exposure during the period prior to commissioning. The 40 mm stone has been included to ensure that the outlet drain is at no time rendered completely unoperational due to breakage and/or blockage of the pipe.

4 OBSERVED BEHAVIOUR OF UNDERDRAINS

4.1 Periodic visual monitoring of outlet drain flows from underdrainage systems which incorporate geofabric as a filter medium have been undertaken to estimate underdrainage efficiency for the tailings dams in question.

These observations indicate that underdrainage systems incorporating either blanket drain Alternatives 3 or 4 (Figs 5 and 6) perform equally well for a period of observation stretching over approximately 6 years.

4.2 Stand pipe piezometers have been installed in a number of tailings dams to determine the position of the phreatic surface with respect to underdrain location and outer wall geometry. Readings are taken monthly and results plotted on surveyed cross-sections at the piezometer locations. At each section, three piezometers are installed, one in the vicinity of the toe/cut-off drain location, one in the vicinity of the blanket drain location and one within the dam basin.

Fig 9 below represents the cross-sections at two piezometer set locations on the same tailings dam.

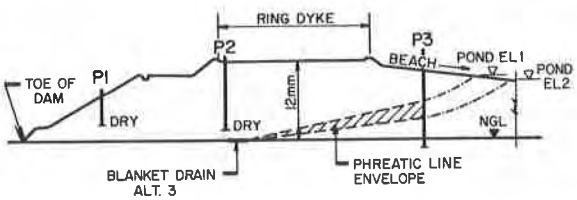


FIG 9a

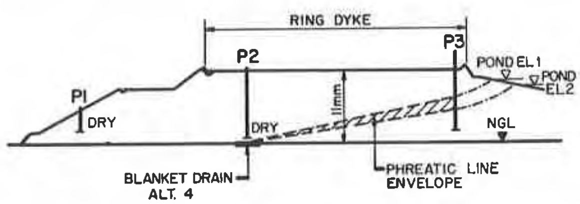


FIG 9b

When the underdrainage system for this tailings dam was designed, blanket drain Alternative 3 was specified, ie a blanket drain incorporating sand, 6 mm, 19 mm and 40 mm stone, with the underside of the blanket section and the base and sides of the trench section lined with geofabric (Ref Fig 9a). However, due to a severe shortage of 6 mm and 19 mm stone during construction, a section of this drain was constructed as on Alternative 4 blanket drain, ie a blanket and trench section of 40 mm stone completely enclosed with geofabric, overlain by filter sand (Refer Fig 9b).

In both cases, piezometers P1 and P2, located near the toe and blanket drain respectively, have been consistently dry for the past 12 months. Both P3 piezometers indicate an annual variation in phreatic surface of 2 m as illustrated by the plotted phreatic line envelope.

The above data have been interpreted to imply that the phreatic surface has been drawn down by both types of blanket drain with equal efficiency.

Where piezometers have been installed on tailings dams with either blanket drain Alternatives 3 or 4, similar results have been obtained.

5 CONSTRUCTION AND POST CONSTRUCTION PROBLEMS RELATED TO THE USE OF GEOFABRICS IN UNDERDRAINS

The Authors' involvement in the supervision of construction of underdrainage systems incorporating geofabric has led to the identification of several construction problems associated with the placing of the geofabric and post-constructional problems encountered during commissioning stages of the dam. These problems and suggested remedies are discussed briefly below.

5.1 Stitching or lapping of geofabric joints

The method of joining is of major importance when constructing toe, outlet and blanket drains and in particular at all junctions of these drain sections. Stitching, as opposed to lapping, is definitely the preferred alternative, being the more positive method of ensuring that there are no gaps or holes in the geofabric prior to and after placing of filter media. Several drain failures have been experienced during the drain covering operations with tailings material on drains which specified lapped joints.

5.2 Blanket bed and trench preparation

Trimming and compaction of the blanket drain bed and trimming of the trench base is necessary prior to placing the geofabric in order to minimise the quantity of loose in situ material at this interface. Light compaction of the stone within the trench is recommended to improve the uniformity of the stone/geofabric/in situ soil interface.

5.3 Weeds and vegetation

The germination and growth of weeds and other vegetation beneath the blanket section has resulted in both the puncturing of the geofabric and the loosening of in situ soil. To avoid this occurrence, treatment of the prepared blanket bed with weed killer is recommended.

5.4 Filter sand cover to geofabric drains

A filter sand cover to the geofabric surface of the drain is considered necessary for the following reasons:

- The time period between completion of construction of the underdrains and the total covering with tailings material can be substantial (typically 6 to 12 months). In order, therefore, to minimize unnecessary exposure to sunlight and consequent ultra violet attack on the geofabric, the sand covering is recommended.
- Transported in situ fine sand often contaminates the drain surface. Prior to covering the drains with the tailings material, it is essential that these contaminants are removed to ensure maximum drain efficiency. It is considerably easier for those involved in covering the drains with tailing slurry to skim off the contaminants from a layer of sand, using rakes, than to effectively remove them from a geofabric surface.

5.5 Rodents

Holes formed in geofabric lining to underdrains by rodents (rats and ground squirrels) have been identified in several cases.

This problem generally only arises during the construction period and the time prior to the deposition of tailings material. Rodents cannot survive after commissioning due to the resulting environmental changes.

The only practical method of locating these holes is, therefore, to undertake careful visual inspection of the sand covered drains immediately before covering with the tailings material. Rodent activity can be generally located by the detection of disturbed areas of the surface sand layer in the form of holes and mounds. Repairs should be undertaken by exposing and patching the damaged geofabric.

5.6 Recommended geofabric grades for various applications related to tailings dam construction

The following recommendations regarding the choice of geofabric grade for the various applications associated with tailings dam construction based on observations made during construction supervision, are:

- Underdrainage system to tailings dams as previously described - due to economic considerations the light grade (bursting strength approximately 2 000 kPa) is generally specified. This is normally acceptable provided the filter stone size does not exceed 40 mm diameter and stringent site control is continually exercised.

- Pollution cut-off drainage trenches - the depth of these trenches is often of the order of 3 m in order to effectively intercept the more permeable surface geological horizons. After lining the trench with geofabric, clean mine waste rock (approximately 150 mm average diameter) is commonly utilized as a backfill material in preference to the more expensive concrete aggregate stone. Hence, if rock is backfilled in trenches to depths greater than approximately 1 m and, in particular, if mine waste rock is used, it is recommended that medium to heavy grade geofabric is specified (bursting strength 2 500 to 3 000 kPa).

- Heavy grade geofabric (bursting strength 4 000 - 5 000 kPa) should always be specified when used as a filter layer below rock rip-rap wave protection to upstream faces of water reservoirs associated with tailings dams.

  This application is also applicable when rock toe buttresses are constructed to improve the stability of the outer slopes of tailings dams.

  In both these cases, mine waste rock having an average diameter of 150 mm is generally used.

6 CHEMICAL EFFECTS ON GEOFABRIC PERFORMANCE

In addition to the problems related to the use of geofabrics in underdrains, discussed in Section 5 of the paper, a potential exists for chemically-induced blockage and chemical attack of geofabric materials. These effects are related to the chemical composition of the liquid discharged from the tailings to the drains.

Typically, tailings liquids from gold tailings dams will be chemically saturated and carry high levels of total dissolved solids. However, the chemical composition of these liquids are subject to variation depending on the following:

- the level and extent of neutralisation of tailings after gold removal
- the pyrite (iron sulphide) content of the tailings
- the residual levels of pyrite in the tailings, when floatation removal of pyrite is practised

The general case for the most recent established gold mines in South Africa is that tailings are neutralised to pH7 or greater and pyrite removal is practised.

Iron levels in tailings solutions can be relatively high despite the above processes. It is observed that iron can be precipitated from gold tailings solutions in contact with air, by the oxidation of ferrous to ferric iron, the latter species having a considerably lower solubility product. If precipitation occurred in or on the geofabric, blinding of the fabric could occur.

In practice, however, no examples of such precipitation have been found on operational tailings dams. Exposure of fabric forming part of a toe drain showed no iron precipitation in the fabric initially. With the drain uncovered, a red iron precipitate began to be formed on the fabric with time, indicating that the potential for ferric iron formation existed. This would appear to support the proposition that tailings solutions from gold dams are chemically reduced systems whilst in the tailings dam itself, but that they become relatively oxidised after discharging from the underdrain into the solution trenches where iron precipitation can be observed.

In addition, as practice in South Africa has shown, geofabrics appear to be immune to chemical attack by liquids from gold tailings. It may be that, in the unneutralised tailings case, liquids with pH values less than $\pm$ 2, resultant from sulphuric acid leaching of gold ore, could degrade geofabric with time. This is the subject of further study.

CONCLUSIONS

In view of periodic difficulties experienced in obtaining sufficient quantities of natural filter media, in particular 6 mm and 19 mm stone, geofabric has been extensively used as an alternative filter medium in underdrainage systems of gold tailings dams in the Orange Free State Goldfields

The relative supply and installation costs indicate that the use of geofabric in underdrains is economically viable. The cost savings are primarily due to the simplification of drain construction techniques and consequent shorter construction periods.

Visual observations and piezometer readings indicate that gold tailings dams underdrainage systems incorporating geofabric operate efficiently in the medium term (6 years to date) and achieve their design functions in terms of accelerating the rate of consolidation of material within the ring dyke and ensuring the overall stability of the outer dyke walls by drawing down the phreatic surface within the tailings mass.

Most problems associated with the use of geofabric in underdrainage systems can be eliminated by ensuring adequate site control during construction and immediately prior to and during initial tailings deposition.

Observations to date suggest that the performance of geofabrics in underdrains is not adversely affected by the chemistry associated with gold tailings solutions.

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Use of Geotextiles in Dam Construction

Utilisation de géotextiles dans la construction des barrages

Seepage erosion and piping are responsible for a major part of embankment dam failures. Use of Geotextiles can prevent piping due to internal erosion, especially if natural material is not available in the quality and quantity desired and soil-cement diaphragm walls are used as sealing elements. The application possibilities for geotextiles as self-healing elements to avoid erosion through cracks are described. For the investigation of the properties of geotextiles, different laboratory tests are usual. In this paper, some test procedures and test results are reported.

In Bavaria, geotextiles were used as a self-healing filter for the "dry trench diaphragm walls" of two dams. Those are the 33 m high Förmitz dam in northern Bavaria and the 86 m high Frauenau dam in eastern Bavaria. Some details of the projects, especially for the diaphragm walls and of the measurement results of both dams are reported.

La formation de renards par érosion est responsable pour un grand part des ruptures des barrages en remblai. L'érosion interne peut etre évitée par géotextiles, especiallement en cas d'utilisation des parois-noyeux en sol-ciment comme éléments d'étanchéité. Les possibilités d'application des géotextiles pour éviter l'érosion au travers des fissures sont traitées. Les characreristiques des géotextiles sont étudées normalement en laboratoire. Ce rapport traite quelques essais et leurs resultats.

En Baviére, dans la construction de deux barrages en remblai, géotextiles sont etés utilisés pour la protection des parois-noyeux. Ce sont: la barrage de Förmitz en Baviére du nord, d'hauteur de 33m et la barrage de Frauenau, d'hauteur de 86 m. Quelques details des projects, especiallement des parois-noyeux et resultats de mésures sont rapportés.

INTRODUCTION

In constructing fill dams the required dam building materials are not always available in nature in the quantity and quality desired. Therefore, low-grade or artificially prepared material must sometimes be used. In recent times core sealing elements have been produced using the trench diaphragm method and including geotextiles. Geotextiles can contribute to the raising of the safety of an earth dam primarily against erosion, if it is warranted, that their qualities remain essentially unchanged for a sufficient period of time.
There are two options for the execution of the trenches and the subsequent introduction of the sealing material together with the geotextile. In the "wet method" the trench is filled and propped with a suspension followed by the sealing element material inserted mostly on the tremie method under displacement of the grout. The insertion of the geotextile follows a particular arrangement whereby it is pressed against the excavation wall. The problems of the installation joints are solved separately using this method. The "wet method" is able to be used after the dam has been partially or fully filled.

According to the so-called "dry method", an open continuos trench is formed in the dam core zone once the fill has reached a depth of one to three metres, the core fill being composed of more or less cohesive loose material.After the installation of a lapped geotextile on the downstream face of the trench, the trench is filled with soil-cement.

There are no problems of forming joints using this method.

Because of the small thickness of the wall, large hydraulic gradients occur. The geotextile has a particular function to fulfill concerning safety against erosion. In order to compare the various geotextiles and to check their performance for particular uses, this department carried out a series of tests. Tests were made on the mat weight, the water intake, the effective opening width, the permeability vertical to and parallel to the geotextile surface, the deformability, the sealing action and the friction angle of the geotextiles. The three-layer, sewn geotextile produced by the firm Naue, type 1002 NS from PES/PA with a mat weight of 1200 cm/m^2 proved to be particulary well suited because of its ability to accept large deformations without structural change and without change in the effectiveness of the filter.

Independent of the method chosen for the construction of trench diaphragms as a sealing element of a dam, the stiffness of the sealing element must largely match that of the adjacent dam material. The achieved compatibility of any deformations, which may result from stressing due to the building operations and later due to the water retention, leads to the avoidance of stress concentrations and associated possible rupturing of the sealing element. Nevertheless, tests with failure of the sealing were carried out. It is recommended that sealing walls have higher deformability and because of the associated low strenght, in this case the geotextile in the core zone is mainly employed as a safeguard against erosion. In other dam zones the geotextiles are used for drainage purposes or for the joining of fill layers. In Germany, geotextiles have been used in the construction of two large dams and the methods used are reported below.

E X A M P L E S O F U S E

Förmitz Dam.

For the construction of the Förmitz Dam, 33m high and approximately 700 m long, there was no suitable core material conveniently available; therefore, after through testing, the so-called dry trench diaphragm method was used for the production of the dam sealing element (1). In Fig. 1 is shown a dam cross section giving the arrangement of the geotextile. The sealing element was formed in 3 m high wall lamellas; in the central part, where the dam was higher, the element was connected to an inspection gallery, whereas at the flanks it was founded on and connected to a cut-off wall. Seepage water was collected in a concrete trough on the air side.

Fig. 2 shows how the pressure is reduced through the geotextile. The function of the geotextile was consequently two-fold. Firstly, it was used as a safeguard against erosion of the soil-cement so that fine particles could not migrate from the sealing element into the adjacent fill material and secondly, it was used to lead away incoming dam seepage water. The geotextile performed both functions well. The entire seepage water flow from the 240 m long middle section amounts to only 0.4 l/sec and has remained constant for years.

In the illustrated construction example it was observed that the soil-cement wall performed an important sealing function; thus strong claims should be made for soil-cement. These are: low permeability, sufficient flexibility, adequate strenght and safeguard against erosion. After

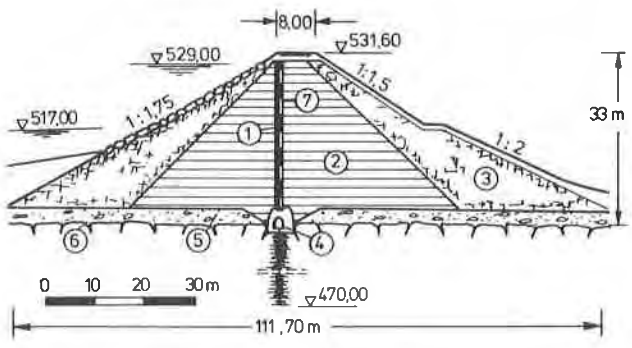

Fig. 1 Förmitz dam, cross section

1 Dry diaphragm wall
2 Semi-impervious material
3 Rockfill shoulders
4 Grouting and controll gallery
5 Alluvium
6 Bedrock
7 Geotextile

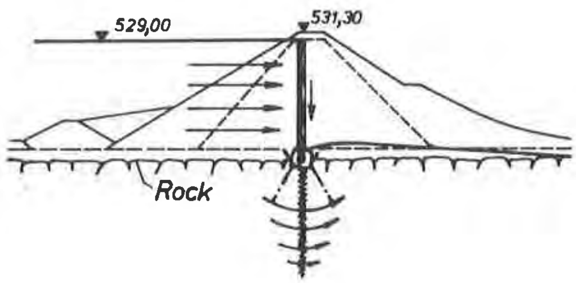

Fig. 2 Förmitz dam, phreatic line

extensive laboratory testing the optimal mix for the soil-cement was chosen as follows:

85% sand with approximately 20% silt content, 10% clay flour and about 5% cement, as well as a plasticiser.

The dry density of the soil-cement amounted to 1600 kg/m^3; for an addition of 400 l water the wet density was 2000 kg/m^3. The permeability coefficient obtained by the tests was 5 x 10^{-9}m/sec for a hydraulic gradient of i = 100. The triaxial cell was used to seek the stress and strain behaviour of the soil-cement. From this it was seen that axial strains of up to 4% could be accommodated without fracture or change of the k - value. In order to check that the deformations were free of fractures it was necessary to also carry out tests of the extendibility of the geotextile together with the soil-cement. From these tests it was shown that, after the production of artificially prepared fractures in soil-cement by the presence of a geotextile on the air side of the soil-cement a drop in the k - value to k = 1.10^{-5}m/sec soon occured which, however, for a small addition of silt quickly returned to a value of k = 10^{-8} or 10^{-9}m/sec, whereas without a geotextile no decrease in the k - value was observed. The erosion impedance or

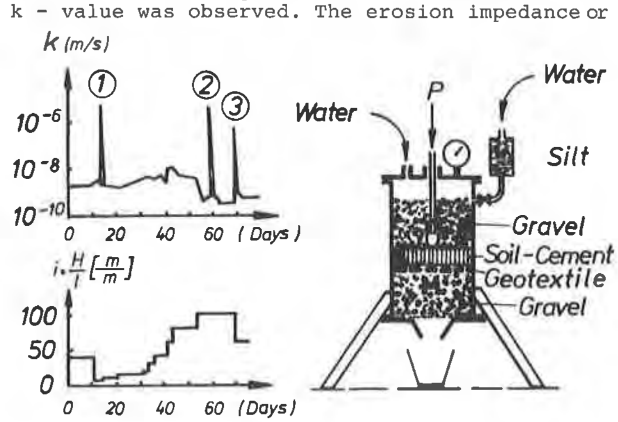

Fig. 3 Self-healing by geotextiles: test aparatus and test results ①, ②, ③ : artificially caused cracks in the soil-cement

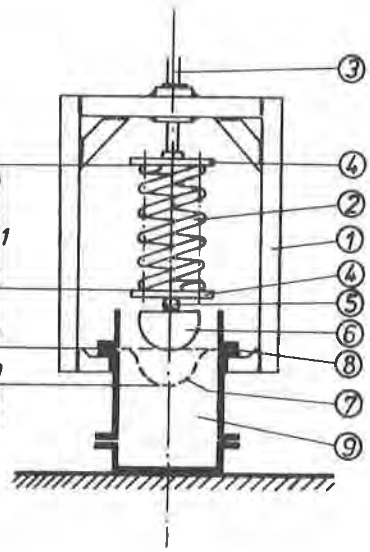

Fig. 4 Mechanical testing machine for geotextiles

1 Frame
2 Spring
3 Spindle
4 Steel Plates
5 Sphere
6 Hemisphere
7 Geotextile
8 Clamp-ring
9 Cylinder Ø 30 cm

self-healing effect of geotextile was seen, that by suitable strong water movements in front of the network of the geotextile a fresh supply of appropriate fine material was deposited to cause a sealing and self-healing effect. In Fig. 3 the testing arrangements and the test results are shown.

In Fig. 4 is shown the test arrangement for the determination of the deformability of the geotextile. According to this, this type of installation of geotextile within the sealing element as part of the construction serves to hinder the leaching and erosion processes with all their most grave manifestations. A further constructive and effective technical function applies, and that is to bind the diaphragm wall and the geotextile to the core foundation. In Fig. 5 is shown the connection of the sealing wall to the inspection gallery. Here, small movements should be accommodated without fracture. Therefore 2% bentonite was added here to the soil-cement. The effectiveness of the geotextile as a drainage course was tested by colour tests (in which colouring was introduced at the crown of the dam).

Frauenau Dam.

After the construction method had proved itself on the Förmitz dam, it was used also for the 86 m high Frauenau Dam. For this project there was available core material of adequate permeability, though it was inhomogeneous. The Frauenau Dam was formed as a rockfill dam with a central sealing core. Additionally in the core centre, as already mentioned, was built a trench diaphragm founded on the injection and inspection gallery. On both sides of the core there are filter or transition zones. The main body of the fill is composed of broken rock material. Details of the constructive shaping of the dam, particularly in the region of the thick permeable overlayer at the right hand side of the valley can be seen in Fig. 6. Here the trench diaphragm acts as an improved zone for the loam core sealing and provides, due to the inbuilt geotextile, additional safety for the structure against possible damage from occurences of leaching and erosion. For this dam, geotextiles were employed for three various functions:

a) Geotextile in the core as a safeguard against erosion following fracturing,
b) Geotextile as a sheet filter and separator between the weathered rock overlayers and rockfill shoulders,
c) Geotextile for the safety of the core excavation and as a filter.

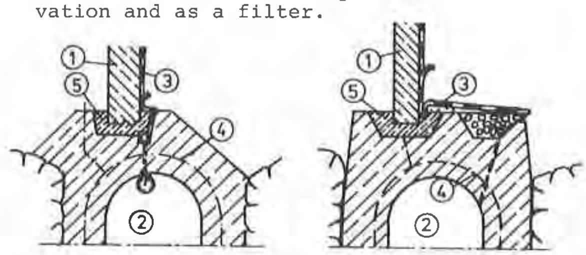

Fig. 5 Connection of the diaphragm wall at the controll gallery

A. Frauenau B. Förmitz
1. Soil-cement diaphragm wall
2. Grounq and controll gallery
3. Geotextile
4. Joint tape
5. Soft soil-cement

During ballasting of the geotextile in core centre, eight sections were created, joined so,that the water pressure and seepage water can be controlled in each section separately. Here were

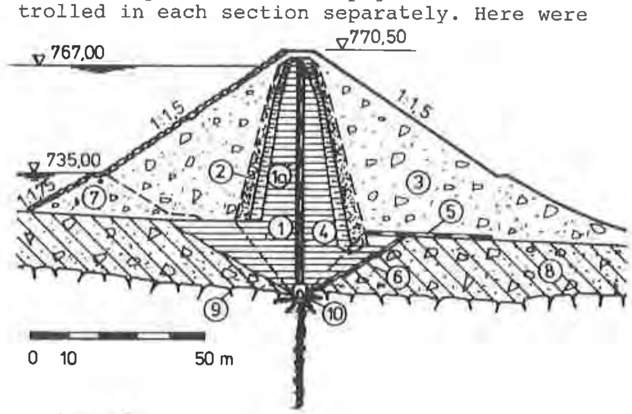

Fig. 6 Frauenau dam, major cross section

1 Core
1a Dry diaphragm wall
2 Transition zones
3 Rockfill shoulders
4 Geotextile for core
5 Geotextile for drainage
6 Geotextile for escavation slope
7 Cofferdam
8 Weathered rock
9 Bedrock, gneiss
10 Grouting and controll gallery

also extensive laboratory investigations carried out on the clay-cement which had been produced from weathered sand, clay flour and cement. In the triaxial test, rupture free deformations of up to 10% were achieved without changes in the nominal value of the k - value. The tests proceeded with shear stresses of $\sigma_1 - \sigma_3$ up to 18 bars. For the execution of the dry trench diaphragm the height of individual lamellas was limited to 1.2 to 1.5 m, which brought advantages to the construction. Moreover, this facilitated the perfect connection between the geotextiles and the wall elements. By means of control borings, the homogeneous condition of the trench diaphragm material and the connections of the wall elements were able to be verified.

In order to supervise the condition of the dam and the dry trench diaphragm both during the construction period and in later working operation, an extensive measurement and control system was installed. With the conclusion of the filling period in 1981 the dam was completed (2). From the many measurement results taken, only those connected with the trench diaphragm are treated in this paper. It should be mentioned here, that in 1981 a longer storage test was carried out with a reservoir water head of about 40 m. In Fig. 7 is shown the distribution of settlement of the dam height at cross section 2 at the end of the filling period. Noteworthy are the relatively small settlement differences between the trench diaphragm and the adjacent core material, amounting to approximately 10 cm. Since the settlements of the trench diaphragm are less than that of the natural core material, since it is stiffer than the core.

The development of the vertical soil stresses in the sealing core during the course of the filling procedure matches the rise of the overburden load γ. h:

$$\sigma_v = \gamma .\ h.$$

Because of the relatively small settlements in the dam core and fill body, no nominal stress redistribution occured. On the basis of the stress condition in the central part of the core and according to the measured deformations one can conjecture that the dry trench diaphragm was not to date, an object of disadvantageous changes. This is also confirmed by the results of the test filling of reservoir. According to this, one can conclude that the soil-cement wall is effective. This shows itself by the strong water pressure difference through the wall and by the very small seepage water discharge measured in the inspection gallery. This is as water tight as one could reasonably expect of a dam section lenght of 50 - 90 m, discharging 0.02 l/sec.

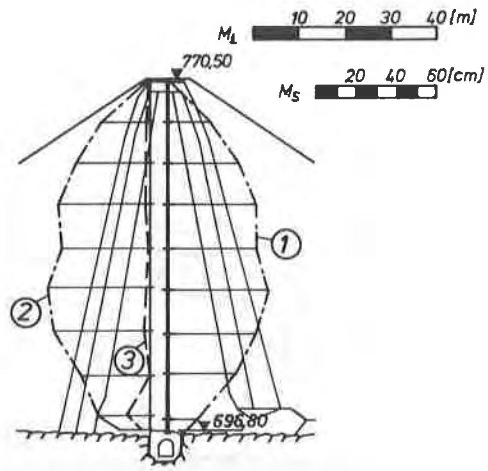

Fig. 7 Frauenau dam, Settlement distribution in diaphragm wall (1), core (2) and differential settlement (3)

CLOSING REMARKS

According to experiences made up to now, the application of the dry trench diaphragm method with soil-cement - provided that appropriate core material of adequate quality is not available - has proved to be an economic construction, as compared to the preparing and artificial upgrading of the core material. Use of this constructing method has shown good harmonization of the deformation condition between the wall material and the adjacent dam. Installing a geotextile on the downstream side of a dry trench diaphragm can be seen to be of particular advantage, because by this means is produced an additional protection against possible erosion.

Relevant literature:

(1) Lorenz, W. and List, F.: "Application of Dry Trench Diaphragm Method in Constructing the Impervious Cores of Dams consisting in part of Low-grade Fill Material", 12th Congress for Large Dams, Mexico 1976

(2) Beier, H. and List, F.: "Trench Diaphragms as Sealing Elements in Earth Dams." 14th Congress on Large Dams. Rio de Janeiro. 1982

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The Saint-Gervais Dam

Le barrage de Saint-Gervais

The hydro-electric and pumped-storage station of Montezic in the "Massif-Central" in France has required the construction of a 30 Mm3 upper reservoir. The normal utilisation is using 10 m (till 30 m) of weekly water level variation. In the point of view of the surrounding, the reservoir tail needs a particular care.
A dam (SAINT-GERVAIS) can keep a fixe reservoir, fed by a small river. The dam is bathed both side by the water and its construction made of granit materials more or less decomposed (in fact it is the only avalaible material) has found its solution in the geotextiles.
It is a 14 meters heigt dam above its foundation, its breaking is not critical and its use is both the water-tightness and the traffic. The construction has required the utilisation of 14.000 m2 of "bidim U44". In order to save from the possible erosion the very decomposed parts of granite, a particular care with the geotextiles utilisation has been given to all the places which could help the running away of the materials subject to the rapid variations of the water.

L'Aménagement de transfert d'énergie par pompage de Montezic dans le "Massif-Central" en France a nécessité la construction d'un réservoir supérieur de 30 Mm3. L'utilisation normale prévue utilise 10 m (à 30 m) de variation de plan d'eau à caractère hebdomadaire. La queue de retenue nécessite sur le plan de l'environnement un soin particulier.
Un barrage (SAINT GERVAIS) permet de laisser un plan d'eau fixe, alimenté par une petite rivière. Il est baigné des eaux des deux côtés et sa construction en granites plus ou moins décomposés (pratiquement le seul matériau disponible) a trouvé sa solution dans les géotextiles.
C'est un barrage de 14 mètres de hauteur sur sa fondation, sa rupture n'est pas critique et son usage est l'étanchéité ainsi que le passage routier. La construction a nécessité l'utilisation de 14.000 m2 de "bidim U44". Etant donné l'érosion possible des parties de granite très altéré, un soin particulier avec utilisation de géotextiles a eu lieu à tous les contacts pouvant favoriser la fuite des matériaux soumis aux variations rapides de l'eau.

INTRODUCTION

Les travaux faisant l'objet de cette communication entrent dans le cadre de l'Aménagement de MONTEZIC qui se situe en FRANCE dans le Nord du département de l'Aveyron (12) sur la rive gauche de la TRUYERE.

Il s'agit d'une usine de transfert d'énergie par pompage dans la retenue de COUESQUE (R.N. 365) et stockage dans une retenue supérieure créée par 2 barrages en terre de 60 et 35 mètres de hauteur avec une retenue normale (R.N.) calée à la cote 703 N.G.F. (Nivellement général de la FRANCE).

En queue de retenue, aux abords du village de St-GERVAIS, EDF a réalisé un plan d'eau fixe afin d'éviter les désagréments des queues de retenues variables en niveau. Pour ce faire un troisième barrage devenait nécessaire.

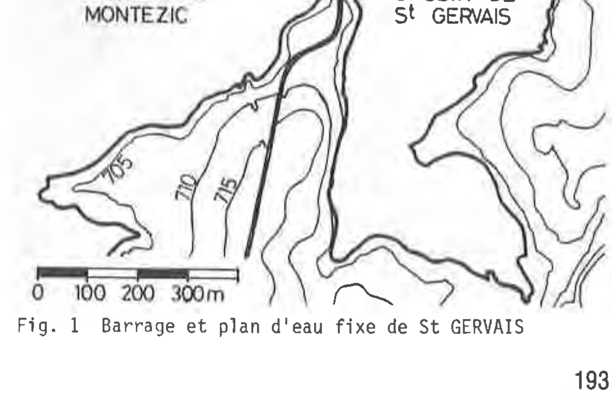

Fig. 1 Barrage et plan d'eau fixe de St GERVAIS

1 DESCRIPTION GENERALE DE L'OUVRAGE

1.1 Le barrage à 14 mètres de hauteur sur sa fouille la plus profonde (soit environ 10 m par rapport au terrain naturel T.N), sa longueur est de 140 mètres environ.

Le revêtement routier à mettre en place sur la crête est prolongé de part et d'autre du barrage sur une longueur totale de route à créer de 450 mètres où l'usage de géotextiles en couche anti-contaminante était recommandé en cas de fondation douteuse.

1.2 Une dérivation de la rivière est à mettre en place dans une fouille à l'air libre, il s'agit d'un tuyau en béton armé de 1 m de diamètre avec vanne de fermeture en position médiane. Les contacts avec le remblai sont assurés par géotextiles.

1.3 La chaussée créée sur la crête en revêtement routier sur géotextile est traitée pour être submersible.

1.4 Un ensemble de 3 ouvrages mixtes (béton pour les têtes dans les enrochements et buses métalliques dans les remblais souple) permet l'écoulement des crues, le maintien du plan d'eau à une cote peu variable (avec possibilité de nettoyage des seuils par remplissage aval, leur calage étant réalisé à 0,20 m au-dessous de la R.N.).

1.5 La masse de l'ouvrage est en granit décomposé et la protection de surface est en enrochements posés sur géotextiles jouant un rôle décontaminant et créateur de drains.

d'énergie oblige à un marnage journalier, ce qui, pour un granit décomposé en "gore" rend obligatoire la tenue du matériau par un filtre d'où utilisation d'un géotextile.

2.3 Un talus de "gore G0 + G1" (cf. Ref. (2)) a donc été revêtu d'un non tissé durant la mise en place, par couche de 1 m d'épaisseur d'une recharge en enrochements sur 6 m de hauteur. Le gore était à 1/1 de pente et les enrochements de surface à 3/2 avec 6 m d'épaisseur à la base.

2.4 Un arrosage abondant dans des enclos de sacs de sable a permis de mesurer le délavage des enrochements.

2.5 Après essai, des tranchées ont permis de voir les matériaux et le contact. Ce contact en géotextile non tissé d'épaisseur moyenne (ici un Bidim U44) a bien tenu la pénétration des blocs sans déchirure.

Des empreintes de 30 à 40 cm très enveloppantes attestaient d'un compactage puissant, au vibrant lourd (12 t) qui a fait pénétrer les blocs dans le gore le plus fin situé derrière le non tissé.

Il était donc possible de placer ou plutôt de déverser des enrochements directement sur le géotextile sans trop de précautions. Evidemment le travail doit être fait le plus délicatement possible et on doit interdire à la lame du bull de s'approcher du non tissé.

2.6 Sous le Bidim le gore est resté très homogène et a obtenu un serrage très satisfaisant. Malgré l'arrosage puissant décrit en § 2.4 la stabilité de cette sous couche de gore est restée parfaite.

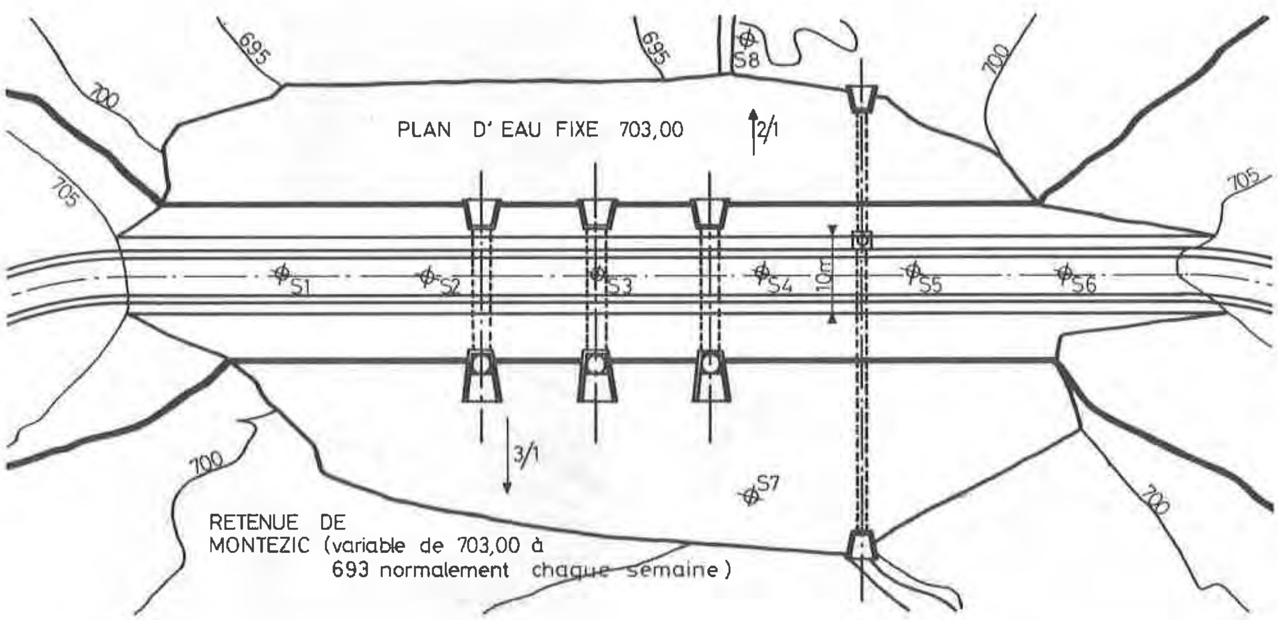

Fig. 2 Vue en plan du barrage

2 ESSAI PREALABLE

2.1 Le seul matériau disponible étant le granit il fallait donc l'utiliser très décomposé ou en cours de décomposition pour le noyau étanche, puis rechercher les blocs "non évolutifs" (soit le granit ayant plus de 2,6 de densité sèche) pour servir de protection de surface. Les volumes étant faibles un géotextile en interface devait être capable d'éviter des zones de transition.

2.2 La gestion du réservoir de MONTEZIC en transfert

3 PRINCIPE DES FOUILLES

3.1 Un ensemble de 7 forages destructifs a été réalisé avec enregistrement des paramètres de foration (Cf. Ref. (1)).

3.2 Sur la fig. 3 les coupes des sondages indiquent d'après la vitesse d'avancement complétée par les pressions d'injection d'eau, que l'on peut percevoir le fond rocheux dès 3 à 4 m de profondeur et ceci dès que la vitesse d'avancement est inférieure à 50 m/heure.

3.3 Le principe des fouilles retenu est donc de créer le contact du noyau sur le rocher en place (de qualité moyenne mais suffisamment étanche et homogène confère les enregistrements des paramètres de foration. Voir fig. 3).

Une fouille de 10 mètres de largeur (6 aux extrémités) avec des talus à 2/1 permet ce contact et ce d'une façon pratiquement théorique, avec avant métré possible.

3.4 Hors la zone citée en § 3.3 le décapage est limité à 0,50 m pour enlever les terres végétales ou redresser le terrain. En zone amont quelques poches tourbeuses ont nécessité un ancrage complémentaire avec remblais sur géotextile limitant la souplesse lors du compactage.

3.5 Enfin le tapis d'enrochements avec un surcreusement de 2 m est posé sur un géotextile non tissé servant d'anti-contaminant.

En amont la présence d'eau dans cette fouille a obligé la pose du géotextile sur sol fraichement décapé suivi rapidement par la pose des encochements. Ceux-ci ne seront compactés que lorsqu'ils seront hors d'eau (il s'agit là d'une solution économique, sans pompages mais seulement faisable si le sol de fondation n'est pas trop souple pour la sécurité de pose).

à des blocs décomposables dès qu'ils sont extraits donc privés de leurs contraintes tri-dimensionnelles naturelles (d allant de 2,3 à 2,5 ou même près de 2,6). Un tel matériau mélange se compacte très bien par couches épaisses (1 m à 1,20 m) pour donner des densités sèches supérieures à 2,10, souvent 2,20 à 2,25, alors que le gore seul ne pourrait pas se serrer à plus de 1,7. Une fois en place ce matériau se délite intérieurement pour créer une masse homogène et très étanche, partant d'un matériau très courant et économique de mise en oeuvre (cf.Réf. (2))

4.1.2 Hélas un tel matériau est sensible à l'érosion externe certainement, et interne s'il y a un défaut d'homogénéité tel l'amoncellement local de blocs parmi les moins évolutifs.

4.1.3 En solution économique et sûre (cf essai § 2) l'enrobage de ce matériau par un Géotextile permet à la fois d'éviter toute migration de fines, tout en favorisant les contacts et le compactage simultané de divers matériaux en raison de la souplesse et de la grande déformabilité d'un non tissé moyennement épais.

4.1.4 Sur le contact de la fondation (rocher) ou des appuis latéraux (matériaux à base de gore), le Géotextile est réputé inutile en raison de la similitude des maté-

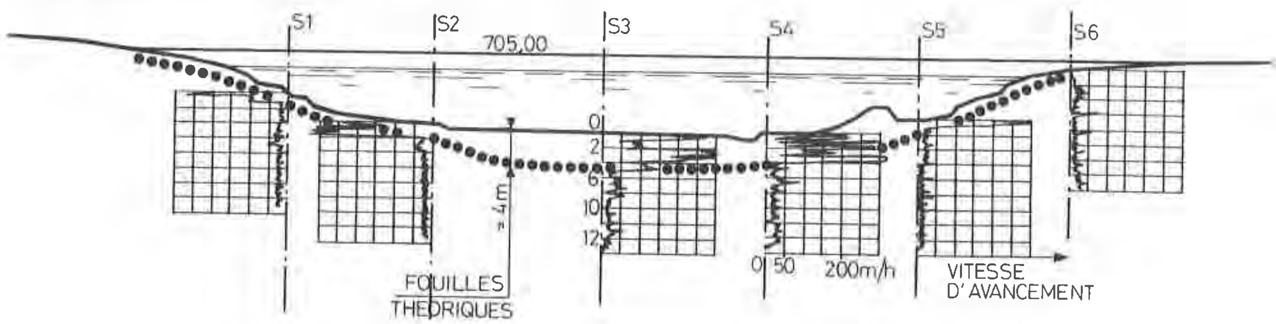

Fig. 3 Profil en travers de la vallée, Sondages avec enregistrements de paramètres

4 EXECUTION DU BARRAGE

4.1 Confection du noyau

4.1.1 La confection de la partie centrale, c'est à dire du noyau, a eu lieu à sec par compactage d'une façon tout à fait traditionnelle avec un matériau tout venant de granite en cours de décomposition allant du gore (d = 1,7)

riaux en contact, sauf en zones souples citées en § 3.4.

4.1.5 Dans la mesure du possible toutes les fouilles sont soigneusement compactées avant mise en remblais, sauf en terrain médiocre ou trop humide où le compactage se fait après pose du "Bidim" et d'une première couche de remblais.

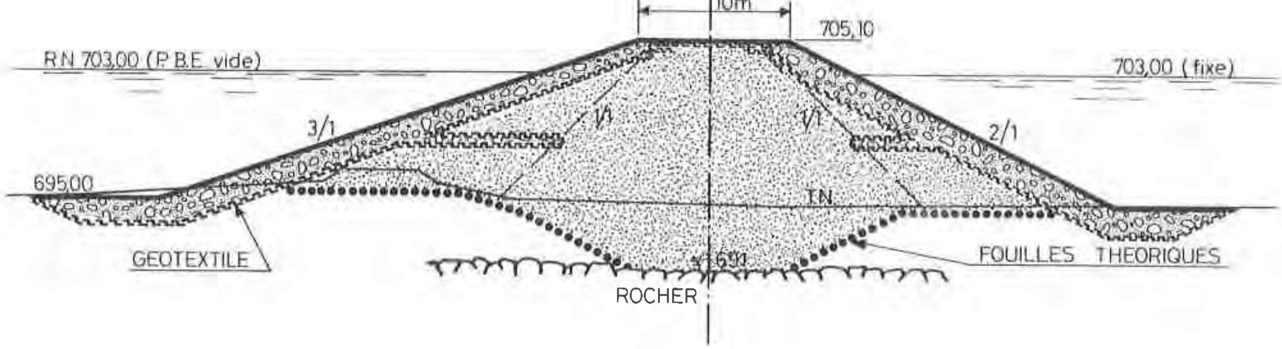

Fig. 4 Profil type du Barrage de St GERVAIS

4.2 Mise en place des enrochements :

4.2.1 Ces enrochements correspondent à du tout venant de carrière en granite classé non évolutif (densité supérieure à 2,6 ou mieux 2,7) et de granulométrie allant de 0 à 800 mm.

4.2.2 Conformément aux indications données avec les fouilles § 3.5, le pied des enrochements a d'abord été mis en place afin de sortir de l'eau et assainir le chantier. Donc pose sur Géotextile non tissé puis compactage.

4.2.3 Le "Bidim" pouvait alors être mis en place conformément à l'essai préalable décrit en § 2 pour réaliser la pose des enrochements sous relative faible épaisseur.

Le talus du noyau était dressé, après que le corps soit compacté, et avant de recevoir le géotextile avec une dénivellée maximum de l'ordre de 3 mètres. Le chantier plaçait une protection de matériau 0-300 mm de 0,50 m d'épaisseur à la pelle sur le "Bidim" avant la mise en place des enrochements 0-800 mm. Un petit bull réalisait le réglage puis le compactage vibrant avait lieu chaque 1,50 m.

4.2.4 Un drainage intermédiaire paraissant souhaitable pour assurer la sécurité du noyau à la vidange rapide, le Géotextile a été utilisé pour éviter la contamination d'une couche drainante de 0,50 m d'épaisseur en matériau 10-100 mm de capacité drainante 10^4 à 10^6 plus grande que le gore (en fait il s'agit plus de casser les pressions dans le massif de gore saturé que de drainer un débit trés faible).

4.3 Crête de l'ouvrage :

4.3.1 Sur une largeur totale de crête de 10 m, la partie centrale de 7 m est en base de noyau de gore enrobé de "Bidim". Dans les angles, la couche de non tissé a été doublée afin de mieux contenir les efforts latéraux des roues de véhicules.

4.3.2 Le revêtement routier était prévu directement sur le "Bidim", en fait une couche de base de 0,15 m a été placée et compactée puis a reçu une semi pénétration de bitume avant la couche de finition en bi-couche.

4.3.3 En fait ce seuil est prévu submersible et sa cote de calage est à 705,10 (10 cm pour contenir les tassements et + 2 m par rapport à la retenue normale). Là encore le Géotextile est capable de garantir une telle submersibilité étendue sur tout le seuil donc à faible charge d'eau.

4.4 Incorporation de la Dérivation Provisoire :

4.4.1 Cet ouvrage en béton armé sous forme d'une buse ronde de 1 mètre de diamètre avec vanne en position centrale, donc avec une amenée d'eau à la cote maximum au centre du barrage lorsque celui-ci est vide sur l'aval. Il fallait donc soigner le contact terre-béton et poser la conduite soit sur le rocher soit sur des plots béton.

4.4.2 Chaque joint entre buses bien qu'étanche avec joint incorporé devait recevoir à l'extérieur au contact des terres une bande de Géotextiles évitant tout risque de délavage du matériau du noyau (voir Fig. 5).

TETE AVAL

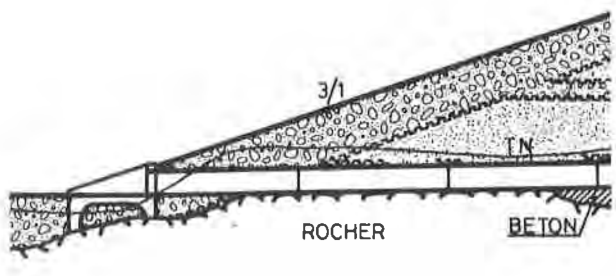

Fig. 5 Pose de la Dérivation provisoire - Vidange

4.4.3 Enfin à l'extrémité du noyau un Géotextile se retournant sur le tuyau devait pouvoir contenir, comme dans un sac, le matériau du noyau évitant là encore l'érosion de cette zone de contact entre un béton coffré et les terres du noyau, d'où une forte discontinuité de granulométrie surtout dans la partie inférieure du tuyau rond.

4.5 Incorporation des trois déversoirs :

4.5.1 La souplesse du remblai a conduit à préférer des ouvrages traversants souples donc un choix de buses métalliques genre "Armco" à la place d'ouvrages béton trop rigides. Le débit centenaire de crue étant de l'ordre de 16 m^3/s, le choix d'écoulement s'est porté sur 3 buses arches de 2,50 m de portée et 1,75 m de hauteur. La forme moins courbe du radier étant favorable à l'appui, celui-ci étant pré-formé et compacté.

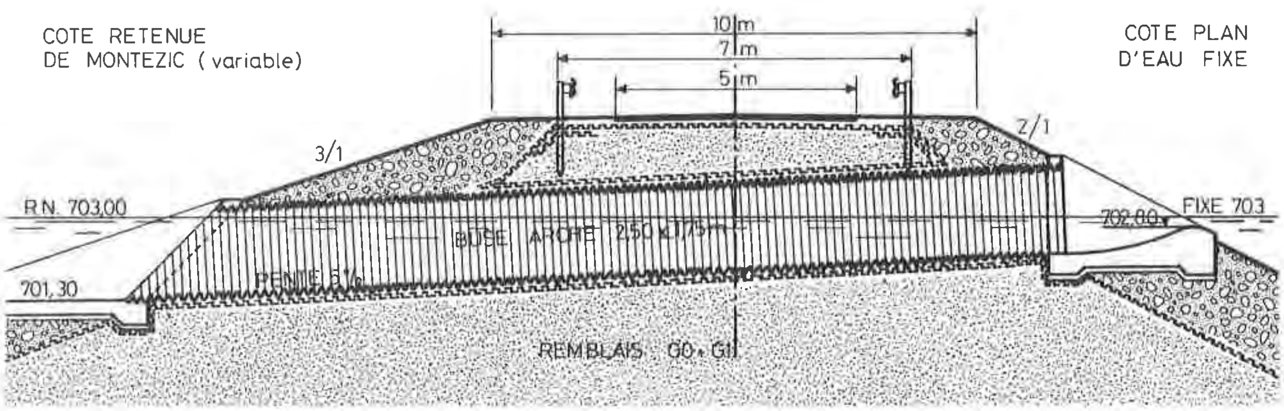

Fig. 6 Mise en place des déversoirs et détails du couronnement

La cote du seuil amont, calé à 20 cm sous le niveau normal, permet lors du remplissage aval un très léger retour d'eau permettant le décolmatage des dépots flottants, du reste répartis par l'utilisation de trois seuils indépendants.

4.5.2 Le contact du gore avec les ondes métalliques est favorable, mais les nombreux joints, prévus étanches par incorporation d'un produit de joint au montage, ont été réalisés en fait fer sur fer pour mieux contrôler le serrage. Après peinture bitumineuse l'ensemble des buses à reçu coté terre une bande de Géotextile non tissé évitant là encore tout délavage du gore soumis dans ces parties hautes de l'ouvrage à une alternance "air-eau" théoriquement néfaste à leur stabilité sans protection.

4.5.3 Là encore comme pour la D.P. (cf. § 4.4.3) le matériau du noyau est comme mis en sac à l'extrémité du contact de chaque côté.

4.5.4 Les bétons des parties d'ouvrages d'extrémité posés sur le matériau du noyaû sont coulés sur un Géotextile afin de favoriser le contact, en évitant un délavage capable d'affaiblir la fondation (là encore un non tissé est préférable).

5 APRES LA MISE EN EAU DU BARRAGE

5.1 La mise en eau s'est faite sans problème en formant la vanne de vidange de fond (en fait utilisée en Dérivation Provisoire cf. § 4.4).

5.2 Les seuils fonctionnent normalement mais sur l'aval, lorsque la retenue de MONTEZIC est vide, les enrochements ne sont pas suffisants, peut être en qualité, à une pente moyenne adoucie à 3/1 sur l'aval contre 2/1 à l'amont. Il a fallu renforcer, dans les mois qui ont suivi, la stabilité de ces blocs. Un enrobé de brai-vinyl disponible avec l'exécution des barrages principaux, a été mis en place au seau afin de limiter les mouvements des blocs.

5.3 A priori les géotextiles utilisés se comportent bien et disons qu'à part le problème cité ci-dessus en § 5.2, la construction, puis la mise en eau, enfin l'insertion dans le paysage, sont passés si inaperçus que ce barrage est resté relativement inconnu même des proches d'E.D.F.

Un dernier facteur intéressant a été le règlement définitif au forfait sur une base d'appel d'offres (pratique peu courante sur les barrages à E.D.F.) à un coût moyen relativement peu élevé pour un ouvrage déjà important (140 m de long et 10 m de hauteur sur le T.N. soit 14 m sur sa fondation).

6 CONCLUSIONS

6.1 Sur le plan technique, la présence d'un seul matériau de remblai : le granit plus ou moins décomposé ou décomposable, rendait sans le Géotextile la durabilité de l'ouvrage, soumis à un tel marnage journalier avec de l'eau des deux côtés, assez délicate.

Il aurait fallu utiliser des zones de filtres et transitions et réaliser certainement des fondations nettement plus importantes.

6.2 L'utilisation de plus de 14 000 m^2 de Géotextiles non tissés n'a causé qu'une plus-value de 5 % sur l'ensemble traité au forfait y compris les installations de chantier et les ouvrages divers.

Cette plus-value passe à 9 % si l'on ne regarde que les seuls terrassements (il s'agit de l'incidence du Géotextile en place soit environ un prix double de celui du seul achat).

6.3 Une des retombées principales et déjà citée en § 5.3 ci-dessus se retrouve dans un travail au forfait où les aléas sont faibles pour une qualité de travail prévue. Les fouilles, contrairement aux habitudes ont pu être traitées au forfait, bien que le marché prévoyait un réajustement si le montant issu des quantités effectivement réalisées différaient du projet en plus ou en moins de seulement 5 %. Il n'a pas été fait usage de cet article (tout au plus une somme de travaux complémentaires a atteint 4 % du forfait).

6.4 Pour des questions de crues ou de temps cet ouvrage a été prévu et réalisé en 6 mois de Mai à Octobre 1980, la définition à priori des fouilles donc des volumes à mettre en oeuvre y était pour beaucoup.

6.5 Il s'agit d'ouvrages capables d'être réalisés presque n'importe où, en particulier dans les pays en voie de développement et disposant de peu de matériels, en gardant l'assurance d'une bonne pérrénité. En effet si certains Géotextiles sont sensibles aux ultra-violets, ici tous les textiles sont enterrés et là, la pérrénité est considérée comme excellente. Quant aux risques de colmatage ils ne sont pas de mise ici puisque surtout l'effet amont est recherché.

6.6 Dans la gamme des tissus s'offrant maintenant à l'Ingénieur il semble que, dans l'ouvrage que l'on vient de décrire, de part sa souplesse extrême et ses qualités de transition au contact des bétons ou des blocs, le non tissé semblait mieux adapté que le tissé.

7. REMERCIEMENTS

Mes remerciements iront d'abord à M. BLANCHARD mon Chef de Service qui a bien voulu accepter mon projet qui, avouons-le, sort des habitudes des grands travaux hydrauliques où nous recherchons la sécurité avant toute chose. Bien entendu tout le monde, enfin tous ceux qui ont compris l'usage raisonné de ces Géotextiles, savent que ces produits bien utilisés sont sûrs. Mais il fallait accepter et ensuite réaliser l'objet par l'Entreprise avec le contrôle d'un Aménagement dont le Chef était M. ASTRUC. L'Entreprise était "CHANTIERS MODERNES" dirigée sur place par M. CASTELLETTO.

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Experiences in the Use of Geofabrics in Underdrainage of Residue Deposits

Experimentation dans l'usage de géotextiles pour le drainage sous les residus

Underdrainage of a residue deposit helps the stability of the perimeter slopes by drawing down the seepage surface, preventing its emergence and reducing pore water pressures. Filter layers separate the deposited material and the underlying soil from the drainage medium while passing the seepage water. In South Africa, geofabrics have been used in filter drains under the walls of several gold slimes dams. At the ERGO dam, constructed by hydrocyclone separation, details of the filter drains have been improved with experience of their performance. Conditions where the filters can become clogged by iron precipitates should be avoided. At Crown Mines, new deposits are placed by the paddock method on top of existing slimes dams with inadequate underdrainage. New toe drains, incorporating geotextiles and open plastic mesh as separator-filters and to discharge seepage water, have been detailed with a view to fast, safe, easy installation. The use of geofabrics in residue underdrainage applications must still be proved by satisfactory long term performance.

Le drainage des résidus stabilise les pentes périmétriques en abaissant la surface d'infiltration, en empêchant son émergence et en réduisant la pression interstitielle de l'eau. Des couches filtrantes séparent le matériau des drains des résidus et du sol. En Afrique du Sud, des géotextiles ont été utilisés dans des drains-filtres sous les murs de plusieurs barrages pour les boues des mines d'or. Au barrage d'ERGO - système cyclone -, des details des drains-filtres ont été améliorés en fonction de l'expérience acquise. Il convient d'éviter le blocage des filtres par des précipités ferrugineux. Aux Crown Mines, des dépots sont ajoutés à des barrages anciens dont le drainage est insuffisant. Des drains nouveaux aux pieds des pentes comprennent des géotextiles et des treillis en plastique comme filtres-séparateurs et l'écoulement d'infiltration. Leur conception permet une installation rapide, simple et sûre. La performance à long terme des géotextiles dans ces applications reste cependant à confirmer.

INTRODUCTION

Considerable quantities of waste residues are produced when valuable constituents are extracted from mineral ores. The waste products, often in slurry form, are commonly deposited in tailings or slimes dams.

The fine wet residue deposits are retained behind confining walls which are often constructed from the waste material itself. It is obviously essential to maintain the stability of the whole perimeter of each dam.

A major contribution to the slope stability can be obtained by drawing down the seepage surface by the provision of underdrainage. In this way the pore water pressures in the deposit are reduced and the material strength correspondingly increased. In addition seepage is prevented from emerging on the slopes.

It is important to ensure that the drainage system will always operate efficiently, both during construction of the residue deposit over a period of ten years or more and subsequent to its completion. For this reason, filter layers are provided to separate the deposited material and the underlying soil from the drainage medium while at the same time permitting the seepage water to pass through.

In South Africa, geofabrics have been used in the construction of filter drains under the walls of a number of slimes dams. Experiences with these applications at several gold slimes dams on the Witwatersrand are described below.

ERGO TAILINGS DAM

Project Description

The East Rand Gold & Uranium (ERGO) project has been established to recover mineral-rich material from a number of old residue deposits slimes dams in the Springs area and to extract gold, uranium and sulphuric acid. The resultant residues are disposed of in a new slimes dam sited across a shallow valley some 10 km away from the process plant.

The walls of the dam are constructed of cycloned coarse slimes within earthfill toe walls. The cyclone underflow produces a free-draining, free-standing, high strength outer wall within which the fine overflow material is deposited. The dam will eventually reach a maximum height of 64 m above the valley floor.

The dam, one of the largest in the southern hemisphere, covers an area of 790 ha and, within its perimeter of 11 700 m, 360 million tonnes of slimes will be deposited at the rate of 1,5 million tonnes per month.

Filter drains

To ensure that the outer impounding walls are properly drained, filter drains have been installed prior to the start of deposition. Longitudinal drains are provided under the centre of the coarse slimes wall and close to the toe wall and, in the valley sections where the wall will be high and wide, intermediate between the two. Lateral outlet drains are provided at intervals from 30 to 200 m depending on the expected flow volume.

The drains have been laid in stages as the wall extends further up the valley flanks. As indicated in Fig. 1, the construction details of the original installation in 1977 have been modified for the 1979 extensions as a result of observations of the drain performance. For example, it has been found beneficial to use 19 mm rather than 13 mm filter stone and to introduce a 150 mm diameter perforated pipe into the centre of the drains.

The provision of more than two layers of natural filter aggregates would have been unduly laborious and, furthermore, natural gravels for intermediate filters were not readily obtainable. Hence, geofabrics were used to wrap around the coarse filter stone and separate it from the surrounding natural ground or filter sand. This system provied to be effective and simple to lay.

On the South African Highveld 1 600 m above sea level, the sunlight is strong and it is essential to use ultra-violet resistant materials. Otherwise the geofabric is likely to become brittle and disintegrate before it is buried under the tailings cover.

Problems

Several years after the drains had been laid, it was noticed that at certain places the drains were not flowing satisfactorily.

In one area where coarse slimes deposition had started only recently, the drain became exposed by flowing stormwater (Photos 1 and 2). In places, the fabric was torn by sharp aggregate particles carried by the water, but otherwise it was not damaged physically at this stage. However, where it had been exposed to air for any length of time, as illustrated in the lower part of Photo 2, it was discoloured and impermeable to water. Freshly exposed sections, such as in the upper part of Photo 2; were still permeable. Examination of the fabric showed that it was clogged with fine slimes particles held in place by an iron oxide precipitate.

As the deterioration of the geofabric was restricted to areas where it had been exposed to the atmosphere, it was concluded that as long as the drains remained covered there was no cause for concern. An attempt was made to reproduce the phenomenon in the laboratory using the same materials (slimes, geofabric, water collected from the filter drain) as on site and exposed to air and sunlight. The test arrangement is shown in Photo 3. However, the clogging observed in the field could not be reproduced in the laboratory in five months of experiment.

A few months after the first clogging observations, it was noticed that certain of the lateral filter drains in areas close to fairly high sections of the coarse wall appeared to have insufficient drainage capacity. When one of these drains was opened up for inspection, there was little apparent clogging due to slimes. However, a noticeable ochre-brown deposit was observed on the geofabric and on the enclosed stone aggregate.

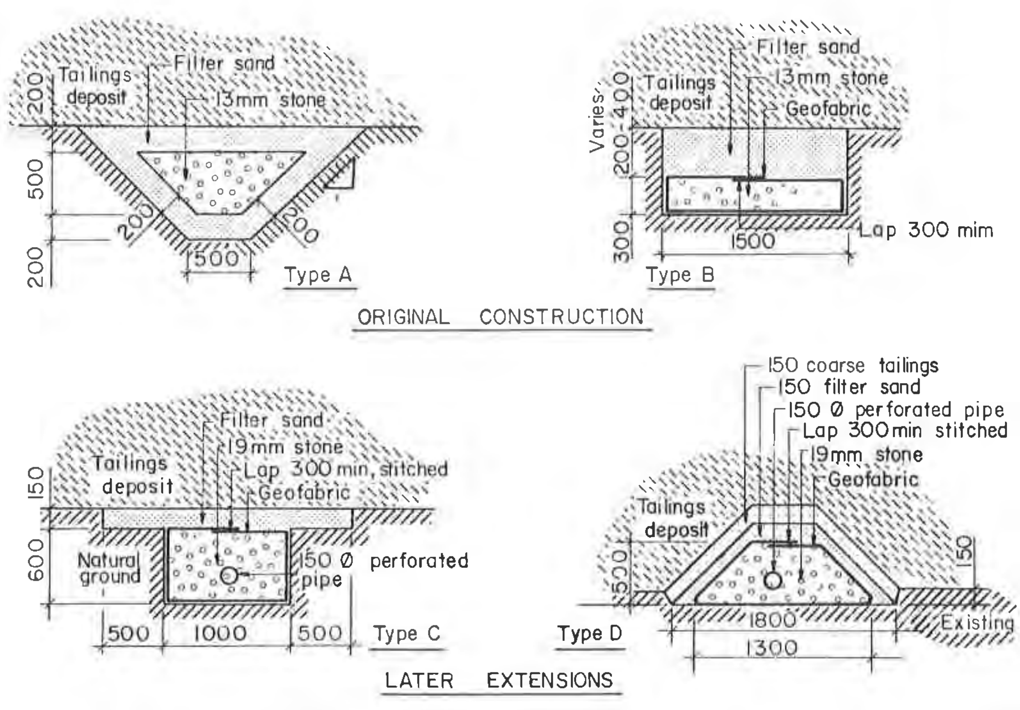

Fig. 1 : ERGO Tailings dam - Typical filter drain details

Photo 1 : ERGO Tailings dam - Exposed filter drain, no physical damage to fabric.

Photo 2 : Close-up of photo 1 - Top recently exposed and still pervious, bottom discoloured and impervious.

The observed clogging of the geofabric and the filter material has been explained as due to the precipitation of ferric hydroxides, such as goethite, on oxidation of ferrous compounds in solution at pH less than about 5. Usually the pH of the water reaching the slimes dam is above 6 and any insoluble iron compounds have been precipitated out at a much earlier stage in the process. However, when certain items of the plant are out of commission and the pH is not properly controlled, it is noticeable how quickly the ferric precipitate can build up in the filter drains at locations where oxidation can take place. This is a situation to beware of.

CROWN SANDS SLIMES DISPOSAL

Project Description

Old gold-containing sand dumps and slimes dams in the Crown Mines area of Johannesburg are being retreated in a recently completed plant. As there are no areas available in the vicinity of the city for the establishment of new residue deposits, the wastes produced by the new plant are to be placed on top of existing slimes dams which have been dormant for some years.

The new deposits will eventually add a maximum of 33 m height to the dams which are already up to 31 m above original ground level. The material is being placed by the paddock method traditional to the gold mines in South Africa.

In this method the slimes dam wall is built up in stages in paddocks 20 to 30 m wide and up to 200 or 300 m long, as follows:

1) A layer of slurry is deposited in the paddock.
2) This deposit is allowed to dry out for a few days while the slurry is directed to other paddocks.
3) As soon as the material has developed some strength, ridges 10 to 15 cm high and 30 to 50 cm wide are packed up by hand along the edges of the paddock.
4) Another layer of slurry is directed into the paddock.

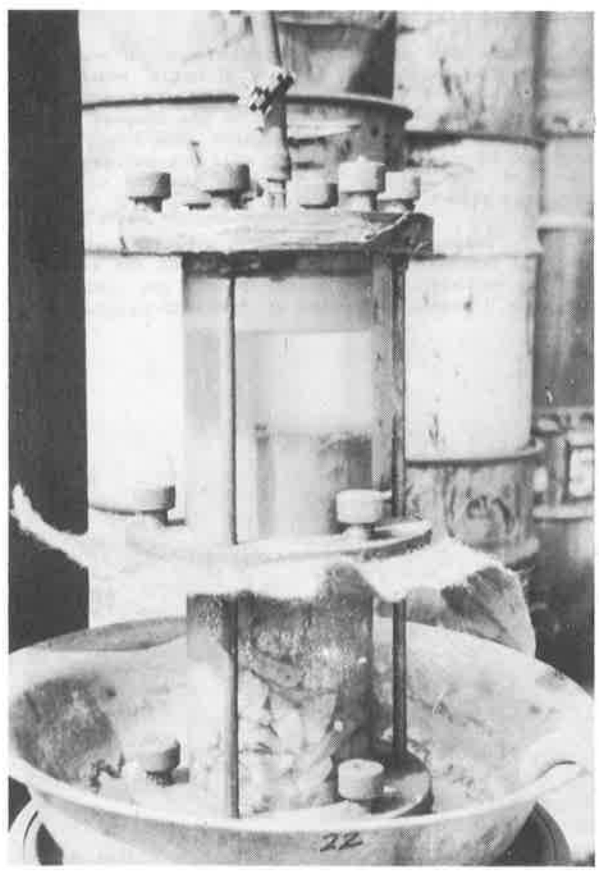

Photo 3 : Laboratory simulation of filter drain performance.

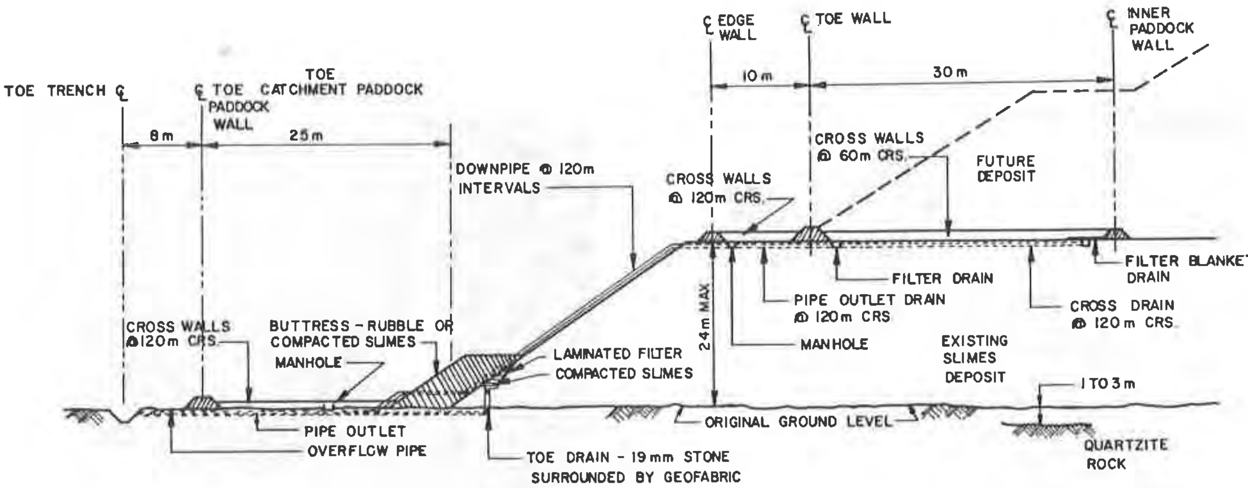

Fig. 2 : Crown Sand Slimes Disposal - Typical section on existing dam

A total of about 75 million tonnes of gold slimes will be placed at the rate of 370 000 tonnes per month on top of two existing deposits covering approximately 285 ha with a total perimeter of 7 800 m.

Drainage Provisions

By present day standards, and bearing in mind the considerable height of deposition to be added, the underdrainage of the existing dams is considered inadequate. Computer simulations of the seepage regime have shown that two sets of drains (see Fig. 2) are required to ensure that seepage will not emerge at the face and that a high piezometric surface will not adversely affect the slope stability.

Firstly, filter drains have been provided below the new wall paddocks around the perimeter of the existing dam top surfaces. These filter drains are similar to the Type C drains installed at the ERGO Tailings dam and illustrated in Fig. 1. A typical installation is shown in Photo 4.

Secondly, new interceptor drains have been installed at the toes of the existing slopes. As shown in the typical detail in Fig. 3, geotextiles have been used both as separator-filters and, in conjunction with an open plastic mesh, to intercept and discharge seepage water in the plane of the fabric. The existing slimes have a grading (100% finer than 425 um, 30 to 95% finer than 75 um and 9 to 18% finer than 2 um) typical of silt or silty fine sand.

Photo 4 : Crown Sand Slimes Disposal - Filter drain under construction with fabric turned back to expose the stone.

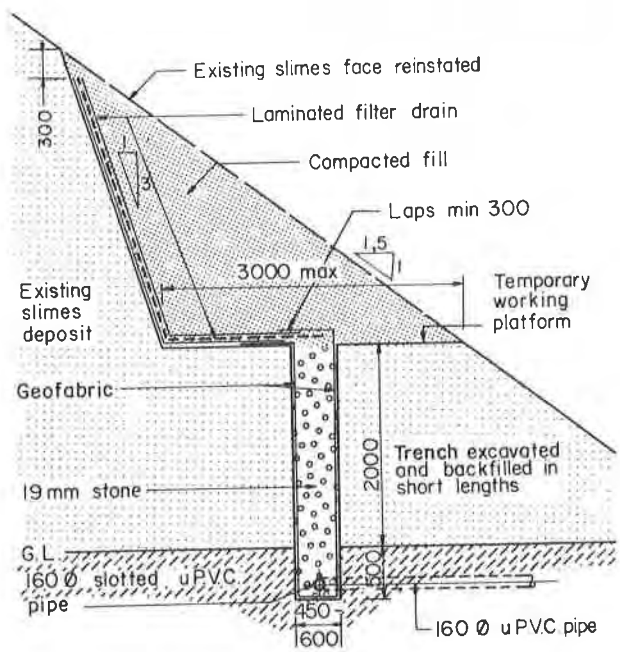

Fig. 3 : Crown Sand Slimes Disposal - Toe drain detail

Photo 5 : Toe drain under construction - Fabric laid in excavated trench, slotted pipe inserted and stone filling started.

Photo 6 : Toe drain under construction - Trench backfilled with filter stone.

The toe drain consists of a 3 m high laminated filter drain leading into a 2 m deep, stone-filled trench which is drained by uPVC outlet pipes. The geofabric prevents the fines from entering the drain while the plastic mesh provides a discharge path for the seepage water to reach the trench and outlet pipes. The laminated filter drain is actually a sandwich of a heavy grade open plastic mesh sandwiched between two layers of geofabric.

The drain was constructed by digging the trench from a working platform excavated into the face above the toe. A sheet of geofabric was inserted, the slotted pipe dropped in (Photo 5) and the trench filled with stone (Photo 6). Before the cut was backfilled with compacted slimes, the laminated filter drain was installed by simply pinning sheets of the material to the near vertical cut face (Photo 7).

Photo 7 : Placing of laminated filter drain.

Photo 8 : Crown Sand Slimes Disposal - Filter drain under healing wall.

Because of the ease and speed of installation the method eliminated construction problems which would otherwise have been experienced with conventional sand filter drains. For safety reasons, most of the working operations were carried out from the platforms and, as far as possible, personnel were kept out of the trench. The trench was opened up in short lengths which could be refilled the same day. In certain areas where the material was soft and wet, the trench had to be refilled immediately after excavation before the sides caved in.

In addition to the drains already described, filter drains were also installed to collect and remove seepage water from a particularly wet area on the side of one of the existing dams. In this area washaways had occurred in the past and a "healing wall" was now built to complete the perimeter for renewed deposition. The construction of this drain is illustrated in Photo 8.

At the time of writing, operation of the scheme is about to begin, so the efficiency of the drainage measures has not yet been observed.

CONCLUSIONS

Two examples of the use of geofabrics in the underdrainage of gold slimes residue deposits have been described. Geofabrics have also been installed in filter drains in other situations, including gypsum and diamond slimes dams.

The earliest applications have been operating for 5 years, while others have been installed only recently. Problems have been encountered at the ERGO Tailings dam with the precipitation of ferric compounds on the geofabric, but possibly similar clogging could have occurred also in graded sand drains without the geofabric. Elsewhere, in the author's experience, the geofabric drains have functioned satisfactorily.

Nevertheless, before the use of geofabrics in residue underdrainage applications can be generally accepted, the most effective installation details must still be confirmed and their satisfactory long term performance proved in practice.

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Geotextiles in Tailings Dike Construction: Laboratory and Field Observations

Les géotextiles dans les digues en stériles: observations en laboratoire et sur le terrain

An experimental dike built to contain tailings by maximizing the use of waste materials in its construction is described. Geotextiles were used to insure the integrity of a blanket drain placed within a starter dike. The results of an equivalent opening size test were used to make a preliminary selection, followed by a permeability-type test. The amount of tailings passing through the geotextiles in this test was used to further narrow the field of candidate fabrics. Field experience in the construction of an experimental dike 800 meters long showed relatively few problems. Based on the test program and the field experience, a list of items to consider in the selection of fabrics to be used in the retention of tailings is suggested.

Voilà une description d'une digue expérimentale construite pour contenir les dépots de résidu en stérile de mine, en utilisant au maximum les matériaux de rebut (déchets). Les géotextiles étaient utilisés pour assurer l'intégrité du drain "de couverture" placé à l'intérieur de la digue initiale. Pour faire la sélection préliminaire, on se servait du résultat d'un essai de dimension d'ouverture équivalente suivi par un essai de perméabilité. La sélection était encore rétrécie en employant la quantité de résidu (en stérile de mine) qui percole à traverse le géotextile dans cet essai. La construction d'une digue expérimentale de 800 m de longueur, sur le terrain, démontrait peu de problèmes. Basé sur le programme d'essai et l'expérience sur le terrain, voici un liste de considerations dans la sélection de textile utilisé à retenir les residus en stérile de mine.

INTRODUCTION

Large quantities of solid waste materials are produced as a result of the mining and processing of mineral ores. Depletion of richer ores has led to the mining of lower-grade ores with lower recovery factors and consequently even larger proportions of waste material. To liberate the mineral values from the ore, it is often necessary to grind the ore down to a very fine powder, thus most of the resulting tailings are finer than 0.075 mm (US sieve 200). The typical recovery process produces large quantities of slurry which tend to form unstable deposits. These must be contained behind suitable dikes.

The design and construction of tailings retention structures depends on many factors. For example, the Canadian Department of Energy, Mines, and Resources has pointed out in its Tentative Design Guide for Mine Waste Embankments in Canada:

> "The design of mine tailings embankments should consider the economics of alternate sites, types of embankment and methods of waste disposal; these will be interdependent. The availability of construction materials, the quantities and characteristics of the wastes, climate, topography, and the nature of the foundations at alternative sites will all be factors" (4).

It is important that the retention structures be designed and constructed to safely store the wastes. Furthermore, as there is no fiscal return from them, the storage system must be as economical as possible.

In some cases, the dam is immediately constructed to the full height required for life-of-mine. This has the advantage that it is easier to control the construction, therefore easier to build a safe dam. Also, the filling process is simpler (and less costly) to control. It is often possible to let the tailings spill into the basin with a minimum of daily attention. In short, this is the low-maintenance facility.

Such construction may be costly, however, and requires a capital investment to be made long in advance of realizing an economic return from the mining venture. Thus there is considerable interest in tailings dike systems which can be constructed only as they are needed, and in making maximum use of the tailings as the construction material.

Three basic methods have evolved for construction with tailings: the upstream method, the downstream method, and the centerline method (4). In the upstream method, a starter dike is constructed of natural soil, and the slurry is spiggoted from the upstream shoulder (Fig. 1). The natural sedimentation process results in the coarsest portion of the tailings being deposited adjacent to the starter dike, with progressively finer material deposited further out into the pond. The clear water is decanted off in a suitable manner. When the basin formed by the starter dike is full, the

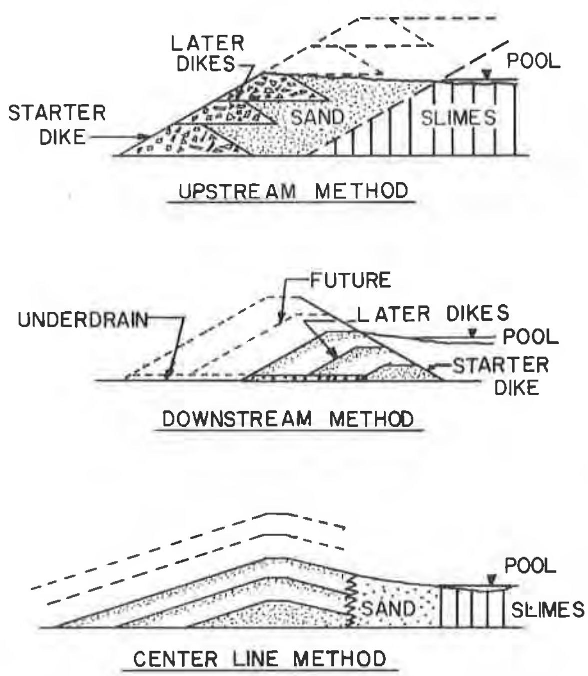

Fig. 1 Three Basic Methods of Constructing Dikes from Tailings.

"sands" are bulldozed or draglined up to the top of the starter dike to form a second level or lift, typically 2 to 8 meters high. Alternately, natural soils may be used. The tailings pipeline is then raised to this new level and filling continued. The method is relatively simple and inexpensive, and can work well if the composition of the tailings is such that an adequate amount of sand or course material is produced. There is one major disadvantage, however, and that is when the dike is raised to large heights, the potential slope failure surface no longer passes through the coarse, relatively stable sands but through the finer "slimes" which may not have sufficient strength to resist shearing (3).

This problem is overcome in the downstream method, in which the sand is removed from the beach and placed on the downstream slope. Alternately, the sand may be separated from the slimes by hydrocycloning, with the sand then being placed directly on the downstream slope (5). Thus the coarse free-draining material is placed where it will be most advantageous. Also, this results in a wide base of stable material such that the critical surface passes through the coarse material only. While this method generally results in a safe dam, it requires a much greater quantity of sand, plus the continual extra cost of equipment to shift the sand. If it is difficult to obtain sufficient coarse material in a given case, this method may not be feasible.

The centerline method is similar to the downstream method in that coarse material from the beach is placed on the downstream side, but the centerline or axis of the dike remains fixed, rather than shifting upstream or downstream as in the previous methods.

Although these methods are attractive because of their simplicity and relatively low cost, they cannot be used in all situations. For example, if the grind of the tailings is very fine, there simply will not be enough coarse material present for any of the above methods to be successful. Also, in the cases where it is desired to raise a filled basin above its original design level, the foundation of the dike may of necessity be on the relatively weak fines, thus sharing the disadvantage of the upstream method. Therefore innovative methods must be developed which are both safe and economical.

As the key to success in earth structures of this kind is drainage, there should be appropriate uses for geotextiles. It is the purpose of this paper to examine some of these possibilities in a specific case.

ORIGINAL CONCEPTS OF FABRIC UTILIZATION

As the original tailings basin in this case study filled up, it became necessary to find additional storage space. Building additional basins on adjacent land was deemed too costly. Therefore other approaches were sought. These generally focused on continuing to use the existing "full" basin by piling the future tailings on top of the tailings already there, and to use the tailings as construction material to the greatest extent practical.

In 1971 a homogeneous cross-section dike was built using tailings as the construction material (1). It was located on a foundation of well drained coarse tailings. In anticipation of the required future construction, exploration was carried out on portions of the basin that were not so well drained and which contained finer grained tailings. Samples were analyzed for shear strength (2), (10) and susceptibility to frost action (9). A study was made of the possibilities of hydrocycloning to obtain coarse tailings for some form of hydraulic construction, but it was concluded that obtaining enough coarse material would be difficult, perhaps impossible (5). Therefore a modified homogeneous cross-section with a blanket drain was selected.

The earlier dam showed a marked susceptibility to erosion (1), therefore it was considered very important to provide adequate filters to prevent piping (6), (8). In addition, stability analysis indicated that a dike could be built if there were adequate provision for drainage of the foundation tailings (10). A blanket drain would thus serve to drain the foundation as it came under load from the embankment, and serve as a toe drain to the tailings slope which would be eventually raised above the starter dike. Both the blanket drain and the tailings fill above it would be placed by conventional earthmoving equipment. Therefore to create the new storage basin, modified homogeneous dam sections would be placed around the perimeter of the existing basin. To gain practical experience with this method before construction became urgent, a trial embankment was to be constructed on a portion of the basin reserved for research purposes. This would also provide experience with the foundation of the dike, consisting of deposits of fine-grained tailings ranging in thickness to 25 meters.

It was originally planned to use waste rock from the mine in the blanket drain, as this would put some of the abundant waste rock to good use, and the resulting drain would have excellent permeability. However, it was determined that this would require several layers of intermediate materials, and the total thickness required would be too great. At this point, the possible use of a "filter fabric" began to be attractive.

To intercept the anticipated horizontal seepage through the stratified foundation tailings, a vertical cut-off drain was to be tied in to the blanket drain (Fig. 2). Fabric would be placed completely around the rock to prevent the tailings from moving into and clogging the drain.

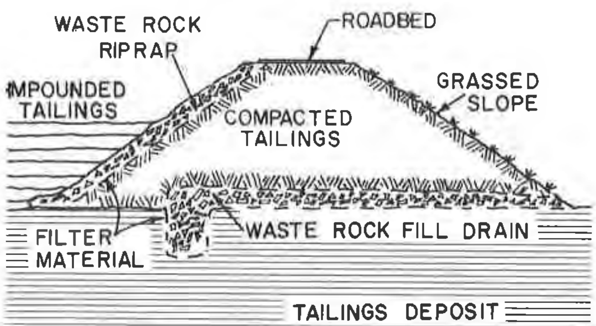

Fig. 2 Preliminary Concept of Coarse Rock Blanket Drain Protected by Filter Fabric.

To investigate the potential for using geotextiles, the available materials and their characteristics were reviewed and textile selection criteria were developed (7). Preventing the migration of fines was of key importance because little experience had been reported in the literature with soils as fine-grained as these tailings. With these considerations in mind, an equivalent opening size test was developed (7). The tailings were washed through a piece of fabric clamped between two parts

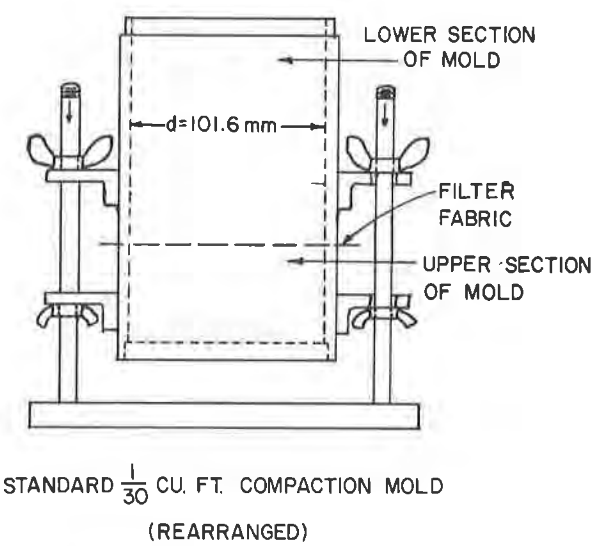

Fig. 3 Equipment for Improvised Equivalent Opening Size Test.

of a compaction mold and the percent passing determined (Fig. 3). Any tendency for the tailings to excessively abrade the fabric was evaluated on a visual basis.

Using this as a first screening test, the possible field of candidate fabrics was narrowed to four which were then subjected to additional testing. This test involved the use of a permeameter-like device, with the fabric holding a compacted sample of tailings in place while water flowed downward through the sample under constant head conditions (Fig. 4). In addition to evaluating the permeability of the system, this permitted a

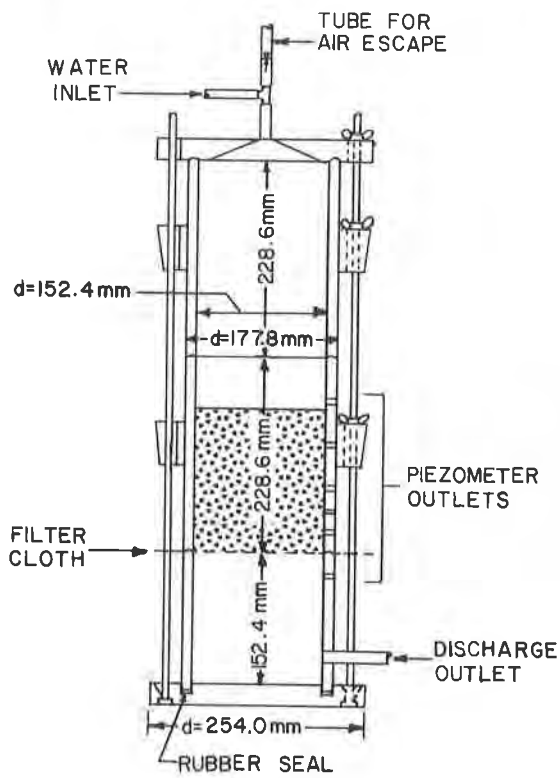

Fig. 4 Permeameter-Type Devise for Determining Loss of Tailings Through Fabric

visual check of whether any tailings were passing through the fabric (7).

This part of the study concluded that either "A" or "C" would be suitable, with "A" apparently having a slight edge. Table 1 summarizes the results of the two stages in the selection process, showing how the original seven candidates were reduced to four by the EOS test and other considerations, these four being further narrowed down to two.

MODIFIED DESIGN SELECTED

Because of economic conditions and mine operating considerations, the construction of the dike was delayed for three years following the study on fabrics. In the interim, foundation studies were

Table 1. Fabric Test Results and Selection Decisions

Fabric	Type	Equivalent Opening Size Test			First Stage Decision	Permeability Test	
		% Soil passing	EOS mm	Equivalent US sieve		Soil Passing	Suitable?
A	non-woven	77.2	0.09	200	test further	negligible	Yes
B	non-woven	90.1	0.12	120	eliminated (a)*	---	(No)
C	non-woven	91.3	0.12	120	test further	very little	Yes
D	non-woven	97.3	0.15	100	test further	considerable	No (d)
E	woven	97.7	0.15	100	eliminated (b)	---	(No)
F	woven	98.7	0.16	100	test further	excessive	(No) (e)
G	woven	99.7	--	---	eliminated (c)	---	(No)

* NOTES: (a) high cost (b) apparent low strength and durability (c) functionally ineffective (d) stretched excessively (e) functionally ineffective

conducted with dynamic and quasi-static cone penetrometers, and the proposed alinement of the dike was shifted to adapt to changed operating conditions. Cone penetrometer studies on the new alinement showed a relatively stiff dry layer of tailings overlying a softer deposit. This crust was about 3 meters thick on the south end of the proposed dike axis, and gradually thinned to about 1.5 meters at the north end. It was thought that the weight of the dike and the eventual filling behind the dike would result in excessive settlement or instability due to the presence of this softer layer at depth.

To simplify construction, it was decided to eliminate the key or trench drain feature of the original cross-section design. Two drain designs would be used. One would be a simple blanket drain about 1 meter in thickness, while the other would have a raised portion near the upstream face. This raised portion would provide better drainage for the tailings to be placed behind the dike (Fig. 5).

As the time for construction drew near, concern developed about possible construction difficulties, especially in the vicinity of a low spot about 100 meters across where water had ponded. There was some difficulty draining it before construction began. Therefore it was decided to use a high-modulus woven fabric, which had not been available at the time of the lab evaluation, in addition to fabric "A". Thus the project would be divided into four sections, one representing each of the four possible combinations of cross-section and fabric (Fig. 5).

CONSTRUCTION PROCEDURES

The construction sequence was to place the lower fabric, then place the sand over it in a 0.5 meter lift. Following that, the sand was brought up to full height, then the second layer of fabric was placed. With the upper fabric layer in place, the cross-section was completed with the coarsest available tailings.

Construction began on the south end, nearest the source of the sand borrow. The slope stakes provided a guide for placing the fabric, which was rolled out perpendicular to the dam axis, with approximately 0.4 meter overlap. It was decided to run the fabric across the dike rather than parallel, as the most likely movement would be a spreading in the transverse direction.

The work generally proceeded well, in spite of often windy conditions. The two men who rolled out the fabric were able to anchor the fabric by hand shoveling sand to tie down the edges, and still manage to lay down the fabric rapidly enough to keep ahead of the scrapers bringing in the sand.

The contractor was reluctant to have the scrapers run off the end of the fill and over the bare fabric, so the scrapers dumped their loads and let a dozer push the sand ahead to cover the fabric. To avoid excessive backing and loss of time, the scrapers were driven off the side of the fill so that they could return to the borrow pit by running over the tailings surface. This generally worked well until excessive rutting was noted at one point, and the non-woven fabric being used there failed in tension over the pressure ridge formed between the two wheel tracks (Fig. 5). Fortunately there was no evidence of failure under the body of the fill; also, the scheduled change from non-woven to woven fabric occurred just a few meters away.

The reason for the difficulty in this area was that water had ponded there during the spring and early summer, and had not drained or evaporated. It was drained by a ditching operation only a few weeks before construction began, and thus was still wet. Changing to the high-modulus fabric permitted the construction to proceed, although it was also necessary to increase the lift thickness to successfully get across this spot.

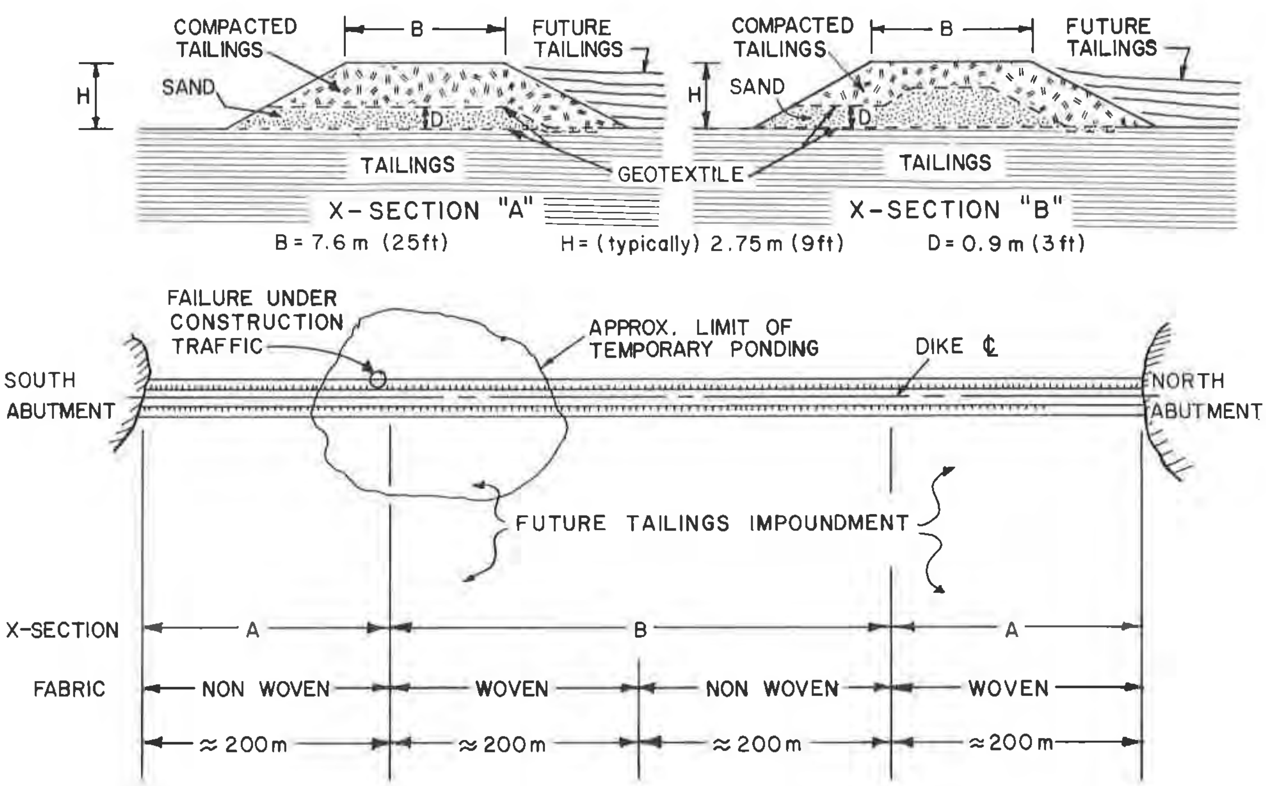

Fig. 5 Typical Cross-Sections and General Plan of Experimental Tailings Dike

With further experience, the contractor found that he could run the scrapers off the end of the fill across the bare fabric if the machines moved slowly and steadily. There were no further difficulties with bearing capacity, although when it was later necessary to build a side ramp for the scrapers to turn off from the dike, fabric was used to insure that there would be no difficulty.

When the sand for the drain had been placed, the second layer of fabric was placed over it, with the fabric overlapping the first layer at the toe of the slope. The dike was then brought up to grade with tailings with no unusual difficulty. A layer of gravel was placed on the top of the dike to carry truck traffic, and rip-rap was placed on the slopes.

OBSERVATIONS AND CONCLUSIONS

The deep-seated instability by a rotational or sliding block movement that was considered a possibility did not, in fact, occur. There was no apparent buildup of pore pressure due to the load of the embankment. The peizometers did not show any change in pore pressure after the dike was placed.

There has been no opportunity to observe the ultimate behavior of the project, as economic conditions caused a delay in the filling behind the dike.

Both fabrics were easy to handle, in spite of some fairly strong winds. This is important in application to tailings basins, as they are often subject to high winds because of their exposure. Laying down a minimum amount of fabric in advance of covering with fill reduces the chance for difficulties due to sudden gusts of wind.

When heavy earthmoving equipment is used, variations in the trafficability of the top meter or so of the foundation tailings may be significant to the success of the placement of the first lift of fill. To continue successfully with construction, it may be necessary to change the thickness of a layer, as was done in the case described, thus changing the cross-section as designed. If such a design change due to field conditions is unacceptable, other alternatives should be readily available. One alternative would be to use lighter equipment. This probably would have worked in the case cited. Another possibility is to have alternate fabrics on hand on the jobsite, so that localized difficult spots can be readily dealt with. While intensive sampling or surface testing, say with a cone penetrometer, would help anticipate these spots, careful observation of the machinery and its effects on the ground can be very effective in locating difficult spots in advance.

Careful study of the tailings to be protected, along with consideration of the true function of the fabric, is necessary. In the case reported here, it was possible to use tailings in the embankment which were somewhat coarser than originally planned for. This would mean that a larger EOS might have been acceptable.

FABRIC SELECTION FOR USE ON TAILINGS: A CHECK LIST

Based on experience with the project cited above, a check list for fabric selection is presented. It is not claimed that this list is all-inclusive, but it should be useful to those contemplating tailings dike construction with fabrics.

1. What is the function of the fabric? What objective is to be realized?
   A. Separation, such as keeping the tailings out of the permeable drainage layer.
   B. Facilitate construction traffic.
   C. Direct drainage, that is the textile itself is the prime drainage agent, as by wick action.
   D. Other.
   RESULT: This should help to establish the general type of fabric desired.

2. What is the range of gradation of the tailings?
   A. Relatively consistent.
   B. Stratified.
   C. Systematic change in gradation, as for example, from coarse at the point of discharge to very fine at the pond.
   RESULT: Indication of the required EOS.

3. What materials are available for the drainage layers, traffic layers, etc., and what are the pertinent characteristics of these materials?
   A. Coarse, angular, sharp particles, such as mine stripping rock, would be potentially damaging to the fabric.
   B. Pit run sand and gravel would tend to be more friendly to the fabric.
   RESULT: Relative importance of tear strength and modulus as they affect construction survivability.

4. What equipment and construction methods will probably be used?
   A. Heavy earthmoving equipment will result in greater tensile stresses in the fabric.
   B. Heavy equipment combined with sharp angular large particles would increase the tendency to tear the fabric.
   RESULT: Further definition of strength and other survivability properties required of the fabric.

5. What are likely to be the conditions at the time of construction? Are they likely to change on a seasonal basis?
   A. Extremely windy.
   B. Extremely wet.
   RESULT: In windy conditions, small rolls of fabric may minimize the problems of the fabric being blown away. If conditions are so wet that the fabric must be handled mostly by manpower, smaller rolls will be better.

6. What is the probability that unexpected conditions will appear during construction?
   A. Exceptionally wet or weak foundation.
   B. Would critical locations control the entire design, or should special designs be developed, perhaps using fabrics specially selected for these conditions?
   RESULT: A decision would be made whether to develop alternate designs, and if so, to be sure the necessary materials were available if and when needed.

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Continuous Retaining Dikes by Means of Geotextiles

Digues continues de rétention au moyen de géotextiles

In the city of Cubatão, in São Paulo, Brazil, the City Hall, in conjunction with the D.A.E.E. Department of Water and Electric Energy, has been carrying out a large hydraulic fill project since 1980, a fill upon which will be built the city of "New Cubatão", with roughly 32.500 inhabitants. It was expected that this fill be contained along its entire length by dikes built with the soil derived from on-site excavation of the projected drainage channels. However, during construction on the very soft organic clay existing particularly in some spots, the drag lines would sink, despite the fact that the work was done on wooden grating. Field tests were conducted using geotextiles sewn together lengthwise and filled with the dredged material. Theses envelopes of fabric varying in length and with an average height of 1.4 meters successfully replaced the conventional dikes without burdening the work and allowed for a threefold increase (or more) in work performance time, even with rain. This paper deals with work site conditions, field tests performed, improvement of the filling process, and the building of more than 4.000 meters of continuous geotextile dikes.

La préfecture de Cubatão, São Paulo - Brésil, et le DAEE Département d'Eaux et Enérgie Électrique, exécutent depuis 1980, un grand terrassement hydraulique où sera construite la "Nouvelle Cubatão", ville qui aura une population d'à peu près 32.500 habitants. Ce terrassement aurait dû être retenu par des digues construites avec le sol local, provenant des creusements de canaux de drainage. Toutefois, pendant la construction, les "drag-lines" s'enfonçaint, particulièrement sur l'argile très molle existente en certains endroits, même travaillant sur des treillis de bois. Des essais de champ ont été alors exécutés, en employant des géotextiles cousus et remplis avec le matériel dragué. Ces éléments, de longueur variables et hauteur moyenne de 1,4 m, ont substitué avec succès les digues conventionnelles, sans être onéreux au chantier et permettant une vitesse d'exécution au moins trois fois plus grande, même sous la pluie. Ce travail rapporte les conditions locales du chantier, les essais effectués en champ et la construction de plus de 4.000 m de digues continues avec géotextile.

I . INTRODUCTION

In various Brazilians cities, a low-cost housing program is being developed to house the low income classes that usually inhabit the shanties in substandard living conditions offering poor health and hygiene conditions. Because it is a far-reaching and far-extending program, the solution making it economically viable lies on reclaiming useless land by building polders, mechanical fills, or hydraulic fills.

One of the areas to be benefitted by this programa is in Cubatão, São Paulo where a "New Cubatão" is to be erected with the infra-structure needed to house roughly 32.500 inhabitants in 6.500 homes. This area, however, was subject to flooding because of the tidal range, a problem requiring the study of two basic solutions: converting the entire area into a polder or making a hydraulic fill secured by dikes. The hydraulic fill proved a more economical solution provided that all dikes would be built with the material retrieved from projected channel drainage excavations.

When the work began in July, 1980, dragline performance was seen to be low, primarily when working on very soft silty organic clay. The machines would sink in the soft soil even when treading on wooden grating, and recovery of the machines was slow and difficult. Erection of the dikes with imported soil was out of the question in that the average distance of transport from source was 15 kilometers a factor that would make the project economically unfeasible.

The idea of using geotextiles as a containing medium arose in late 1980 with the news of a similar project being conducted on the Seine in France. Unfortunately, enough information on the project was not available, prompting the conducting of field tests to ascertain the problems associated with using the medium, observe what shape the medium would acquire, and determine fabric quantities and labor requirements for the job.

II. LOCAL CONDITIONS

The areas to be reclaimed are on the São Paulo State coastline in the boggy marshland between the Anchieta and Imigrantes highways. Filling the 130 hectares that make up the four large-sized blocks of the residential project call for 4×10^6 m^3 of material being dredged from a 40 hectare plot of land that, once the work is concluded, will be turned into a lake for the new city.

Probes made on site revealed the existence of three basic types of soil :

- fine sand ("good" soil)
- silty or clayey sand or sandy clay ("fair" soil)
- silty dark grey and very soft organic clay ("poor" soil)

Capacity of the "good" soil is likewise good, and therefore can be easily dredged, without presenting any problems The "fair" and "poor" soils, besides being problematic make up most of the area in question. After performing various tests, the following average values were adopted for calculating load resistance and capacity:

"Poor" soil: $\Phi_u = 0^o$; $C_u = 8.0$ kPa ; $\gamma_{sat} = 14.0$ kN/m^3

"Fair" soil: $\Phi_u = 15^o$; $C_u = 8.0$ kPa ; $\gamma_{sat} = 16.0$ kN/m^3

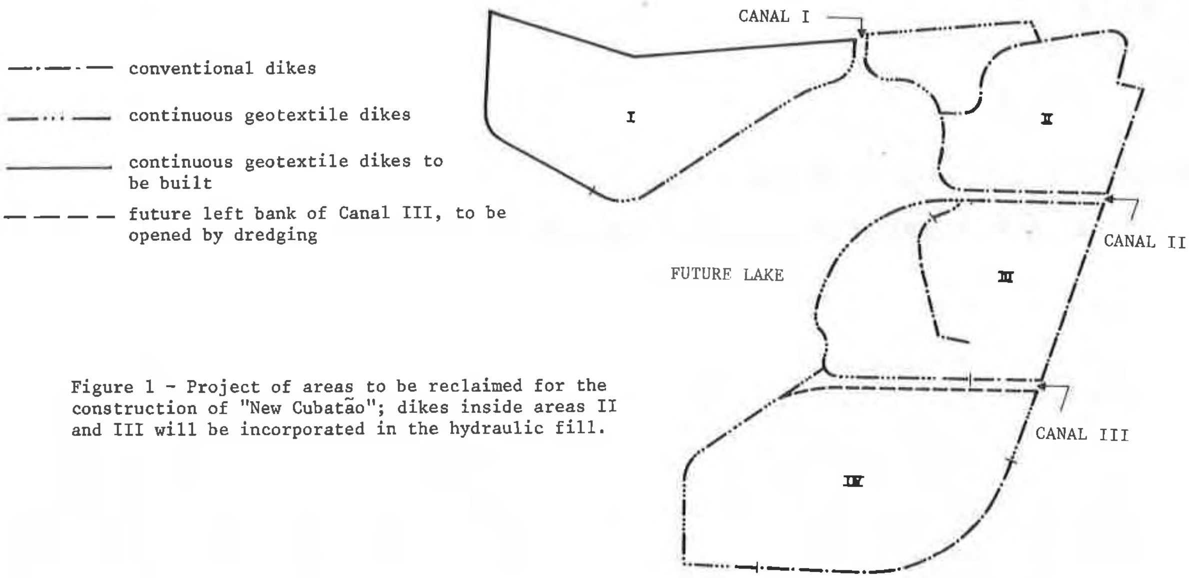

Figure 1 - Project of areas to be reclaimed for the construction of "New Cubatão"; dikes inside areas II and III will be incorporated in the hydraulic fill.

III. THE PROJECT AND THE WORK

The project calls for the building of 4 fill zones separated by drainage channels and adjacent to the future lake from where the soil will be taken. Datum points of fills, along their extremities and in contact with the dikes braces, range from 1.40 to 1.45 (height of the dikes is between 0.95 and 1.00, respectively). Toward their center, fills should have a datum point reaching 1.70 m, but average surface declivity should be 1 $^{o}/oo$ (figure 1).

After the site was cleared work began with dikes built by draglines, and fills built by dredging machines. In building the dikes two problems upset the work schedule :

- Regardless of the type of soil excavated, the deposited soil shape did not conform to project demands; the solution was to wait until the saturated soil dried and to form the dike manually in accordance with the project.
- In "poor" soils the draglines would sink even though treading over wooden grating; this hindered average machine performance considerably. Operating both over "good" or "fair" soil, each dragline was able to perform an average of 4 meters per hours of dike work; however, over the "poor" soil, its performance would drop to half of that amount. Therefore, in a daily 10 hour work period, each machine could render an average production rate of 20 to 40 meters of dike building.

The final shape of the dikes exhibited the following geometric features :

- height: 0.95 - slope inclination: 1V : 4H
- largest base: 15.10 m - cross section area: S = 10.73 m^2

IV. TESTS WITH CONTINUOUS DIKES

a) In France

In late 1980 we learned of an experiment performed using a nonwoven geotextile in a similar project. The goal of this experiment (1) conducted in August 1977 was to determine if, by filling a geotextile with dredged material, a sausage could be made to serve a provisional dike for a large hydraulic fill to be made at the estuary of the Seine in France. For this purpose, 300 meters of dike were built using two rolls of 5.30 m wide Bidim U-64 folded and sewn lengthwise. This enourmous "sausage" was filled by way of metal tubing joined to one of the ends of the "sausage".

During the filling process a rupture developedin the tube/geotextile connection, though it was fixed without any problem.

Also observed was a shift in the position of the dike toward the sea as the dike was being built, due to the launching of the fill and filling of the dike at the same time. The dike took on the shape of an arc, though this factor did not upset its purpose.

This first experiment done with nonwoven material was a complete success and encouraged us to conduct supplementary tests in Brazil for immediate on-site application.

b) In Brazil

first test :

At the same time as work was being initiated in Cubatão, São Paulo, a similar project had started in the city of São Luiz, Maranhão. In line with the program of building low-cost houses, the São Luiz project intended to build hydraulic fills secured by rock fill, since the tide range in that region was 7 meters. But the building of these rock fill embankments proved very expensive due to the great transport distance and great volumes involved. As a result, two solutions using geotextiles were proposed:

- containment using "sausages"
- containment using palisades covered with a geotextile and protected by small rock fills

In late 1980 expriments using the palisades were conducted following through with the entire building sequence. This solution proved to be technically and economically viable.

In January 1981 we began the first experiment with "sausages" in Brazil. The work conditions at this site differed greatly from those in Cubatão in that the dikes were built on beach sand and filled with sand. To broaden our knowledge of continuous dikes we decided to use Bidim OP-30, whose resistance to monodirectional traction is 16 kN/m, about half the resistance of the U-64 type.

Acoordingly, 50 meters of 4.30 m wide OP-30 were foldedin half and sewn lenthwise. One of the extremities was sewn together and the other joined to the metallic tube. The geotextile was extended and positioned so that the closed extremety lay at a lower datum point than the dredge-fed extremity. Filling of the sausage started at low tide while the hydraulic fill was being from one of the dredge tube sources. The sausage feeding pipe had a gate to regulate the outflow which could be controlled within a range from 0 to 1,000 m^3/h, and a 15% solid material concentration. Two problems arose: bursting of the nonwoven fabric 2 m beyond the tube/sausage junction, and bursting at the upper generatrix one meter from the closed end of the sausage. Also observed was a near 2 meter shifting of the dike (translation and rotation), that did not, however, hamper the experiment. The rupture at junction was due to the erosive cutting action of the jet of dredged material containing, besides sand, small sharp shells. The rupture at the opposite end was probably due to greater hydraulic pressure and seepage force, considering the spot was at a lower datum point.

We sewed a new sausage with the same measurements, but with a triple layer of nonwoven fabric in the first and last 8 meters. We proceeded to fill it in the same manner but this time no problems ensued. We observed that the dike takes on the shape into which it has been filled, further determined by its length, and that its cross section resembles an ellipse.

To derive the numerical correlations of the experiment, we presumed that the cross section was an ellipse. Thus the following formulas become valid:

$$S = \frac{\pi}{4} \cdot B \cdot H \quad \text{.......} \quad (1)$$

$$p = \frac{\pi}{2} \cdot (B + H) \cdot \frac{64 - 3\left(\frac{B - H}{B + H}\right)^4}{64 - 16\left(\frac{B - H}{B + H}\right)^2} \quad \text{.......} \quad (2)$$

In that : S = cross section area of ellipse

p = perimeter of ellipse

B = width of ellipse

H = height of ellipse

In the center cross section we measured the width "B", half-height " $\frac{H}{2}$ " and apparent height " H_a " -- defined in figure 2. Owing to the inclination of the beach surface, it was not possible to measure the useful height " H_u ". The following measurements were taken:

p_o = 4.20 m (undeformed perimeter)

B = 2.00 m (width of the dike)

H = 0.80 m (effective height of the dike)

H_a = 0.70 m (apparent height of the dike)

The following ratios were then determined :

$F = \frac{H}{B}$ = shape factor (adimentional) .... (3)

$\frac{p_o}{H}$ = perimeter/height ratio (adimentional)... (4)

$H - H_u$ = real sinkage (m) .... (5)

$H - H_a$ = apparent sinkage (m) .... (6)

$\varepsilon = \frac{(p - p_o)}{p_o} \times 100$ = average specific deformation along the length of the perimeter (%) .... (7)

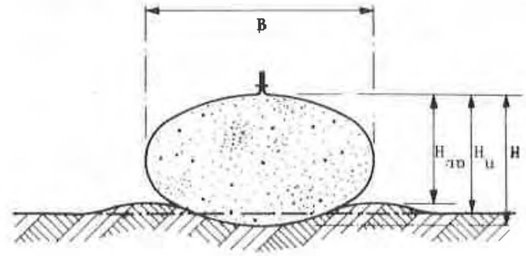

Figure 2 - Continuous geotextile dike cross section; symbols used: width (B), apparent height (H_{ap}), useful height (H_u) and effective height (**H**).

The following calculations were then made :

- using (1) .... S = 1.26 m^2 (cross section area of dike)
- using (2) .... p = 4.60 m (work perimeter of dike)

F = 0.4 (shape factor) .... (8)

$\frac{p_o}{H}$ = 5.25 (perimeter/height ratio) .... (9)

$H - H_a$ = (apparent sinkage) .... (10)

ξ = 9.5% (average specific deformation) ...... (11)

Because the project specified the height of the fill as 4.00 m, one possible solution was to terrace the fill on tree levels with an useful height of 1.33 m, each terrace. Assuming the real sinkage of the dike was 5 cm (half the apparent sinkage), according to (5) we have :

$H = H_u + 0.05 \text{ m} = 1.38 \text{ m}$

Thus three dikes would be needed with effective heights of 1.38 m, the undeformed perimeter of each being, according to (9):

$p_o = H \times 5.25 = 1.38 \times 5.25 = 7.25 \text{ m}$

Although the sausage solution was technically viable for this project, the alternative using palisades with Bidim OP-30 was adopted. This solution proved perfect: technically, economically and socially viable, absorbing a large local labor force recruited from future inhabitants of the area. (figure 3)

second test :

The second experiment with continuous dikes was conducted in February 1981 on the worksite in Cubatão. Test conditions, however, proved to be more several since the foundation was a saturated organic silty clay with low bearing

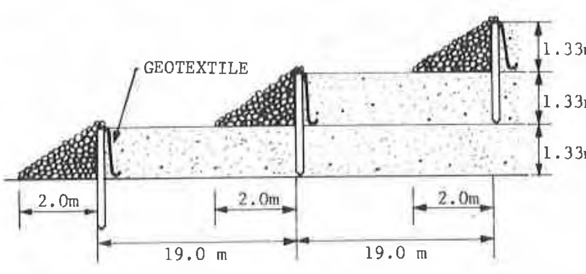

Figure 3 - Solution using palisades: wooden grates staked into the ground and covered with nonwoven geotextile

capacity, and the dredged material was the organic clay itself. Therefore, we could expect to have greater sinkage of the sausage and a greater strain on geotextile traction, since the clayey and/or silty particle would form a cake on the entire nonwoven inside surface. We opted in favor of using 4.30 m wide Bidim OP-60 with a 32 k N/m resistance to monodirectional traction. The geotextile, 90 m long, was sewn in the same manner as in the first test, with a double layer of fabric along the first 10 meters of the end connected to the tube and with a valve in the center section. The seam running lengthwise was positioned on the upper generatrix of the sausage. Were the internal pressure to rise greatly from the effect of the cake of fine particles, the seam on the valve would come apart. The tubing inlet had a 12 inch diameter and a gate allowing for outflow control within the range from 0 to 400 m^3 per hour, with a 20% concentration of solids. Once the test began, it took only 30 minutes of pumping to fill the entire sausage, which grew to an apparent height of 0.85 m.

During the filling process water managed to permeate the geotextile only in the first 15 minutes, subsiding to a very low level of permeability of the cake/geotextile during the latter 15 minute period. Once the control valve was closed the apparent height of the sausage had fallen to 0.65 m after 8 hours. New pumping cycles were effected but the height remained virtually unchanged.

From this test we could conclude that the "poor" material very conclusively could not serve as a filling for the sausage and that certain devices had to be created to allow for discharge of the clayey particles suspended in water during the filling process.

Is was not possible to measure the size of the dike's cross section in this test, in that an ellipse dit not take shape.

third test :

In March 1981 the third test was conducted. Based on results from the first two tests, the filling process was changed. As the idea was to incorporate this test into the work being done on site, it was necessary to estimate the perimeter "p_o" of the sausage to an useful height of 1 meter.

Estimating a real sinkage of 10 cm (double the amount adopted for the sandy foundation), using (5) and (9),we obtained :

$$H = 1.00 + 0.10 = 1.10 \text{ m} \; ; \; p_o = 1.10 \cdot 5.25 = 5.78 \text{ m}$$

To attain this perimeter we had to make two longitudinal seams by joining two sheets of fabric. Since the loss of material in each seam was set at 0.10 m, two sheets of fabric would be needed, whose widths measured 5.98 m or more together. The solution was to use two bobins of OP-60 with a 100 m length, in that one's width was 4.30m and the other's was 2.15 m. Thus we achieved an undeformed perimeter of 6.25 m. The sheets of fabric were sewn with a portable Newlong NP-7 sewing machine, and a 2820 dtex nylon thread. The two ends were sewn together, and at every 20 meters (at the upper generatrix) a 0.30 m diameter opening was left unsewn. The filling process was done with a flexible plastic tube measuring 6" in diameter, inserted in the upper openings (figure 4). The dredger selected only "good" or "fair" material, and avoided picking up the organic clay. While the mixture of water and solid particles would enter one of the openings, only water with clay in suspension would be released from the other openings, thus eliminating the problem of increased hydraulic pressure inside the sausage. Only one problem arose during the experiment: at a given

Figure 4 - Sausage filling through a flexible plastic pipe.

moment it was noticed the occurance of a new material called "tabatinga", a soft pasty sedimentary clay with some grade of organic matter, wich during the pumping formed round clay pellets up to 10 cm diameter. As the pellets became deposited, advancement of the flow inside the sausage became obstructed; the solution was to switch the flexible pipe over to another opening. The test resulted in a dike measuring an useful height of 1.06 m filled hydraulically at atmospheric pressure.

V. ERECTING CONTINUOUS DIKES AT THE "NEW CUBATÃO" SITE

The total extent of dikes projected was 9,000 meters. Initially, 3,900 meters were built by excavating via draglines. However, 5,100 m still had to be built with the geotextile.

In June 1981 the continuous dikes started to be built with the adoption of one change in the spacing of the upper openings admitting and discharging the dredged material, from 20 meters to 8 meter (figure 5).

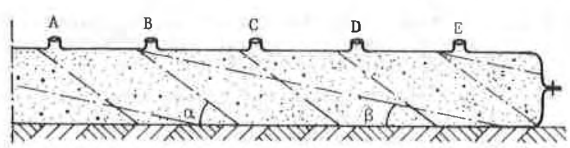

Figure 5 - Longitudinal section of the sausage; if the material is "tabatinga", feeding should be done successively through opening A, B, C, D, and E (material with a resting angle of α); if filling is done with "fair" soil, use only opening B and E (material with a resting angle of β).

In this fashion, the dikes could be filled with both the "tabatinga" and the "fair" material (figure 6), the only difference being the operational yield: the "fair" soil spread better, resulting in an approximate slope of 1 V : 20H, whereas the "tabatinga" settled with an approximate slope of 1 V : 8H, and required the successive introduction of the flexible feeding pipe in all dike opening. To avoid translation and/or rotation of the dikes during the filling process, which is done at the same time as the

Figure 6 - Cut section of sausage showing filling material already settled.

landfill process, the upper generatrix was anchored to the ground by ropes tied at 10 meter intervals. Filling was done via flexible plastic tubing fed by a I.H.C. 1500 dredger, a process wherein 250 m^3/h of dredged mixture was cast into the dikes and the balance of 550 m^3/h used to fill the area. In this way, a maximum of 16 m of sausage could be filled per hour per machine, and, on average terms (dredger maintenance), 12m/h/machine. Three dredgers operated 24-hours-a-day on the site, but only two had sausage-feeding inlets.

By employing continuous dikes at the site, changes could be made in the construction project of radial canals. In the original project, the canals were excavated by draglines and the excavated material used to build the dikes. With the new process, the areas are first filled and, after the fills have settled (an average of from 3 to 6 months), the canals are opened by the dredgers (figure 1). Hence, draglines were eliminated in site operations.

This was the process used to build 3,850 m of sausages (figures 7 and 8), using 25,000 m^2 of geotextile fabric, accomplished by the end of February 1982. By August 1982, the remainning 1,250 m should be ready, in all, a total of 33,000 m^2 of geotextile fabric.

To determine cross section features of these dikes we took measurements at 10 different spots along the length of the dikes built, and have shown the results in table 1, wherein all tests and projects described in this paper are summarized. In table 2 we have summarized features of the geotextiles employed. The economic studies made, determining the costs of such items as materials, labor, machinery and social charges, show that overall costs of the two construction processes, namely conventional dikes and continuous dikes with a nonwoven geotextile, are on a par. It can be said that there was a virtual compensation of energy costs spent in the original machinery-oriented project by the cost of the geotextile in the alternative project adopted.

Figure 7 - Final phase of hydraulic construction.

CONCLUSIONS

The experiments and sites described in this paper allow us to draw the following conclusions :

- It is entirely possible to construct continuous dikes by using nonwoven geotextiles of varying lengths and heights, and filled hydraulically with sand or other type of soil, provided it is a sandy one.
- The perimeter of the cross section of these envelopes resembles an ellipse.
- The greater the density of the solid material dragged into the sausage and/or the smaller the deformation module of the nonwoven fabric, the flatter the ellipse.
- In regions where the soil is of low bearing capacity, this system proves to be of particular interest; at the Cubatão work-site the introduction of continuous

Table 1 - Summary of experiments and sites with continuous dikes reported in this paper.

site/place	geotextiles	foundation	dredged material	lenght (m)	P_o (m)	B (m)	H (m)	H_{ap} (m)	H_u (m)	$H-H_{ap}$ (m)	$H-H_u$ (m)	F (-)	P_o/H (-)	S (m2)	p (m)	ε (%)
test in France	Bidim U-64	sand	sand	300	-	-	-	-	-	-	-	-	-	-	-	-
test in Brazil São Luiz-MA	Bidim OP-30	sand	sand	50	4,20	2,0	0,80	0,70	0,75*	0,10	0,05*	0,40	5,25	1,26	4,60	9,5
test in Brazil Cubatão-SP	Bidim OP-60	"poor" soil	"poor" soil	90	4,20	-	-	0,65	-	-	-	-	-	-	-	-
test in Brazil Cubatão-SP	Bidim OP-60	"poor" soil	"fair" soil or "tabatinga"	100	6,25	-	-	-	1,10	-	-	-	-	-	-	-
work in Brazil Cubatão-SP	Bidim OP-60	"poor" soil	"fair" soil or "tabatinga"	3850	6,25	2,70	1,42	0,82	1,06	0,60	0,36	0,53	4,40	3,01	6,63	6,1

* assumed values

dikes increased work performance time from a minimum of three times over to a maximum of eight times over the conventional process.

The lower the bearing capacity of the foundation, the greater the sinkage of the dike, reducing useful height.

Figure 8 - Finished dike and landfill; the filled area to the right and site of the future lake to the left.

The average specific deformations of the geotextile calculated for cross sections of the dike were less than 10%; in standard bidementional tests of stress vs. strain, the deformation at rupture of the nonwoven Bidim geotextile runs in the neighborhood of 30%; this leads us to assume a near 3 safety coefficient.

Analyzing the experiments performed, it can be concluded that there is much that can still be done, primarily in terms of better design of the dikes. Questions relating to stress vs. deformation at long term, filling with other materials, strains caused by overloading, sinkage as related to bearing capacity of foundation, the shape

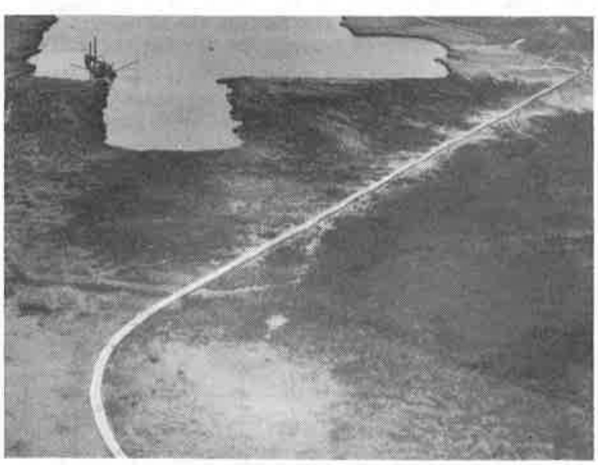

Figure 9 - Aerial view of continuous dike.

factor in terms of geotextile deformation module, the shape factor in terms od density of filling material, specific deformation along the cross section perimeter, etc., all require that further field and laboratory tests be performed in the future.

Table 2 - Features of geotextiles mentioned in this paper

nature	type	mass (g/m2)	normal permeabi lity (m/s)	monodi rectional traction (kN/m)	manu- factu rer
nonwoven fabrics produced with mechanically needled conti- nuous polyester filaments.	Bidim OP-30	300	$2,2x10^{-3}$	16	Rhodia Brazil
	Bidim OP-60	600	$2,2x10^{-3}$	32	Rhodia Brazil
	Bidim U-64	550	$3,0x10^{-3}$	33	Rhône Poulenc France

Because it is an easy and quick method, we believe this new manner of constructing dikes can replace, in some cases, the traditional procedure used in building dams and roads, and in fluvial and marine construction sites.

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Geotextile Applications to Slope Protection for the Tennessee-Tombigbee Waterway Divide Cut

Applications géotextiles pour la protection des terrassements de partage des eaux du Tennessee-Tombigbee Waterway

Woven filter cloth is being used in lieu of conventional granular bedding material beneath slope protection stone on the slopes of the Divide Cut of the Tennessee Tombigbee Waterway in Northeastern Mississippi, USA. Protection from 0.9 meter (3 foot) traffic generated waves must be provided for 1V on 2H to 1V on 3H slopes largely underlain by fine silty sands. Numerous test sections were evaluated to arrive at the optimum stone placement procedure to minimize damage to the cloth. A specially designed stone placement machine which causes practically no damage to the cloth is currently in use. This device utilizes a cable drawn skip pan, supported on a crawler mounted truss bridge spanning the slope. Stone is fed by an end loader to a movable hopper which feeds the skip pan. Other placement methods have resulted in numerous filter cloth tears and punctures. The use of pins to hold the cloth resulted in more tears than when the cloth was placed loosely. Clogging of the openings in the cloth by silty fines also occured, resulting in a groundwater rise of several feet in the protected slope.

Le tissu filtrant tissé s'emploie au lieu de matériau granulaire conventionel de litage sous les pierres qui protègent les terrassements du partage des eaux du Tennessee-Tombigbee Waterway dans le nord-est du Mississippi aux Etats-Unis. Ces pentes de 27° et de 18°, avec des sables limoneux fins sous-jacents, doivent être protégées des vagues de 0,9 m créées par la circulation. De nombreuses sections d'essai ont été évaluées afin de trouver le meilleur procédé de placement des pierres pour minimiser les dommages au tissu. Un appareil a été spécialement conçu pour le placement des pierres qui ne produit presque aucun dommage au tissu actuellement employé. Cet appareil comprend un monte-charge à benne tiré par câble, soutenu par un pont qui enjambe la pente et qui est monté sur un véhicule à chenilles. Une chargeuse alimente les pierres par une trémie mobile qui alimente la benne. D'autres méthodes de placement ont abouti à de nombreuses déchirures et piqûres du tissu. L'emploi des fiches pour tenir le tissu a donné lieu à plus de déchirures que lorsque le tissu a été posé en liberté. Il y a eu aussi du bouchage des ouvertures du tissu par des particules limoneuses fines, ce qui a fait monter de plusieurs pieds le niveau piézométrique de la pente protégée.

INTRODUCTION

The Tennessee-Tombigbee Waterway is a 373 km (232 mile) long navigation canal linking the Tennessee River system with the port of Mobile and the Gulf of Mexico. The waterway leaves Pickwick Lake on the Tennessee River near where the states of Alabama, Mississippi, and Tennessee join, and proceeds south, through the topographic divide between the Tennessee and Tombigbee River basins, to the already navigable Black Warrior River which flows to Mobile Bay. A 2.74m deep (nine foot), 85.3m (280 foot) minimum width, ice-free, slack water route will be provided. A total lift of 103.9m (341 feet) between Demopolis, Alabama and the Tennessee River will be accomplished by a series of ten locks, the highest of these being the 25.6m (84 foot) lift at Bay Springs, the northernmost navigation structure.

Above Bay Springs Lock is the 43.45 km (27 mile) long Divide Cut section of the waterway, which approaches an excavation depth of 55m (180 feet) at the topographic divide. It is in the Divide Cut where geotextiles are being utilized under stone slope protection and to control side slope seepage.

GEOLOGY

The Divide Cut lies entirely within Tishomingo County, Mississippi, and is on the eastern edge of the Mississippi embayment. (Fig. 1) It is cut through mostly unconsolidated sands, silts, clays, and gravels of Cretaceous age. The general dip of the beds toward the Mississippi River gives rise to artesian groundwater conditions in the area of the Divide Cut. (Fig. 2) The groundwater, combined with the very erodible fine sands and silty fine sands of the Cretaceous Eutaw formation dictate that slope protection be provided to prevent piping from the slopes during construction and to resist tow-generated wave action in the completed waterway.

DIVIDE CUT DESIGN

The route of the Divide Cut follows Yellow Creek from Pickwick Lake on the north, to the topographic divide, then south along the alignment of Mackey's Creek. Much of this route was occupied by a railroad prior to construction. Side slopes for the excavation vary from 1V on 2H in cuts less than 18.29m (60 feet) high, to 1V on 3H in the maximum cut areas. (Fig. 3) Conventional stability factors of safety were lowered during the design to allow the excavation quantity to be reduced from 126 million cubic meters (165 million cubic yards) to 115 million cubic meters (150 million cubic yards). Because of the artesian groundwater conditions, deep wells were utilized on both sides of most of the canal during construction. Permanent relief wells on 152.4m (500 foot) spacing provide artesian pressure control for 9.3 km (5.8 miles) of the deepest cut.

The excavated materials are disposed of in specially designed areas adjacent to the waterway. The erodiability of the soils is evident not only in the disposal areas, but also in the waterway slopes. Much of the material is in the silty fine sand range (Fig. 4).

The design has encompassed measures to control surface water as well as groundwater during and after construction.

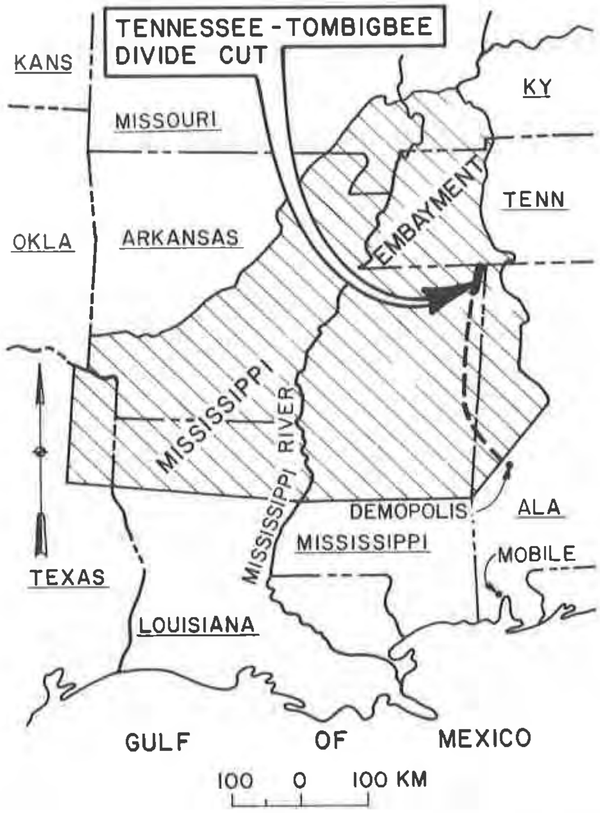

Fig. 1 Project Location and Extent of Upper Mississippi Embayment

SLOPE PROTECTION DESIGN

One important aspect is to protect the erodible slopes from waves generated by waterway traffic. The lower reach of the slope is protected from waves to 0.91m (3 feet) in height by 0.46m (18 inches) of dumped limestone riprap having a maximum size from 158.8 to 181.4 kg (350 to 400 pounds). The riprap generally averages 40.8 kg (90 pounds) and has less than 15 percent smaller than 6.8 kg (15 pounds).

Initial designs included a 0.23m (nine inch) sand and gravel bedding layer under the riprap; however, value engineering studies showed that filter fabric would be more economical in this application. A controlling factor was that the 0.23m (nine inches) occupied by the bedding layer would not have to be provided, thus reducing the entire excavation width by 0.23m (nine inches) for its full depth on both sides of the waterway.

Clay layers in the excavation slopes cause horizontal discharge of groundwater accompanied by piping and erosion of the associated sands. This condition is persistent immediately above the slope that is protected by the riprap system. Here, filter fabric covered with a 15.2 cm (six inch) thick layer of 7.6 cm (three inch) maximum size crushed stone is used to control the seepage. (Fig. 3)

Because of the fine grained nature of the soils to be protected, a reasonably small opening was specified for the cloth (EOS-70). This was later increased to EOS 40 when clogging of the cloth and pressure buildup behind the cloth were noted. Table 1 gives the specifications for the fabric used on the waterway. A total of 2,147,179 square meters (2,568,000 square yards) of filter cloth is involved in this project.

Details of anchoring the fabric at the top and at the base of the slope proved to be critical. Surface water concentrations caused washouts of the fine cohesionless materials from below the cloth, causing tears in the cloth and ultimately failures in the slope protection. An example is shown in Figure 5. This damage was minimized by the details shown in Figure 6 for anchorage at the top of the slope.

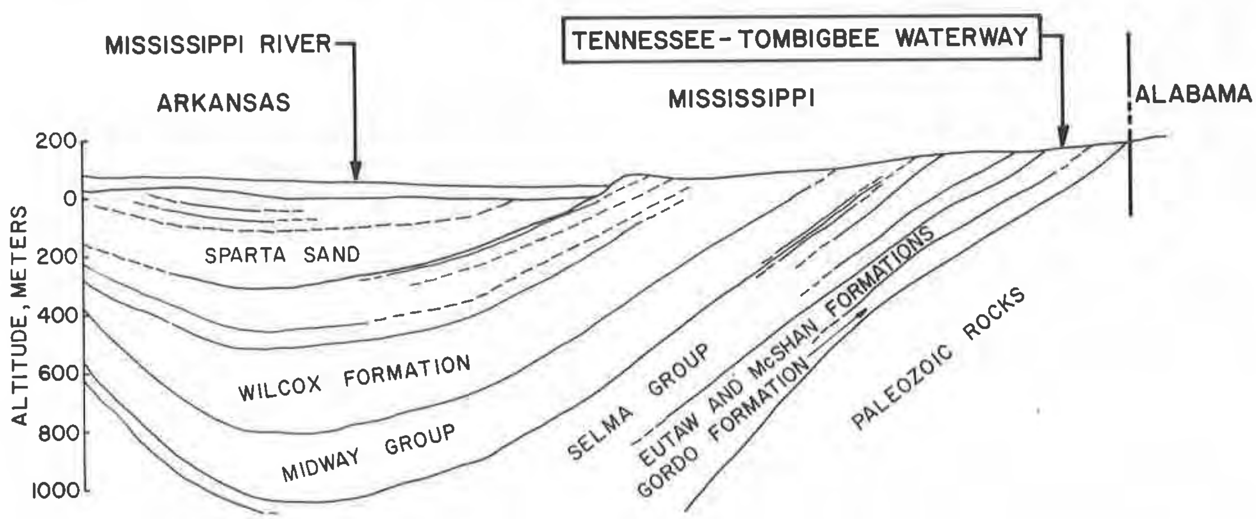

Fig. 2 Geologic Cross Section of Mississippi Embayment Looking North

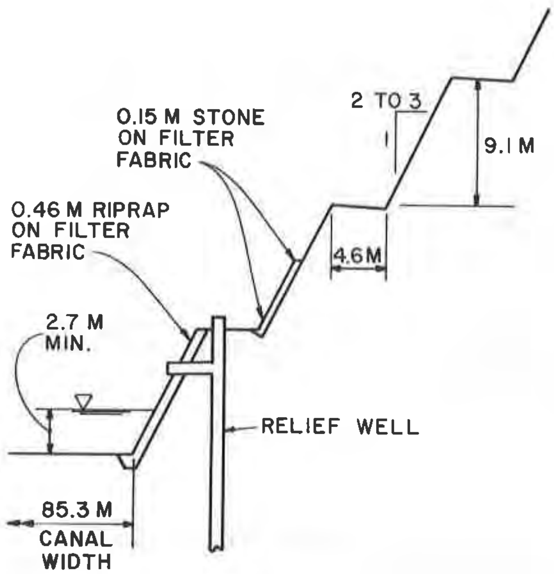

Fig. 3 Typical Divide Cut Cross Section

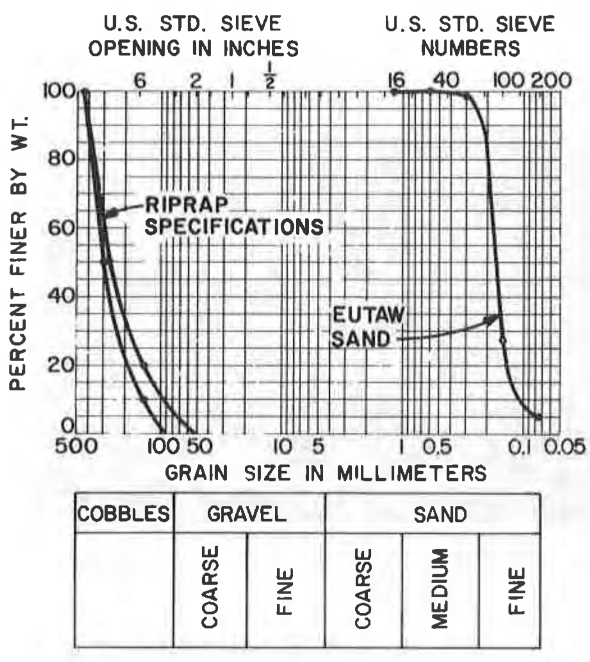

Fig. 4 Typical Gradation Curve for Eutaw Sand

SLOPE PROTECTION CONSTRUCTION

Initial construction procedures included placing the cloth on newly excavated slopes with seams vertical. The adjacent pieces were either sewn together or overlapped by .46 meters (1.5 feet). The cloth was pinned to the slopes. Stone placement was mainly by large backhoe, but some was placed with draglines, clamshells and orangepeels. The surface of the stone was dressed with a hydraulically operated telescoping boom backhoe to present a smooth appearance. Early placements on the 1V on 2H slopes showed that the filter fabric could not withstand the stone placement without tearing, if it was pinned to the slope. The pinning was discontinued and the fabric allowed to move somewhat downslope as the stone was placed. This increased the amount of cloth but eliminated tears because of pinning. Routine test plots 3.05m x 3.05m (10 x 10 feet) used to grade the stone and inspect the cloth revealed that tears were still occurring in the cloth. At other locations tears resulted in loss of the protected material and ultimately failure of the slope protection. The test sections were expanded and a rather general condition of tears in the cloth was revealed. (Fig. 7)

Fig. 5 Typical Slope Protection Failure Related to Tears in Filter Fabric

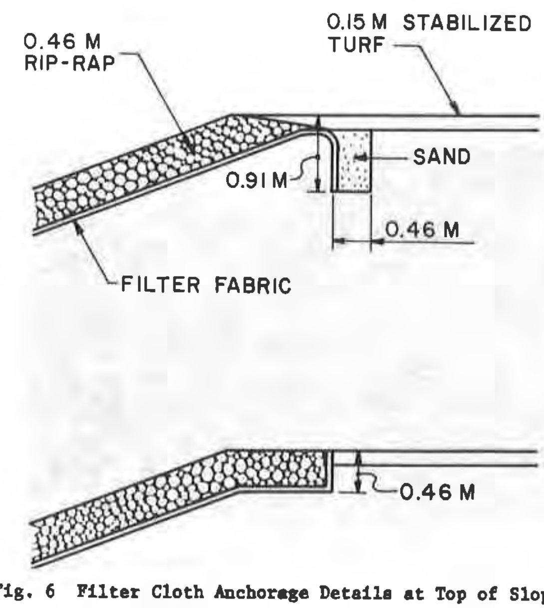

Fig. 6 Filter Cloth Anchorage Details at Top of Slope

Table 1 - Filter Fabric Physical Requirements

Physical Property	Test Procedure	**Acceptable Test Results
Tensile Strength + (unaged fabric)	ASTM D 1682 Grab Test Method using 6.45 cm^2 (1 square inch) jaws and a travel rate of 30.48 cm (12 inches) per minute.	90.72 kg (200 pound) minimum in any principal direction.
Puncture Strength + (unaged fabric)	ASTM D 751 Tension Testing Machine with Ring Clamp; steel ball replaced with a 0.79 cm (5/16 inch) diameter solid steel cylinder centered with a hemispherical tip within the ring clamp.	36.29 kg (80 pound) minimum.
Abrasion Resistance	ASTM D 1682 as above, after abraded as in ASTM D 1175 Rotary Platform, Double Head Method; rubber-based abrasive wheels equal to CS-17 "Calibrase" by Taber Instrument Co; 1 kilogram (2.20 pound) load per wheel; 1000 revolutions.	24.95 kg (55 pound) minimum in any principal direction.
EOS	Standard Glass Beads	70 - 100
Open Area	Percent Open Area of Total Area	5 - 10

**Acceptable test results may be reduced 50 percent for fabric to be used in drainage trenches beneath concrete slabs or to be cushioned from rock placement by a layer of sand or by no drop height placement.

+Unaged fabric is defined as fabric in the condition received from the manufacturer or distributor.

Fig. 7 Typical Test Section Showing Tears and Punctures

Fig. 8 Crushed Stone Blanket Placed Between Riprap and Filter Cloth

Some test sections were done using sand and crushed stone over the fabric. (Fig. 8) This worked well but could greatly increase the cost. Surface dressing with equipment was replaced by limited hand dressing, and a special machine was devised by the contractor to place the rock without damaging the filter fabric. The machine is a crawler-mounted truss 44.2m (145 feet) long that traverses the entire area upon which stone is to be placed. (Fig. 9) The machine is powered by a 400 HP diesel engine and places stone at 244,940 to 272,155 kg (270 to 300 tons) per hour. Each pass of the machine places a 1.83m (6 foot) width of stone from bottom to top on the slope at a 20° angle. The machine is fed by a large front-end loader which dumps rock into a traveling hopper which in turn feeds a traveling vibratory feeder with a deflector plate that gently places the rock upon the fabric. (Fig. 10 thru 12)

Test sections where rock was removed from top to bottom of the slope revealed that there were essentially no tears in the cloth beneath rock placed by the machine. (Fig. 13)

Clogging of the fabric was noted in several instances. At one location, it was necessary to remove a section of finished slope protection. Piezometers in the sand behind the slope dropped 0.61 to 0.91m (2 to 3 feet) when the riprap and filter fabric were removed.

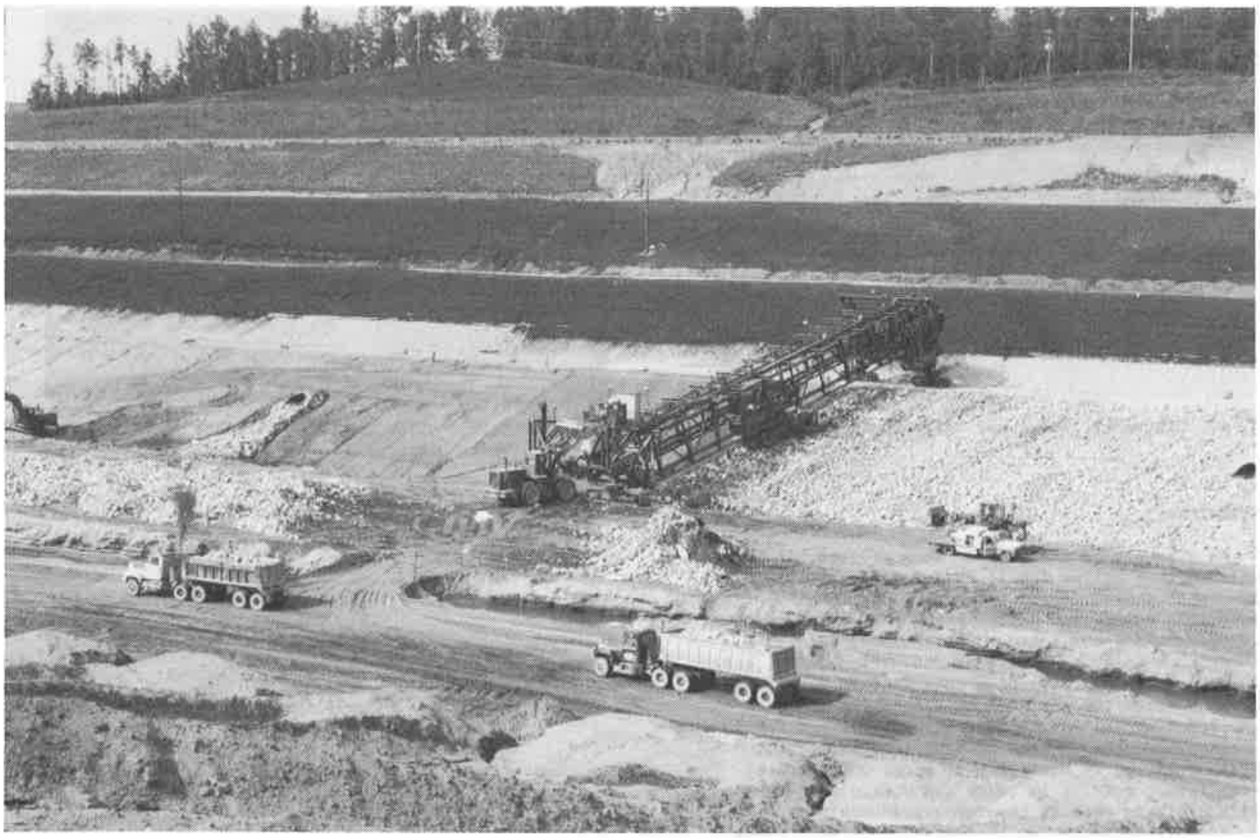

Fig. 9 General View of Riprap Machine in Operation

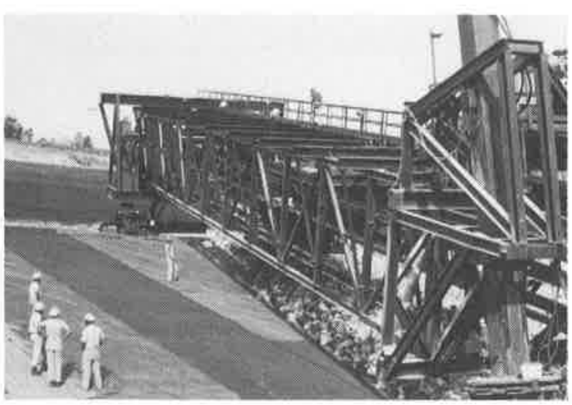

Fig. 10 Closeup of Riprap Machine

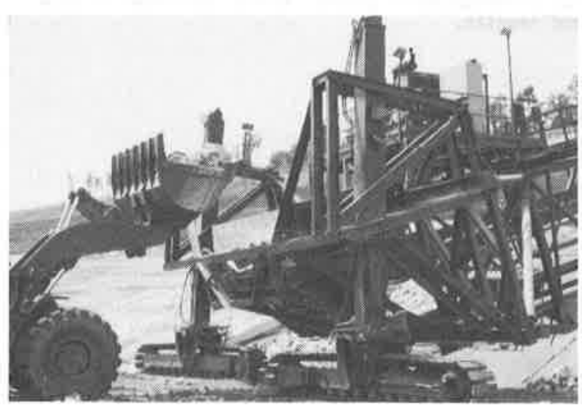

Fig. 11 Riprap Machine Fed by Front End Loader

Fig. 12 Vibratory Feeder and Deflector Plate

Fig. 13 Test Section Showing Condition of Filter Fabric Under Riprap Placed by Special Machine

CONCLUSIONS

As a result of the work on this project, it is concluded that filter fabric used under conventionally-placed dumped riprap on slopes underlain by highly erodible soils will have difficulties unless extensive precautions are taken. Conventional placement methods such as skip pans, backhoes, orangepeels, etc. result in tears and punctures that will lead to failure. A cushion layer placed on the fabric prior to each placement will satisfactorily protect the fabric if reasonable care in rock placement is exercised. Surface dressing by equipment of stone underlain by filter fabric results in tears and punctures. On larger projects such as the Tennessee-Tombigbee Divide Cut, special machines can be devised that will place rock directly on filter fabric with minimum damage to the cloth. If cohesionless fines are present in the protected material, groundwater flow will carry the fines into the pores of the filter fabric and cause clogging, which can ultimately lead to failure of the slope protection system. Design and construction considerations must be carefully evaluated in applications of filter fabric to unusual situations and complex soil conditions. Merely assuming that filter fabric can replace a conventional graded material can lead to undesired results.

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Constructive Elements for River Bank Defense Structures Using Woven Geotextiles

Eléments constructifs pour la protection des berges utilisant des géotextiles tissés

The present paper brings forth for the river banks protection reinforcing works a non-conventional solution in which use is made, in the achievement of the protection toe, of gabion-bag type structural elements made of geotextiles and filled with local soil. The advantage of such a solution lies in the fact that unprocessed local materials are used. Economically the selection of this solution is entirely justifiable in the cases when the rockfill has to be transported from long distances. A full-scale test had been performed during the spring of 1980. Since than several high floods have been recorded on that river. The observations concerning the work revealed the fact that a good arrangement and interlocking of the prefabricated elements in the protective toe had been achieved and maintained. During both the construction and the operation the geotextile the gabion-bags had been made of had a satisfactory behaviour comparable to that resulting from the laboratory tests.

L'article présente une solution non traditionnelle pour l'exécution d'un massif de protection, des éléments de construction du type "gabions-sacs" remplis à terre, réalisés par des géotextiles. L'avantage offert par une telle solutions réside en l'utilisation des matériaux locaux. Telles solutions sont justifiées dans les circonstances où les enrochements doivent être transportés aux grandes distances. Une expérimentation à l'échelle naturelle a été exécutée au printemps de l'année 1980. Dès lors on a enrégistré quelques crues sur la rivière. Les observations effectuées sur cet ouvrage ont montré qu'on a réalisé et maintenu un bon rangement et joint par l'interpénétration des éléments préfabriqués en massif. Les géotextiles utilisées comme matériaux de confection pour de "gabions-sacs" ont eu un comportement satisfaisant tant à la mise en oeuvre que pendant l'exploitation, comparable à ceux résultés par les essais de laboratoire.

INTRODUCTION

The use of woven geotextiles in the river banks protection structures is no longer a novelty (1). In the various solutions that had been experimented and applied the technical and economical obvious advantages offered by this new contruction material had been emphasized.

The present paper shows the results of the authors' studies orientated on the utilization of geotextiles in the execution of the prefabricated elements to replace the riprap in the classical protection toes from the river bank protection-reinforcing works (4).

The prefabricated elements have a cylindrical shape and are of the gabion-bag type. The solution has in view their filling with local soil directly on the site.

Thus, for certain cases a new solution is obtained having indices of efficiency superior to those of the traditional ones. This is due to i) the elimination of unavailable or more expensive materials (such as the rock fill, cement, wire net, a.s.o.); ii) the diminution of fuel consumption in the transport of materials; iii) construction facility; iiii) increased productivity

UTILIZABLE GEOTEXTILES

For the achievement of the structural gabion-bag type elements of the geotextile confection material must meet the following conditions;

- durability to physical environmental factors action taking into account the specific conditions determined by the structural elements permanent or temporary immersion;
- chemical and biological unreactiveness;
- adequate breaking, tearing and bursting strength during the earthfilling manipulation, construction and operation;
- adequate permeability to air and water so that under the severe construction circumstances when the gabion-bags are thrown from the bank into water bursting should be avoided and the earthfill quick saturation should be allowed providing a good arrangement and interlocking of the elements in the structure;
- thinness and marked flexibility so that no difficult problems are met during the fabrication.

The tests concerning the behaviour of some fabrics made of polyamide, polyesters and polypropylene performed on samples subjected to artificial ageing under conditions specific to the river banks protection and reinforcing works led to the conclusion that in point of durability, the fabrics made of polypropylene have the best behaviour. As far as the material is concerned the authors had chosen the geotextiles ALFA-Hessian-type fabrics-obtained from fibrilated polypropylene foil yarns.

Due to the yarns roughness this type of fabric manifests a good dimensional stability and unweaving. strength. The fibrilated yarn also gives the fabric a certain elasticity increasing its crushing strength

As a result of the performed tests the fabrics selected for this utilization have been executed in two variants ; ALFA M and ALFA G.(2).As related to denseness they cover the entire granulometric domain of soils of the sand and gravel type that usually can be taken into consideration as filling materials.

Dimensional defining characteristics as well as the mechanical ones of the ALFA geotextiles utilizable as confection materials for the earth filled bag-type gabions are presented in table 1.

TABLE 1 - ALFA Geotextiles Technical Data

Characteristic		Variants ALFA M	ALFA G
Denseness yarns/10 cm(no)	warp	56	100
	weft	55	55
Linear density of yarns λ (tex)	warp	229	268
	weft	229	268
Mass μ (g/m^2)		240	292
Breaking strength F_G (N)	warp	1300	3800
	weft	1440	1950
Breaking elongation ε (%)	warp	16	28
	weft	17	13

GEOTEXTILES CAPACITY TO RETAIN THE GRAINED MATERIAL

One of the main conditions to be observed by the geotextile the prefabricated materials are to be made of is its capacity to satisfactorily retain the earthfill consistuent particles.

The tests to establish the geotextile capacity to retain the grained material had been performed by filtering a soil suspension with two experimental variants :
- under constant hydraulic head H = o,1 m ;
- under more severe conditions supposing the grained material forced passing by suction.

On the basis of the obtained results the authors considered that for the presumed utilisation the geotextiles must meet the following conditions :
- 25% is the maximum percentage of soil particles (d) that 90% pass through (P90) or 10% remain on (R10) the geotextile; this means:

$$d_{25} \geq P_{90} \quad \text{or} \quad d_{25} \geq R_{10}$$

- 25% is the minimum percentage of the soil particles that are 50% retained on (R50) or 50% pass through (P50) the geotextiles; this means :

$$d_{75} \geq R_{50} \quad \text{or} \quad d_{75} \geq P_{50}$$

For the above mentioned conditions the limitation of the ALFA M geotextile granulometric domain of utilisation is presented in fig.1.

The validity of the granulometric utilization domain thus established had been demonstrated on a physical model in an experimental arrangement in which the earth filled gabion-bag type elements had been impelled in a water basin. The geotextile capacity to retain the earthfill particles had been determined for various velocities of movement (values ranging between 1 ... 5 m/s)

The tests had been performed with earthfill corresponding and noncorresponding to the granulometric utilization domain recomended for the respective geotextile. Admitting that the geotextile restrictive effect cannot be absolute a retention capacity of 80% had been considered as satisfactory.

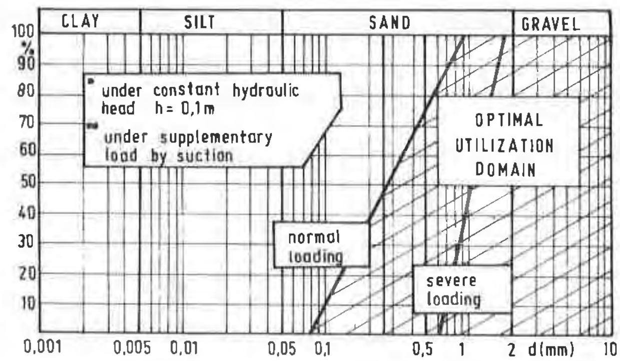


Fig.1 - ALFA M geotextile granulometric domain of efficient utilization

The results obtained for the ALFA M geotextiles (fig.2) led to the following observations :

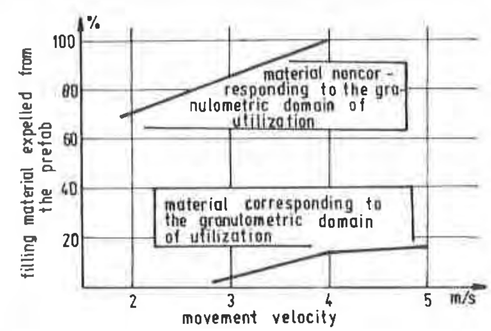


Fig.2 - Checking up of the utilization domain of ALFA M geotextile

- the geotextiles capacity to retain the earthfill depends on the movement velocity ;

- this capacity is clearly differentiated being satisfactory for earth comprised in the granulometric utilization domain and un-satisfactory for not included earth.

From the above presented facts it follows that by determining the geotextile granulometric utilization domain a possibility had been created allowing a correct selection of the filling material for the gabion-bags.

PERMEABILITY TO WATER

Woven geotextiles permeability to water had been determined with a Darcy apparatus (3) under a hydraulic head H = 0.10 m. As for woven geotextiles the hydraulic gradient cannot be determined the testing results have been expressed in apparent velocities (table 2)

TABLE 2 - ALFA Geotextiles Permeability to Water

Geotextile	Apparent velocity (m/sec)
ALFA M	0.154
ALFA G	0.019

CONDITIONS FOR THE RIVER BANKS PROTECTION WORKS ACHIEVEMENT WITH PREFABS OF THE GABION-BAG TYPE

Structural elements used in river banks protection can be made in any location along the river by earth-filling the geotextile bags

The execution consists of two major operations ;

- bags earthfilling;
- river bank protection work implementation.

Studies performed on hydraulic model (4) for the achievement of protection toe from elements of the gabion-bags type replacing the classical solution with rockfill showed that :

- it is necessary and sufficient that the replacing structural elements should have the same weight as the stone blocks used for the rockfill protection toe;
- the gabion-bags dimensions should not render their placing difficult;
- prefabs can be achieved as individual elements, bounded by two or by three;
- in comparison with the classical soulution of riprap the gabion-bags diminish the roughness coefficient increasing the local velocities by 10-13%.

Observing the conditions imposed by the elements stability in the protection toe the prefabs weight should also be taken into consideration so that their placing does not become too difficult. Due to this fact the double-linked prefabs are preferable as they can more easily be manipulated with any type of crane.

The gabion-bags dimensional elements should also guarantee a corresponding interlocking of elements in the protection toe. The performed studies showed that a good interlocking of these elements is attained at a 70% - 80% filling.

The geotextile specific consumption is determined by the elements diameter and length. The possibility of the prefabs dimensional optimization consequently results.

The execution is recomended to be performed during low level periods of the river as in the case of the traditional rockfill protection toe contruction.

NATURAL SCALE EXPERIMENTATION

The experiment aimed at achieving a protection toe of piling gabion-bags included in the usual works of protection and reinforcing on a regulated river section.

The protection toe placed on a fascine work is 1.55 m high and has a 1.30 m crestwidth and a 1:1.5 inclination of slope the geotextile used for the achievement of the gabion-bags being ALFA M.

The prefabricated elements have been used in one or linked by two, each in two dimensional variants (table 3 - case 1 and 2).

Bags dimensions have been chosen taking into consideration the site actual placing possibilities.

For filling the bags earth from the bank and river ballast had been used, the operation being performed on the bank in the site neighbourhood.

The geotextile of tubular shape, previously sewn had been sectioned in accordance with the bags length.

TABLE 3 - Technical Data Regarding the Gabion-bags Dimensional Characteristics and Works Efficiency Indices

Characteristic		Experimental variants: simple bags	Experimental variants: double-linked bags
Bag diameter (m)	case 1	0.4	0.6
	case 2	0.4	0.6
Bag length (m)	case 1	1.20	1.50
	case 2	2.00	2.00
Geotextile consumption (linear m/bag)	case 1	2.00	2.60
	case 2	2.80	3.10
Number of bags per cubic meter of structure	case 1	6.00	2.25
	case 2	3.50	1.50
Geotextile consumption per cubic meter of structure (m^2)	case 1	14.4	10.5
	case 2	11.9	8.4
Average time necessary for the complete achievement of a prefab (minutes)		3	4
Average time necessary for the prefabs placing (min)		5	4
Specific productivity obtained with 1 crane and 2 workers (m^3 of work/hour)		cca.3	

The operations necessary in the achievement of prefabs consisted in the bags earth filling and their consecutive coupling at the bottom, at the center (for double-linked bags) and at the top.

The filled bags had been stored on the river bank in a position favourable to their further manipulation with the crane (photo 1).

The placing had been achieved by linking several bags with a cable attached to the crane (photo 2) workers'intervention being moderate. Special care had been taken in laying the last layer of prefabs so that the pile of bags in the protection toe should observe the work dimensional elements.

The observations made during the implementation allow the following remarks :

- the geotextile had a good resistance to the severe mechanical strains to which it was subjected during the prefabs lifting and launching;
- launching the prefabs from the bank with the crane achieves a good arrangement and interlocking of elements in the protective toe (photo 3):
- the productivity obtained in achieving the works is satisfactory and comparable to that obtained from classical works (table 2).

RESULTS

The full-scale experimentation had been performed during the spring of 1980. Since then several high floods had been registered on the river.

The observations made in time concerning the works showed that :

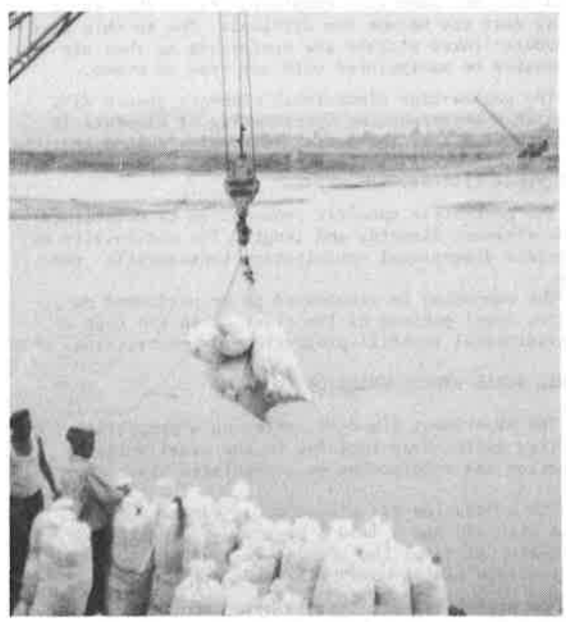
- a good arrangement and interlocking of elements in the work had been performed;

- the bank experimental protection had a good behaviour at high-water levels and for high flow velocities:

- the protection structure underwent no important settling in time and no geometric modification to damage the bank protection;

- the geotextile used as confectioning material for the prefabricated elements had a good behaviour comparable to the laboratory tests results.

The experimental works are still under permanent observation in order to provide data on the behaviour in operation conditions.



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The Expansion of the Belgian Zeebrugge Harbour in the Sea and the Use of the Woven Geotextiles

L'expansion du port de zeebrugge en mer et l'emploi des géotextiles tissés

The paper describes the historical need for a good deep-water harbour on the Belgian Coast, capable of handling a large turnover of high-tonnage vessels.
The multi-billion Belgian francs programme to expand Zeebrugge harbour is described.
In particular, the paper describes the construction principles of the outer break-waters and the integral roll of strong woven geotextiles in the base construction of willow fascine mattresses used for toe scour erosion prevention on the seaward side of the harbour breakwaters.

Le texte décrit le besoin historique pour un port de grande profondeur sur la côte belge en mesure de réceptionner les cargaisons des navires de grande taille.
Le programme d'expansion du port de Zeebrugge explique ce projet dont le coût s'évalue à plusieurs milliards de francs belges.
Le texte met surtout en évidence les principes de construction des grandes jetées extérieures et le rôle intégral des géotextiles de haute tenacité dans l'assemblage des tapis de mer, employés contre l'érosion des jetées.

1.- INTRODUCTION.

1.1.- <u>Historical background</u>.

In 1881, King Leopold II of Belgium declared : "We need a well-equipped harbour on our Belgian Coast, with the facility for ships of all tonnage to enter the harbour at any time of the tide. This can be achieved by the use of modern science and new engineering techniques.

After lengthy discussions, the Belgian Parliament decided in 1895 to build a new coastal sea-harbour close to the city of Brugge, and to be know as Zeebrugge. King Leopold II inaugurated the new harbour complex on the 23rd July 1907. See figure 1.

1.2.- <u>Site location</u>.

The Belgian coast generally trends in a NE to SW direction bordering the North Sea, and Zeebrugge is an industrial harbour centrally situated amongst a number of Belgian holiday resorts.

All the beaches are sandy, with low gradients and are subject to the scouring action of waves and tidal currents. The average sea depth in the near vicinity of the Belgian Coast is relatively shallow, being about 8 metres below the official low water level. However, the entrance channels of the Western Scheldt ports and Zeebrugge have been dredged to 15 metres below low water level and are also linked to the north/south shipping lanes of the English Channel. See figure 1.

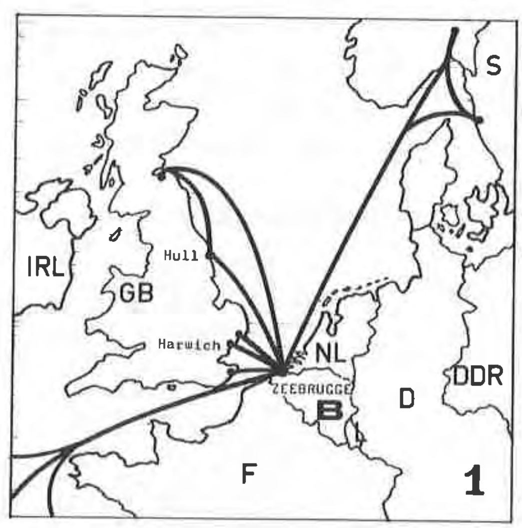


Figure 2.- Site location Zeebrugge in Belgium + Western Scheldt Estuary.

At Zeebrugge the tidal period is typically 12.25 hours for a full cycle, with a tidal range of some 3.7 m . Normal high and low tide are 4.20 m and 0.50 m respectively above the official low water mark (Z-level). At both high and low water the currents are at their strongest. At the old harbour mouth, the maximum surface tidal currents range from 1.6 m/s at spring tide to 0.90 m/s at neap tide. This area of the coast also experiences longfetch waves of considerable height under normal bad weather conditions, and can suffer from exposure to extremely severe seas during exceptional storm conditions. Typical bad weather and storm occurred in 1980 : the height of the waves was 3.50 m for a Beaufort scale of 8 and sometimes 9. In some conditions (spring acting together with a NW-wind at high water) wave height can increase to 6m. Even within the new breakwaters, model tests indicate that internally refracted wave heights may be expected to be 1.40 m to 1.90 m high.

2.- DESCRIPTION OF THE HARBOUR DEVELOPMENT SCHEME.

2.1.- Planned Expansion Programme.

As the old Zeebrugge harbour lay-out and the depth of the outer harbour area proved to be inadequate to receive the ever-increasing number of larger ships, the Belgian Government decided in 1970 to expand the port of Zeebrugge by building new outer harbour breakwaters and in-harbour docking and service facilities. In 1976, the work was placed with a specially formed consortium called "Zeebouw-Zeezand" on the basis of a "frame" contract mutually agreed with the Belgian Government. This project is still under construction and costing many billions of Belgian Francs, comprises :

a) the construction of a major sea-lock for ships of 125.000 tonnes deadweight, linking the new outer harbour and the old inner harbour.

b) the seaward construction of a tidal yet protected outer harbour having a surface area of some 1200 ha (2.965 acres).-This is achieved by the construction of two rubblemound breakwaters on both sides of a dredged access channel. The two breakwaters extend into the sea more than 1750 m. (1913 yards) beyond the existing mole and comprise :

i) the eastern outer breakwater, 2.970 m (3.248 yards) long;
ii) the western outer breakwater, 4.400 m (4.811 yards) long.

c) the construction and equipping of a new inner port with a docking space of approx. 1000 ha (2471 acres) with a minimum waterdepth of 17.50 m (19.10 yards) below low water level.

2.2.- Sea bed soil conditions.

The nature of the sea bed in Zeebrugge area is very complex owing to the heterogeneous composition and unpredictable bedding of the various soils present. In order to evaluate accurately the foundation conditions, a comprehensive general soil investigation programme was initiated in September 1976 and is still being continued to provide updated information for construction purposes.
Owing to the extremely soft nature of the soils revealed by the site investigation programme, it was decided that the breakwaters could not be constructed directly on to the sea bed. Therefore a 150 m wide trench was dredged down to firm soil along the base line of both breakwaters. The trenches were refilled with good quality sea sand and gravel and, where necessary, the dumped sand was densified by vibration to form a firm foundation zone for the construction of the breakwaters.

2.3.- The construction of the outer harbour and the main breakwaters.

The main breakwaters have been designed with several different cross-sections, depending upon the distance out to sea and differing from east to west.

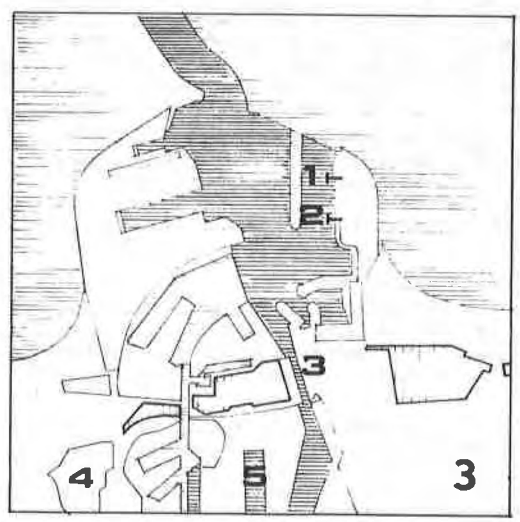


Figure 3. - Lay-out of the new project.

2.3. - The root section of the Eastern Breakwater.

This starts on the beach and is partly constructed in shallow water. On the seaward side, a 150 m wide sand core forms the first part of the eastern breakwater.
In order to prevent erosion taking place at the toe of the breakwater slope, a geotextile willow-fascine mattress was designed and specified to be laid on the sea floor in advance of the breakwater construction. Owing to the proximity to the shore, a lighter-gauge woven geotextile was specified for this section, since extremely arduous conditions were not expected. See figure 4.

2.3.2.- The root section of the western breakwater.

This section was to be built immediately adjacent to a sandy beach used for ammenity purposes by adjoining holiday resorts. Consequently, considerable emphasis was placed on the aesthetic appearance of the root section of the breakwater at this point. The cross-section for this first 500 m out from the sea-front comprises a core of sand, brick debris and quarry run overlaid by larger quarry material varying from 200 to 300 kg which was in turn overlaid by a smooth concrete facing revetment at a slope of 26°. See figure 5.

2.3.3.- The outer Harbour Breakwaters.

In order to protect against toe scour, a large wave breaking carpet was specified to be constructed towards the seaward end of both breakwaters wherever the sea bed was between 2.50 m and 7 m below low water level. The scour protection mattress was designed to be geotextile based, constructed primarily of willow fascine and overlaid by quarrystone layers extending right up to 3000 à 6000 kg rocks of the toe slopes. The outer armour protecting the core has a slope of 34° and on top of this the head of the breakwater is filled with heavy quarry stones weighing between 1000 and 3000 kg. The outsides of the breakwaters are covered with concrete cube armour units weighing 30.000 kg with a concrete density of 2.300 kg/m3. The crest elevation is 13 m above low water level.

The seaward section of the eastern breakwater beyond the root section is approx. 2000 m long. The western outer breakwater will be 4.400 m. long and will be sand-filled internally to form the construction background for two major docks. See figure 6.

3.- THE ROLE OF THE WOVEN GEOTEXTILES IN THE DESIGN AND CONSTRUCTION OF THE BREAKWATERS.

3.1. - The functions of the geotextile.

3.1.1.- Separation.

Anyone who has walked along a beach barefoot and has stood on the sand in moving water, will have experienced the fact that his feet will sink down and become slowly covered with sand. The same happens to stones and even large quarry blocks if laid directly upon the sand surface without a separation layer. The reason is simple : when water is forced around a solid object, its velocity is necessarily increased and it therefore picks up particles preferentially in the vicinity of the body in question. As particles are removed from the base of the obstructing body, then instability results and settlement takes place.
Since the advent of geotextiles, several wave breaking causeways have been constructed on sandy beaches with woven geotextiles used as separation layers. These have already improved to be effective, even after severe storms have attacked the overlying layers of quarrystone.

In the case of large sized breakwaters constructed far out to sea, it was considered almost impossible to create a solid base for the construction without the use of a separation sheet assembled with willow-fascines. The main role of the geotextile in this application is to separate two different solid materials, viz. the layerof quarry blocks from the weaker sand foundation layer. Of course, the fabric must be strong enough not to tear under the weight and point loads generated by the overlying layer of quarry stones.

3.1.2. - Filtration.

Since both wave action and tidal action will doubtless generate hydraulic pressure gradients within the sand foundation layer, it is reasonable to assume that water ingress from time to time takes place outward from the newly constructed sea bed into the overlying boulder layer. In this instance, the geotextile fabric will also perform a filtering function and will withhold the sand particles preventing them from being washed upwards and picked up by any current action. Without a geotextile, the erosion of the foundation sand beneath the quarrystone layer would rapidly result in the collapse of the causeway toe zone.

3.2.- Design specifications.

3.2.1.- Geotextile specification.

"The fabric will be assembled in the form of a willow-fascine mattress; it will be used as a separation sheet and must be strong enough to resist :
a) the pulling out to sea (consequently needing a higher tensile strength lengthwise);
b) tensile stretching between two pontoons before the sinking operation;
c) the dumping of two layers of quarry rocks, with the first layer comprising rocks of 20 to 80 kg. at a placing density of 400 kg/m2 and the second layer comprising quarry rocks of 80 to 300 kg at a placing density of 600 kg/m2. The overall thickness of the rock layer will be about 1 m.

Under tensile stress, the elongation of the fabric may not be too high, in order that the fabric should not become open and loose its effective sand tightness. On the other hand, there must be sufficient elongation to allow the fabric to follow any irregularites on the sand foundation layer.

Water permeability is not critical in this application."

On the basis of the above comments, and the detailed specifications in the tender, a geotextile was produced with the following characteristics :

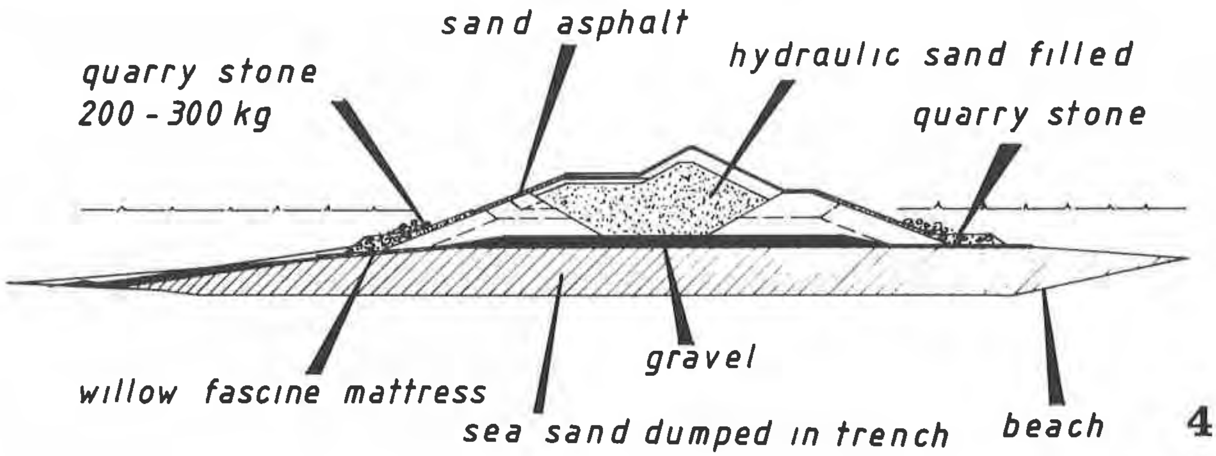


Figure 4. - Cross section of the eastern breakwater.

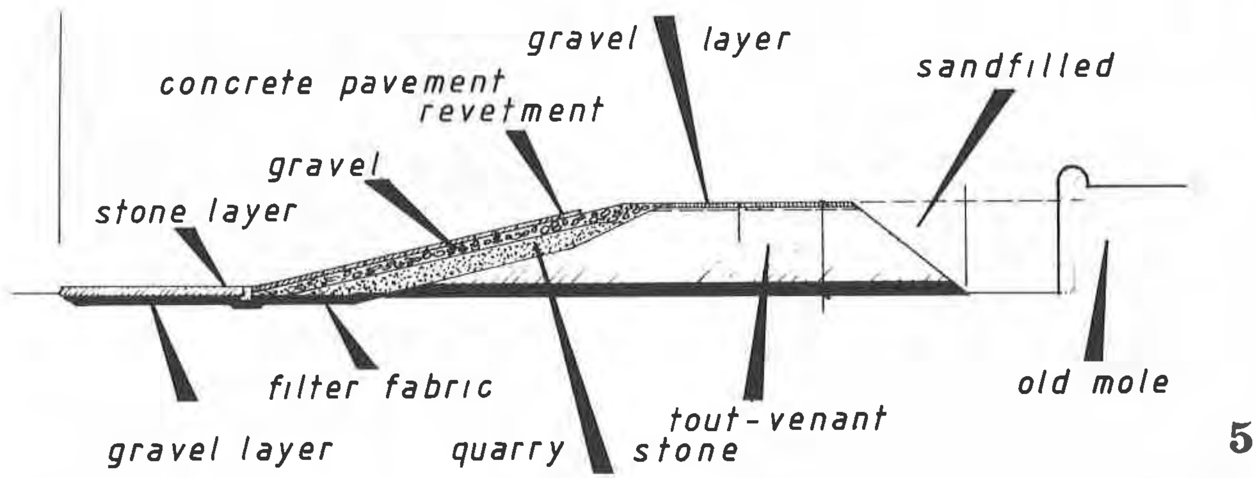


Figure 5. - Cross section of the western breakwater.

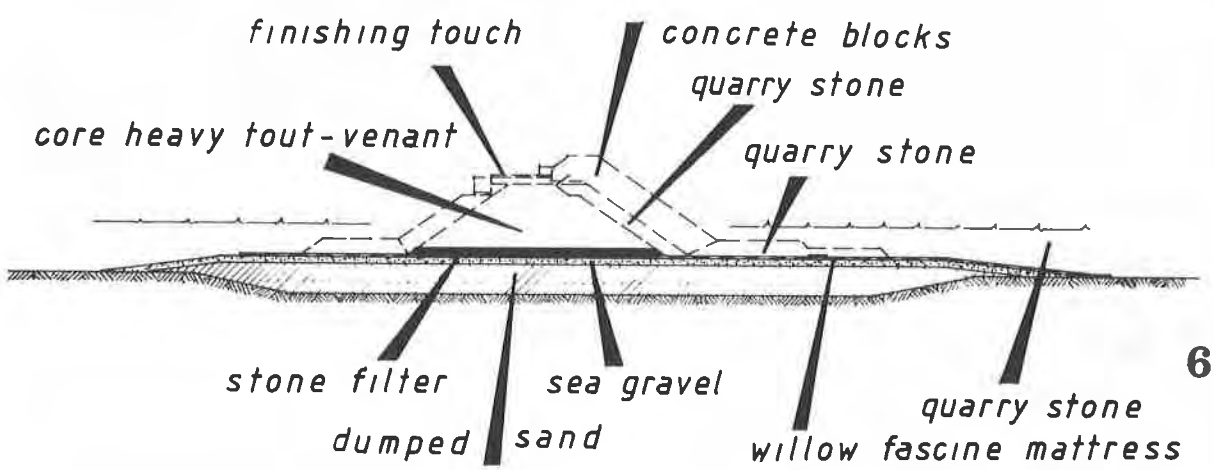


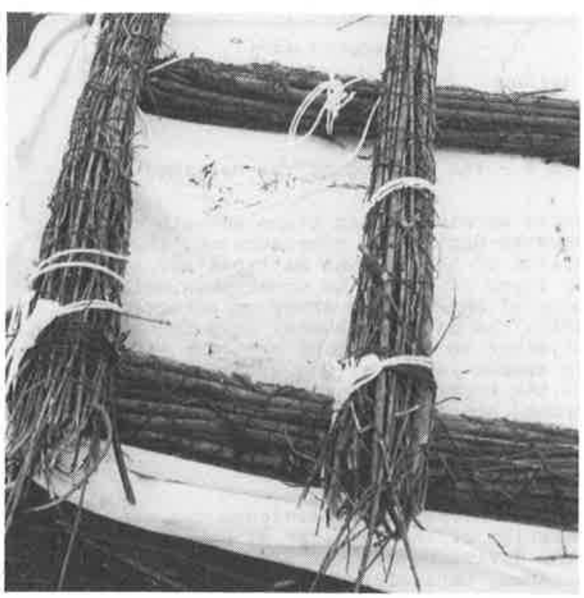
Figure 6. - Cross section of the main outer breakwaters.

1.- a plain weave woven textile,
2.- raw material polyester,
3.- yarn type multi-filament,
4.- tensile strength warp approx. 196 kN/m and weft approx. 98 kN/m.
5.- weight of the fabric 550 gr/m2.
6.- elongation at break in both directions minimum 15 %,
7.- Pore size distribution O_{90} = 80 microns. (specified as max. 130 microns requirement).

3.2.2.- Willow fascine mattress specifications.

" The fascines are to be assembled with twigs of willow, oak, birch, ash, maple or hazel. The diameter of the individual bundles is to be approx. 13 cm. The length must be related to the length and the width of the proposed mattress (viz. 50 m x 30m respect.).
The fascines are to be tied to the geotextile fabric at distances of 1 m x 1 m square and will generally be in three layers depending on the detailed construction of the breakwater.

See photographs 1 and 2.



Photograph 1 - First layer of fascines.

Photograph 2 - Fascines and bush bundles.

3.3. - The function of the willow-fascine mattress.

Along nearly the entire construction length of the breakwater, toe scour protection is effected by the presence of the geotextile fabric separation layer. Because of the difficulty which would be experienced in attempting to lay that fabric under exposed marine conditions, it was decided that it would be essential to combine the fabric with a willow fascine mattress. Overall, there were three main reasons for this decision :

a) the willow fascines, fixed to the fabric in the form of a large grid, would keep the fabric stretched so that it could be sunk to form the separation carpet in exactly the design location

b) between the fascine bundles which form the grid, bushes of willow twigs are placed to fill the intervening spaces to a thickness of about 0.50 m. The purpose of the willow twigs is to break the fall of the dumped quarry blocks and prevent damage of the geotextile fabric. Also, the weight of the overlying rock layer causes the lower rocks to bed into the willow bushes and provides substantially enhanced lateral stability to the rock layer.

c) the willow-fascines and bush bundles, being naturally buoyant, assist in the floating process of towing the assembled mattress out to sea to the required sinking position.

3.4.- The make-up of a willow fascine mattress.

The mattresses are assembled in the immediate vicinity of the breakwater construction. They are assembled on the sloping beach within the tidal range so that at high water the mattress may be easily floated by a tug boat, and may be pulled into the water for towing to its required sinking position.

In accordance with the design specifications, each fabric must have the following dimensions :

width 30 m length 55 m
total weight approx. 900 kg.
total area approx. 1650 m2.

For easier manipulation on site, three widths of fabric were sewn together in the weaving mill into a single 15 m wide roll, each piece therefore weighing approx. 450 kg. Two 15 m. wide pieces were sewn together on site to form the final 30 m x55 m long assembly. A first row of willow fascine bundles is fixed lengthwise to the fabric. The spaces between this single direction fascine assembly are then filled up with bush bundles. A second layer of willow fascine bundles is then fixed on top of the first layer and over the earlier bush bundles but now at right angles (widthways) to the first layer. The openings between these widthways running fascines are now filled up with bush bundles also. Finally, a third layer of fascines and bush bundles are laid and fastened to the lower layers, bringing the total thickness of the mattress up to approx. 0.50 m and the total weight of the mattress to approx. 50 tonnes.

3.5. - The procedure for sinking the mattress.

Mattress sinking is undertaken only under calm sea conditions when the tidal currents are at their **weakest** . Prior to the fascine mattress being tugged to sea, two specially-adapted pontoons are moored over the designed sinking site.

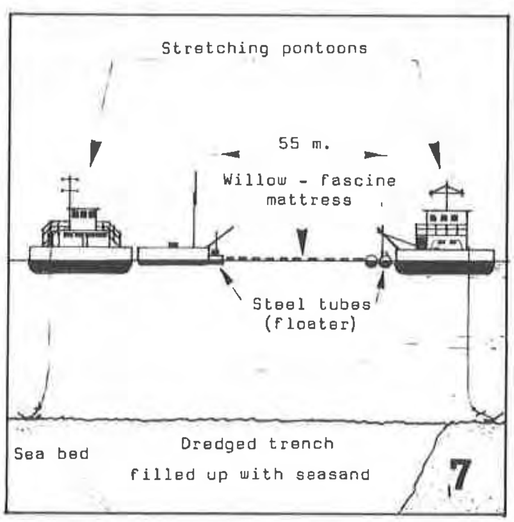


Figure 7- Moored pontoons + stretched willow-fascine mattress.

The fascine mattress is then attached to the two pontoons and stretched out between them. At each end of the fascine mattress, adjacent to the pontoons, are hollow steel tubes, and the steel tube on one side of the mattress is lowered by retaining wires on to the sea bed. A specially -adapted barge with a sloping and vibratable upper deck, loaded with appropriate quarry stones, is brought into position over the fascine mattress. The rocks are then gradually tipped over the side of the barge as it is pulled over the mattress towards the second pontoon. The barge returns several times and continues dumping quarry blocks until a thick layer has been achieved. The first layer of stone ranges from 20 to 80 kg stone weight and is placed at a rate of 400 kg/m2. Heavier stones 80 to 300 kg are placed on top in a second layer at a density of 600 kg/m2 . See figure 8.

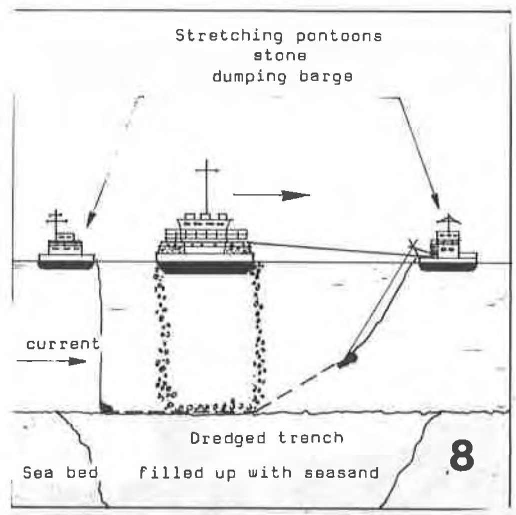


Figure 8 - The sinking of the mattress.

To avoid erosion taking place beneath the breakwater during its construction, the con- struction of the fascine mattress and its overlying layer of stones is undertaken well in advance of the main causeway construction. Finally, the main breakwater core is constructed adjacent to the fascine mattress and the outer armour layers of the breakwater overlap on to the rock and fascine mattress structure described above.

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A Study of Soil-Filled Synthetic Fabric "Pillows" for Erosion Protection

Une étude de sacs en matière synthetic remplis de sable pour la protection contre l'érosion

A medium-scale laboratory model study was performed on a new synthetic fabric designed to be used as slope protection in the form of soil-filled "sand pillows". The study was done to determine the relationships that affect the stability of the pillows and to determine basic design criteria for the protection system. Laboratory tests were conducted to determine the soil retention characteristics of the fabric. A field study of the slope protection system was also conducted to determine the erosion resistance of the system when placed on an actual embankment.

The results of the model investigation were consolidated and analyzed to develop design criteria for the individual pillows. The field evaluation of the slope protection system was performed by monitoring the erosion of unprotected sections of the upstream slope and comparing the results with protected sections of the same embankment.

The results of the laboratory and field evaluation investigation indicate that the sand pillow method offers excellent slope protection. Design equations also were developed.

Une étude sur modèle a demi-échelle a été conduite sur une nouvelle matière synthétique destinée à être utilisée sous forme de sacs remplis de sable pour la protection de talus. Le but de l'étude a été d'etablir les relations gouvernant la stabilité des sacs de sable ainsi que les critères de base pour le projet de système de protection en l'occurence: la retention de la matiere synthetique et la résistance à l'érosion de l'ensemble du système. le premier critère a été etudié au labortoire. quand au second un éxistant remblai a été utilisé. la resistance à l'érosion a été determinée par comparaison des mesures de l'érosion de sections non protegées situées côté amont du remblai aux mesures obtenus pour des sections protegées du même remblai.

Les critères de base, pour un choix adequat de sacs de sable à utiliser pour la protection, ont été degagés de l'analyse des resultats obtenus de l'étude sur modèle.

L'étude conduite au laboratoire aussi bien que sur le site a permis d'établir les équations regissant la stabilité des sacs de sable et a montré l'excellente protection de talus offerte par cette méthode.

I. INTRODUCTION

The purpose of this study was to analyze a new synthetic fabric (Acrilan, which is a registered trademark of Monsanto) in the form of sand pillows to be used as slope protection for irrigation dams. The project was performed in two phases. The first phase of the program consisted of an actual installation and evaluation of a full scale protection system. Phase two of the program was a medium-scale model study of the pillows in a controlled laboratory wave tank equipped with a wave generating device.

1. Field Study. The object of the field study was to determine the feasibility of placing the bags (pillows) with equipment and resources available to a typical farmer, to monitor the erosion protection offered by the system and to compare the performance of various irrigation dams near the project site with the study site. The durability and soil retention characteristics of the fabric were also evaluated.

A full-scale field installation was used to evaluate the placement requirements and erosion protection offered by the sand pillow system. Inspection of several irrigation dams near the project site provided information regarding the relative performance of the system. Laboratory tests were performed to evaluate the resistance of the fabric to field exposure and to evaluate the resistance of the fabric to field exposure and to evaluate the soil retention characteristics of the fabric.

The slope protection system was installed on an operational irrigation reservoir using farm equipment available on a typical Missouri farm to handle the soil material and to fill the bags. Tractors were used to transport the filled bags to the upstream slope of the dam and to provide the electric power require to operate a hand-held sewing machine used to close the bags.

The effectiveness of the slope protection system has been evaluated by monitoring the performance of test sections on the dam. The fabric slope protection system was installed on three sections of the dam subjected to wave erosion. The erosion experienced on sections without protection was monitored at three locations adjacent to the sand pillow sections. Fourteen profiles within these unprotected sections were surveyed at regular intervals.

2. Model Study. The objectives of the model study were to determine the effect of certain wave parameters on the stability of the sand pillow slope protection system, to monitor the erosion resistance offered by the sand pillows and to formulate design criteria for the system based on soil and local wind conditions. The main wave parameters investigated were wave height and wave period. The effects of scaling (viscosity effect), wave steepness, relative wave runup on the sand pillows and embankment slope angle were also investigated. A proposed design equation for determining the average weight of the individual sand pillows to be used on full scale projects was established based on the data obtained in this study. The design weight of the filled bag is a function of wave height, embankment slope angle, soil properties, wave period and scaling effects.

II. FABRIC TESTING PROGRAM

A. Resistance To Deterioration

1. Testing Method. Tensile strength tests were performed to evaluate the resistance of the fabric to deterioration. The tests were performed after intervals of exposure to the environment. The strength of the fabric after exposure was compared to the results of tensile tests on the original fabric. Two types of field exposure were tested. Samples of the fabric were obtained from bags that remain exposed to direct sunlight and weathering agents. Additional tests were performed on fabric samples from bags that remained submerged at the normal reservoir level. These tensile tests were performed according to the ASTM 25.4 mm (1 inch) Raveled Strip Method. The test method is described in ASTM D1682-64 (reapproved 1970), "Breaking Load and Elongation of Textile Fabrics".

2. Results. These test results indicate that the fabric has effectively resisted deterioration during the first year of exposure. Statistical analysis (t-test) indicates that with less than a 5 percent chance of error the tensile strength tests were performed on samples with the same mean.

B. Soil Retention Characteristics of the Fabric

1. Immersion Tests. To determine the ability of the synthetic fabric to retain its fill material when submerged repeatedly in water, a dunking apparatus was designed and built. This test was designed to simulate the constant wave action attack upon prototype sand pillows.

a. Testing Material. Two types of soil were used for fill material during the dunking tests. The first material tested was a fine grained soil named Mexico clay. The second type of soil tested was obtained from near Quincy, Illinois. This soil, herein referred to as Quincy loess, is a brown, silty-loessial soil that is quite common in that Illinois region.

2. Soil Loss Results. The measurement of the amount of soil lost through the fabric by continued wave action was achieved by the use of a laboratory immersion device described by Stephenson, et. al. (1980) (1). Twenty tests were conducted on the silty clay from the field evaluation embankment and thirty tests were conducted on loessial soil from Illinois. From the results of these tests, the following conclusions were reached:

1. The amount of soil lost through the fabric is directly proportional to the number of immersions. The longer the duration of wave pounding, the larger the percentage of soil that will be lost through the fabric. Both types of soil suffered excessive losses when subject to long term wave action.
2. The soil lost during immersion tests was through the fabric itself and not through the sand pillow seams.
3. The amount of vibration of the sand bag during testing greatly affected the results of the Quincy loess samples, with the greatest amount of vibration causing the soil lost to be in excess of 80%. Vibration appeared to have little effect on the Mexico clay samples.
4. The loess samples began losing fines through the fabric at particles sizes less than 0.06 mm, while the clay samples began losing fines at particles sizes less than 0.21 mm. This difference could possibly be attributed to aggregation of the clay particles when initially wet sieved through a No. 200 sieve and the subsequent separation of the particles during the immersion tests. Because of the different results, no conclusion could be made as to the maximum particle size that can be filtered through the fabric.

III. FIELD TESTING OF SAND PILLOWS FOR EROSION CONTROL

A. Introduction

A field evaluation of the acrylic sand pillow method of slope protection has been conducted on an irrigation dam near Mexico, Missouri. The slope protection system was installed on the upstream slope of an irrigation dam where wave erosion was anticipated. The field study deals with the feasibility of filling, closing and placing the bags with equipment and resources available to a typical farmer. The performance of this slope protection system has been monitored by means of monitoring cross sections established on the dam. The relative performance of the test site was compared to the performance of various slope protection systems at several irrigation dams near the project site.

B. Field Evaluation of the Slope Protection System

1. Introduction. The dam, chosen for this field investigation, is the pump-in irrigation dam owned by Mr. Jack Freyer of Laddonia, Missouri. The dam is located 19.31 km (12 miles) east of Mexico, Missouri, approximately .80 km (1/2 mile) west of U.S. Highway 54. The dam, completed in December of 1976, experienced very little wave erosion during 1977. The dam is L-shaped with a total crest length of 408.43m (1340 feet) and reaches a maximum height of 7.62 m (25 feet) in the north-east corner of the reservoir.

2. Installation of the Slope Protection System. It is essential for the economical utilization of this slope protection system that a farmer be able to fill, close and place the bags with equipment and resources available to him. Elaborate equipment capable of filling and closing 500 bags per hour, as used on the Corps of Engineers Red River levee project, is not practical for a small irrigation dam. After some study, it was determined that a fertilizer cart used to spread dry fertilizer on farm fields could be modified to fill the bags (1).

3. In-Situ Monitoring of the Slope Protection System. The performance of the sand pillow protection system was monitored by means of surveying test sections on the dam. Two sections, 2.86 m (75 feet) in length, are located on the north and east upstream slopes of the dam. The third section, 13.72 (45 feet) in length, is located in the northeast corner of the dam. These sections received direct wave attack from the prevailing wind directions south, southwest and west. The sections contain evenly spaced profiles that were surveyed at regular intervals.

4. Results.

a. Field Performance on the Slope Protection System. The performance of the acrylic fabric slope protection system was analyzed to provide specific data on the properties and requirements of this slope protection method. This data will facilitate the evaluation of this slope protection method as an economical solution to erosion control on additional irrigation dams.

b. Feasibility of Installation of Farm Irrigation Dams. The ability to efficiently install this method of slope protection on a typical farm irrigation reservoir was a major aspect of this study. The time and labor required to fill even the relatively small number of bags required for a typical irrigation dam would be prohibitive without mechanical assistance. Several alternate systems were considered as possible methods of filling the bags. An auger or conveyor belt was one possible means of providing a flow of fill material to the bags.

The system was easily installed by personnel unfamiliar with the procedure. Simple instructions and supervision enabled high school students to place the bags upon the slope in a uniform and consistent manner. This produced a competent slope protection system without weak areas. Weak portions of typical slope protection systems have led to the progressive failure of such systems. The sand pillow system conforms easily to the shape of the dam. Vertical or horizontal curves present no installation difficulties.

c. Erosion Resistance of the Slope Protection System. The ability of the sand pillow system to resist the erosive forces of wind generated waves has been evaluated. The test sections were monitored from January, 1978, until June, 1979, for a total of eighteen months. During this period the protected sections of the dam have remained unaltered. Figure 1 shows a view of the slope protection system during the summer of 1978. The sand pillow slope protection system did settle and adjust slightly as various natural forces acted upon the slope. The sand pillow system remained intact during these minor disruptions and provided complete protection for the slope.

In contrast to the protected sections, the unprotected sections showed acute erosion. The rate and amount of erosion depends greatly on the time of year. The greatest erosion occurred between the months of April and May for both 1978 and 1979 which is usually the time of the highest wind velocities and duration for the year as well as the highest water levels in the reservoir. The amount of slope deterioration over a 12 month period from April, 1978, to April, 1979, varied from about 0.3 m to 0.6 m (1 to 2 feet).

IV. MODEL STUDY INVESTIGATIONS OF SAND PILLOWS FOR SLOPE PROTECTION

A. General Model Theory

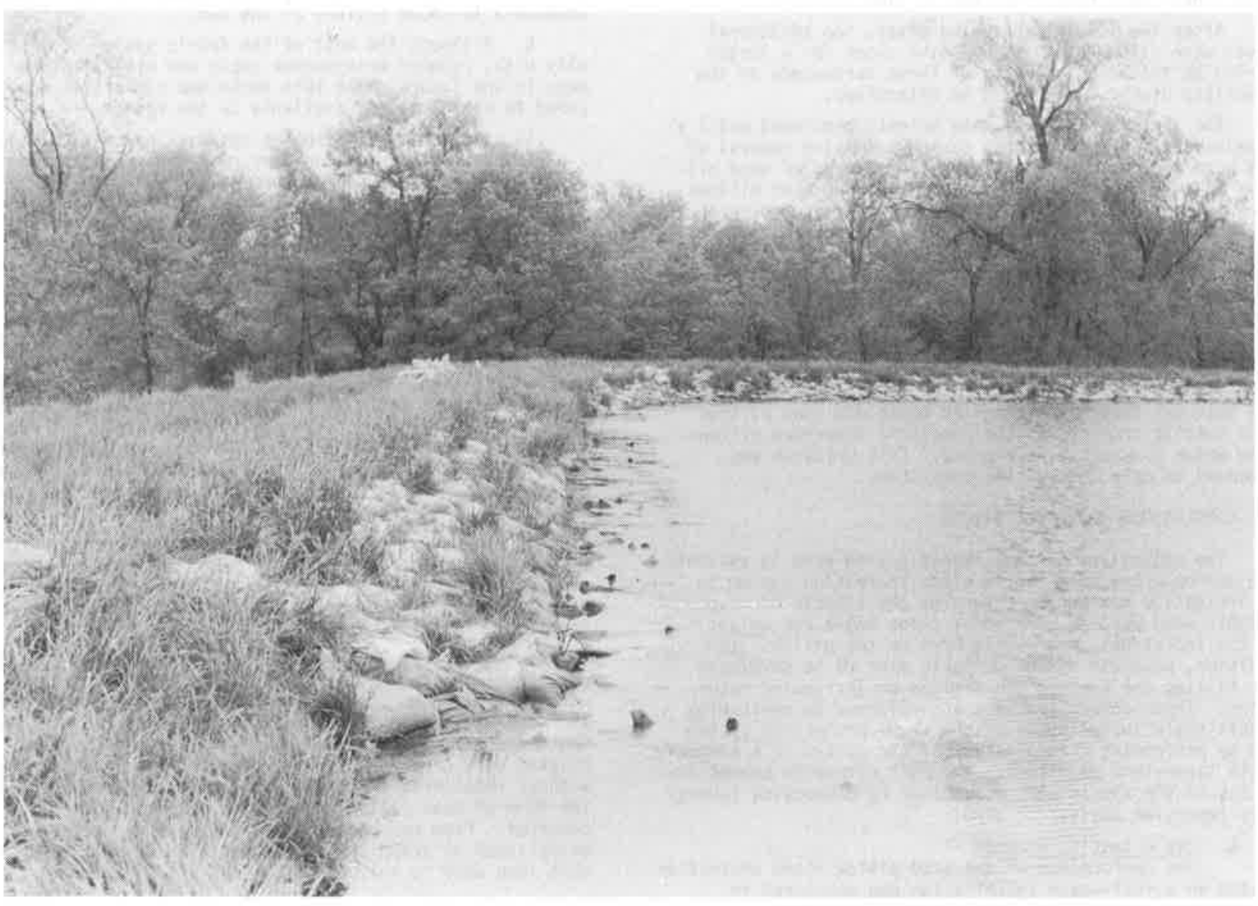
To evaluate the effectiveness of the various slope protection methods, and to determine under what set of conditions these methods were most applicable, the need for controlled testing became obvious. Controlled conditions were achieved by constructing scaled models of the slope and protection materials and simulating natural wind waves by forcing wind over water in closed flumes (2) or by producing waves generated by a paddle (3,4,5,6,7).

B. Testing Equipment and Procedures

1. Introduction. A three phase testing program was incorporated for this study. The three programs included a wave tank model investigation where scaled replicas of the sand pillows were tested, a laboratory testing program where the properties of the synthetic material themselves were tested, and a field evaluation of the synthetic material for use as a slope protection agent was studied on a prototype earth embankment constructed just east of Mexico, Missouri. The prototype embankment site is located in a region where a fine-grained silty clay exists on a relatively flat terrain. These three testing programs were conducted concurrently such that any correlation that exists between them could be quickly and efficiently analyzed.

2. Model Testing Program.

a. Wave Tank and Wave Generator Description. A model study of the protection offered by acrylic sand pillows for use as a slope protection method was conducted in a laboratory wave tank located in the Engineering Research Laboratories building at the University of Missouri-Rolla. This wave tank is a rectangular, steel-framed tank that measures 9.75 m (32 ft) long by 1.07 m (3.5 ft) wide by 0.61 m (2 ft) deep. The wave generation device is the



flap-type design consisting of a reinforced plexiglass paddle driven by a 1/2 horsepower variable speed drive motor.

b. Testing Material. The soil used for the model slope and for the sand pillows' fill material was obtained from an earth embankment located in northern Missouri. This embankment is part of a field evaluation of the sand pillows discussed previously.

The model sand pillows for the slope protection system were made of acrylic fabric that is sold under the trade name of Acrilan. These model bags were made 127 mm (5 in) long by 88.9 mm (3.5 in) wide which represents an area scaling of 1:6 of the prototype sand pillows. In terms of median weights of the slope protection pillows, the model scale was approximately 1:200 for average weight sand pillows and for similar water contents in model and prototype soil.

c. Testing Procedure. The field soil was compacted on a moveable platform that could be oriented in varying slopes with respect to the horizontal. This was done to allow testing at different slope angles. The model soil pillows were then filled with soil and placed on the slope.

After completion of the slope construction, the motor speed and paddle eccentricity were adjusted to develop the desired wave height. Testing began by supplying "seasoning waves", i.e., small waves that might cause some settlement and adjustment but no removal of the armor units. If none of the sand pillows were displaced after approximately 2000 waves had passed over them, the motor speed and paddle eccentricity were modified to obtain the next higher wave height and the testing continued.

After the initial seasoning waves, the additional waves were allowed to pass over the slope for a longer period so that the influence of those parameters on the stability of the slope could be determined.

The increasing of the wave heights continued until a wave height was reached that caused excessive removal of the sand pillows down the slope. The number of sand pillows that had been moved was recorded. The sand pillows were removed from the slope and the slope was surveyed and photographed. A moisture content was obtained from the submerged sand pillows and several of the pillows that were directly attacked by the waves were used in the determination of the percentage of soil lost through the synthetic material due to wave pounding.

Judgment was required to decide when to change the wave height and to determine how much damange was "excessive" such that the test should be terminated. Generally, the test was terminated after at least six sand pillows were totally removed and the remaining submerged pillows were being constantly reoriented. This criteria was intended to make the testing consistent.

V. CONCLUSIONS AND APPLICATIONS

The objectives of this investigation were to evaluate the performances of a fabric slope protection system on an irrigation dam and to determine the effects of wave height, wave period, embankment slope angle and weight of the individual armor units have on the pillow. In addition, possible design criteria were to be developed for filling and placing the pillows on irrigation reservoirs. These objectives were accomplished by monitoring a full-scale installation of the slope protection system and by performing a model study of the system in a medium-scale laboratory wave tank. The soil retention characteristics of the fabric were determined by conducting laboratory immersion tests.

A. Field Testing Program

The performance of the sand pillow slope protection system on a full-scale installation was monitored in northern Missouri. By comparison of results of unprotected sections of an irrigation embankment with sections protected by Acrilan sand pillows fill with on-site material, the effectiveness of the system was evaluated. The monitoring program resulted in the following conclusions:

1. The fabric pillow erosion control system was rapidly and easily placed on a Missouri irrigation dam using resources available to the farmer. Weather conditions during placement were not a factor and specialized personnel or training were not required.

2. The sand pillow protection system offered excellent protection after 18 months of wave and weather exposure particularly when compared to other erosion protection methods on other similar dams. Some of the constantly submerged pillows lost much of their fill material after a few months but when replaced with pillows filled with sand, the protection system performed quite adequately.

3. The amount of wave erosion that occurs on an earth embankment varies with the time of the year. The greatest amount of erosion appears to take place between the months of April and May, when wind velocities and reservoir levels are generally highest.

4. The rate of erosion of silty clay embankments accelerates, to a certain degree, with time. Wave erosion tends to produce unstable vertical cuts on the embankment slope which slumps or slides easily when undercut by constant wave action. With time, the vertical cuts become larger, the instability increases, and the rate of failure increases with relatively small amounts of undercutting necessary to cause failure of the cut.

5. Although the cost of the fabric system is initially high, reduced maintenance costs and other factors may, in the future, make this technique attractive compared to other methods available to the farmer.

In addition, the following observations can be made:

1. Several irrigation dams near the project site were inspected. The performance of slope protection methods on these dams was evaluated. The natural rock materials utilized on these dams lacked adequate size material to resist normal wave action. The slope protection systems utilized also lacked the filter gradation necessary to retain the clay material used to construct the embankment.

2. The erosion observed on Missouri irrigation reservoirs indicates that wind generated waves from any direction will significantly deteriorate typical silty clay dams. The short fetch lengths associated with irrigation reservoirs restrict the size of wave generated, but require only short duration winds to produce the maximum possible wave height. The slopes of these irrigation dams are under maximum attack more frequently due to waves generated by winds from any direction.

3. The acrylic fabric has not been adversely affected by exposure to the environment. After one year of exposure the tensile strength of samples taken from the field installation has remained equivalent to the values obtained from the original fabric.

B. Model Test Program

Sixteen models were tested in the 9.75 m (32 foot) wave tank with the primarily variables in each test being the weight of filled sand bags, the slope of the embankment, and the height of the generated waves. The wave heights were progressively increased during the test without rebuilding the model until extensive damage, in the form of sand pillow removal and displacement, had occurred. From each test a zero-damage wave height was established by graphical techniques. These wave heights were then used to calculate the sand pillow stability.

The various relationships that affect the weight of the individual sand pillows were experimentally determined. From this study the following conclusions were reached:

1. The amount of soil eroded from between the sand pillows was slight compared to the erosion that occurred on an unprotected slope. The erosion that did occur between the bags is a direct function of embankment slope and the weight of the pillow. The amount of erosion decreases as the slope angle (α) flattens and the weight of the armor increases, to a limiting value of area damage. At area damage (D_A) equals 0.27, the amount of damage appears independent of slope or weight.

2. For medium-scale models with a Reynolds number greater than 3.3×10^4, stability is independent of scale. However, from previous works it is shown that small- and medium-scale model tests are less stable than large-scale tests, therefore the need for correcting these tests to large-scale equivalence exists. A correction factor was developed based on the Reynolds number from each test.

3. Stability of the sand pillows is a direct function of wave period. Stability decreases as wave period increases and at a wave steepness (H_o/L_o) less than 0.14, a constant minimum value of stability is obtained that is relatively independent of wave period.

4. Stability increases as the embankment slope becomes flatter. The simple mathematical expression used to define this effect is:

$$N''_{ZD} = 0.605(\cot \alpha)^{0.249}. \qquad (1)$$

This equation is valid for sand pillows only and is limited to slopes in the range of $1.5 \leq \cot \alpha \leq 6.0$.

5. A design equation for calculating individual sand pillow weights was established based on data gathered from this study. The equation,

$$W_{50} = \frac{H_{ZD}{}^3 \gamma_{sat}}{(N''_{ZD})^3 (G_s - 1)} \qquad (2)$$

was based on a previously developed stability equation (8). The stability constants, N''_{ZD}, are unique to this study and sand pillows, and can be obtained for a given slope from Equation 1. The usefulness of this equation was shown by comparing the actual average weight of the sand pillows from a prototype field installation to the calculated weight using Equation 2 based on the prototype soil and wave characteristics. Equation 2 will give conservative values for prototype design because N''_{ZD} is a limiting value with respect to wave period. Details of the analysis are presented by Stephenson, et. al. (1980) (1).

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NOMENCLATURE

α = angle of embankment slope from horizontal

γ_{sat} = saturated unit weight of soil in bag

G_s = specific gravity of soil solids

H_o = wave height

H_{ZD} = zero damage deepwater wave height

L_o = wave length

N''_{ZD} = minimum zero-damage stability number after correct

W_{50} = weight of median size armor units

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Geotextiles Against Wind Erosion

Les géotextiles dans la lutte contre l'érosion éolienne

Le désert avance en moyenne de 5 km/an dans les régions arides : l'érosion éolienne est une des causes de la désertification, danger signalé à plusieurs reprises par l'O.N.U. dans le cadre du grand problème de la faim dans le monde. Par conséquent toute méthode de protection contre le sable visera finalement à le fixer pour réimplanter ensuite une végétation ... La méthode décrite consiste à utiliser deux produits textiles :

- une grille tissé utilisée comme brise-vent ;
- une mèche de filaments continus parallèles, écartés sur le sol, formant un voile de 5 à 6 g/m² qui fixe le sable recouvert et protège la nouvelle végétation.

C'est là un aspect nouveau des géotextiles quoique cette technique soit utilisée depuis quelques années en Afrique. Seuls subsistent quelques problèmes de prix encore anormalement élevés. Cette communication donne des recommandations utiles pour aborder ce sujet, décrit les caractéristiques des produits et leur pose, comare cette technique avec des procédés concurrents, présente quelques résultats significatifs.

Desert is advancing at an average rate of 5 km per year in arid areas : wind erosion is one of phenomena responsible for desertification, threat often emphasized by the U.N.O. with respect to the great problem of hunger in the world. Therefore any method of protection against sand will have the final purpose of stabilizing it, to reinstall afterwards a new vegetation. The method described in the paper involves the use of two textile products :

- a woven grid used as a wind barrier ;
- a bundle of continuous filaments spread on the ground parallel to each other, forming a thin cover weighing 5 to 6 grams per square meter, stabilizing the sand under it and protecting the new vegetation.

That is a new aspect of geotextiles, although this technique has been used for a few years in Africa. Its only presend limitations are some cost problems which should not be so high. The paper gives useful recommandations to get involved in such a matter, describes the products and their use, compares the method with other techniques, presents some typical results.

C'est pour répondre à l'appel du congrès de Téhéran sur la désertification de février-mars 1975, ainsi qu'à celui de la conférence de Naïrobi (Kenya) en 1977 patronnée par l'O.N.U., que nous avons repris nos études et essais datant de 1964. Ces derniers portaient sur la fixation des sables à l'aide de deux géotextiles :

- l'un est un genre de mèche de textile synthétique écartée sur le sol sous forme d'un réseau très clair ;

- l'autre est une grille souple, pliable, également en textile synthétique se présentant sous la forme d'un tissu tissé très ajouré.

Il s'agit donc d'une méthode de lutte contre l'érosion éolienne, c'est-à-dire contre la progression du désert sur des terres encore en végétation. C'est cette technique que nous allons essayer de développer en commençant par énumérer quelques principes et recommandations importantes qui nous ont servi de règles lors de cette recherche.

I - PRINCIPES ET RECOMMANDATIONS IMPORTANTES

1-1 Ces géotextiles ont pour objectif premier d'éviter le départ ou d'arrêter le transport des particules de sable, plus ou moins fines qui, dans les régions concernées, sont poussées par l'air en déplacement à des vitesses peu élevées la plupart du temps.

1-2 Ceci peut se faire en diminuant sensiblement la vitesse du vent au niveau du sol, c'est-à-dire dans la zone où l'air est en contact direct avec la couche de sable. On abaisse donc cette vitesse au-dessous de 25 km/h en disposant entre sol et vent un obstacle ajouré qui jouera le rôle de brise-vent : soit horizontal dans le cas de la mèche, soit vertical dans le cas de la grille.

1-3 Ces dispositifs sont appelés à protéger de grandes surfaces et comme tout autre système de protection ils devraient théoriquement être mis en place sur le lieu d'origine du sable qui le plus souvent est le rivage marin, mais cela n'est pas toujours possible notamment lorsque la zone à protéger se situe à des centaines de kilomètres de ce lieu d'origine.

1-4 Quel que soit le dispositif artificiel adopté, il faut savoir qu'il n'aura pas une durée d'usage très longue en raison de l'usure normale (U.V., bactéries) ou des dégâts causés par l'homme lui-même, par les animaux ou les intempéries excessives. Cependant les produits synthétiques permettent d'atteindre une durée d'usage suffisante pour que la nature secondée par l'homme ait le temps de prendre le relais.

Il faut donc créer une protection permanente qui ne peut être donnée que par une matière vivante, c'est-à-dire qui se régénère par elle-même. Cette matière vivante adaptée au milieu c'est la végétation.

1-5 Pour réaliser cette protection l'homme dispose de trois moyens complémentaires, le plus souvent indissociables dans un plan de stabilisation.

A - Le recouvrement ou couverture

Traditionnellement on recouvre le sol de branchages mais ils sont rares et coûteux dans les pays intéressés. La tendance est donc de les remplacer par d'autres matériaux, non érodables, bon marché et les textiles synthétiques, sous certaines présentations, sont parmi les solutions les plus efficaces et les plus abordables. Le recouvrement permet donc d'arrêter l'érosion du sol recouvert en freinant le passage de l'air au ras du sol :

c'est un genre de brise-vent horizontal qui est mis en place dans les zones d'érosion, ce qui n'empêche pas que ponctuellement il peut être momentanément enseveli d'où la complémentarité de la barrière.

B - La barrière

C'est un brise-vent vertical qui dans toute formule est approximativement semi-perméable à l'air, ralentissant l'air sur toute sa hauteur de telle façon que les particules de sable en cours de migration se déposent en aval du filet par rapport au vent et un peu en amont en raison d'une légère turbulence. C'est un dispositif efficace mais de protection temporaire. En effet le dépôt de sable augmentant petit à petit, le brise-vent vertical sera submergé, ensablé. Il faudra donc le surélever. Ce système de barrière est donc utilisé :

- pour protéger sa partie aval lors du démarrage d'une nouvelle végétation implantée par l'homme ou parfois même par la nature elle-même ;

- pour protéger une zone d'habitations, une route, etc. ;

- pour réaliser une dune, qui après surélévations successives de cette barrière ou brise-vent, atteindra la hauteur prévue, sera fixée artificiellement et donnera ainsi une protection sur une vaste zone dans sa partie aval.

Cette barrière ou obstacle au vent peut être réalisée de différentes façons :

a) par des branchages d'environ 1m20 de longueur. Ils sont piqués verticalement dans le sol à proximité les uns des autres, leurs intervalles étant à peu près égaux à leur diamètre ;

b) par des branchages entrelacés sur claies ou cadres rectangulaires en bois de 1 m x 3 m, fixés sur des poteaux ;

c) ou par une grille tissée, en fils textiles synthétiques, fixée sur de solides piquets.

L'emploi d'une telle barrière suppose donc toujours un futur amoncellement de sable. En conséquence dès le début de l'opération il faut le prévoir et s'organiser de façon à ce que cette nouvelle dune participe à la protection d'ensemble. Cela demande donc un suivi rigoureux de l'évolution du site.

C - La végététion

A l'état adulte la végétation aura pour but de remplacer définitivement les techniques artificielles qui lui sont absolument indispensables pendant la période de son développement. En effet pour que la jeune végétation implantée par l'homme puisse croître dans ce milieu hostile et aride le plus souvent, il faut lui aider à survivre pendant la période de première croissance. C'est ainsi qu'il faudra aider la plante :

- en procédant à la plantation juste avant la période des pluies ;

- à ne pas être emportée par le vent après déchaussement de ses racines par érosion ;

- à lutter contre la sécheresse au début de sa croissance. Pendant cette période très critique la jeune plante a absolument besoin qu'on aide par arrosages ponctuels ou par irrigation son système radiculaire à traverser la couche de sable sec pour trouver un peu plus profondément le minimum d'humidité nécessaire. Selon les lieux celle-ci se situe à des profondeurs variables : 30 à 40 cm est une observation assez courante ;

- à ne pas être détruite par le "mitraillage" du sable et à ne pas être éventuellement enfouie sous de nouveaux apports de sable, en utilisant l'aide des brise-vent.

La végétation jouera plus ou moins un rôle de couverture ou de barrière, selon les espèces utilisées. Grâce à sa grande adaptation aux sévères conditions du milieu : vents violents, sécheresse, abrasion du sable, parfois embruns salés mais aussi à la vigilance de l'homme, la végétation pourra se réimplanter et peut-être même adoucir certaines données climatologiques. La sélection des espèces est assez bien connue pour permettre d'entreprendre des programmes de rénovation importants mais l'approvisionnement est quelquefois difficile.

1-6 Coût d'ensemble le plus bas possible : pour les raisons essentielles suivantes, le prix de revient de toute solution proposée doit être le plus faible possible :

- très grandes surfaces à traiter : il ne s'agit pas de recouvrir entièrement les déserts mais seulement les zones présentant les caractéristiques climatologiques minima pour la survie de la rare végétation adaptée à ces lieux, c'est-à-dire le plus souvent provenant de régions proches. Cela représente des dizaines, des centaines de milliers d'hectares pour chacun de ces pays d'après l'enquête que nous avons menée dans tous les pays du nord et nord-ouest de l'Afrique ainsi qu'au Proche-Orient.
- Budgets très limités.

2 - PRESENTATION DES PRODUITS : NATURE - RÔLE

2-1 La mèche

Ces filaments de textile synthétique se présentent sous la forme d'une mèche contenue dans un sac plastique. Cette mèche appelée aussi câble dans l'industrie textile est constituée de plusieurs dizaines de milliers de filaments continus, théoriquement parallèles, en matière synthétique acrylique, chacun ayant un diamètre approximatif de 15 à 35 μ selon le type retenu.

Ils vont être écartés sur la surface du sol, toujours maintenus très tendus afin d'obtenir et de conserver une répartition homogène sous l'effet destructeur du vent. Ainsi on forme un réseau de filaments à peine visible de 5 à 6 g/m² soit 50 kg/ha au ras de la surface du sol, suffisamment dense cependant pour ralentir la vitesse du vent de manière à ce que celui-ci n'ait plus assez de force pour soulever les particules de sable.

Lorsque le réseau est en place on obtient une répartition moyenne approximative de 5 à 10 petits filaments par mm.

Il s'agit donc en fait d'un brise-vent horizontal correspondant au principe du recouvrement qui, comme la technique des branchages, permettra au travers de ses mailles le développement normal des plantes, c'est-à-dire le passage de l'air, de la lumière, de l'eau (rosée ou pluie) et de toutes sortes de végétations.

2-2 La grille

2-2-1 Caractéristiques techniques

Cette grille n'est autre qu'un tissu ajouré léger (environ 100 g/m²), souple, peu encombrant et donc très facile d'emploi, celui-ci consistant à la fixer sur de gros piquets de bois. L'ensemble doit pouvoir résister aux plus fortes poussées du vent. Ce géotextile est constitué de fils (assemblage de filaments) de textile synthétique tissés et parfois même tricotés. Les tissés sont particulièrement résistants à la traction et imputrescibles en raison de la nature même des polymères de base qui sont le polyester ou le polyamide. On ne doit pas être cependant très exigeant quant à la tenue à la lumière solaire car cette grille sera progressivement ensablée, ensevelie. En outre, elle doit présenter une certaine rigidité afin qu'elle soit plus facilement manipulable lors de la mise en place. Elle subit donc en fabrication un traitement d'encollage.

Malgré une très faible contexture le sens chaîne doit être le plus résistant car le plus exposé à la force des vents.

La résistance à la rupture est en général de 25 à 30 kg/cm environ, ce qui à première vue paraît exagéré mais est cependant nécessaire pour les raisons de manipulation et de sécurité de l'installation. Cette faible contexture :

- chaîne 3 à 4 fils/ cm 1100 décitex
- trame 1 à 2 fils/ cm " "

est indispensable pour obtenir la perméabilité à l'air qui doit être de 50 % environ.

Le sable se déplaçant en grande partie ($\simeq$ 90 %) sur une faible hauteur d'environ 30 cm, la hauteur du brise-vent, c'est-à-dire la laize livrée à l'utilisateur, sera de 1 mètre environ de hauteur utile et facile à poser.

2-2-2 Fonction

Cette grille en textile synthétique répond au deuxième principe énoncé ci-dessus : celui de la barrière ou obstacle au vent faisant office de brise-vent vertical. Comme tout brise-vent naturel (arbres et autres végétaux) ou artificiel (planches, claies diverses) celui-ci ne doit pas arrêter brutalement le déplacement de l'air : cela aurait pour effet de créer des turbulences aussi néfastes que le vent lui-même. Pour cela ce tissu-grille doit obligatoirement comporter environ 50 % de vide entre ses "mailles". Ce pourcentage est cependant assez relatif car il varie en fonction de l'aérodynamisme (CX) des éléments constituant la grille.

Il s'agit là d'une caractéristique tirée des brise-vent naturels ayant donné à l'expérimentation des résultats satisfaisants.

Ce dispositif va donc abaisser la vitesse de l'air au-dessous de 25 km/h approximativement. Passant au travers de la grille, l'air n'aura plus l'énergie suffisante pour propulser les grains de sable. Ceux-ci vont donc tomber, se déposer sur le sol selon leurs densités et leurs formes. Ils vont former un amoncellement qui s'étendra à la base du brise-vent, pour une petite partie (environ 1 m de large), devant et pour la plus grande partie (5 m, 10 m, 20 m) après le brise-vent par rapport au sens du vent. Bien entendu cette dernière observation n'est valable que dans les cas où le brise-vent est mis en place sur un sol à peu près horizontal.

Cet amoncellement de sable représente le début d'une formation de dune qui va plus ou moins rapidement ensevelir le brise-vent. Il faudra alors le surélever ou plus simplement en rajouter un deuxième par dessus le premier et ainsi de suite jusqu'à une hauteur déterminée à l'avance. Aussi obtiendra-t-on la protection d'une très vaste zone grâce à cette nouvelle dune qui alors sera fixée artificiellement et entretenue régulièrement.

On peut donc procéder selon cette technique à la protection de champs agricoles, de zones d'habitation, de voies de circulation et des zones protégées par le système de mèches pour éviter son ensablement.

3 - METHODES DE POSE

3-1 Pose de la mèche

Succintement on peut dire que l'on procède de la façon suivante :

- repérer le site à recouvrir, l'orientation du vent dominant, les pentes, les voies d'accès et autres facteurs gênants ;

- procéder au quadrillage du terrain par l'implantation des piquets des lignes de départ et des lignes longitudinales en orientant la pose dans le sens du vent dominant ;

- puis fixer sur cinq petits piquets de 50 cm solidement enfoncés dans le sol cinq petites mèches venant de la division approximative de la mèche principale qui doit toujours rester tendue en vue d'éviter la désorganisation et la salissure des filaments avant écartement ;

- ensuite le porteur du sac s'éloigne de la ligne de départ en reculant : ainsi la mèche principale, toujours en tension, sort du sac et les poseurs se tenant à une quinzaine de mètres du porteur fixent les bords du réseau sur des piquets latéraux du même type que les précédents. Ils ont été au préalable positionnés et enfoncés dans le sable à intervalles de 5 à 6 m et se situent sur deux lignes parallèles au sens de pose et éloignées d'une distance égale à la largeur d'écartement du réseau.

La fixation définitive des bords du voile sur les piquets latéraux se fait à l'aide de liens métallo-plastiques.

- au cours de la pose on contrôle et l'on rectifie s'il y a lieu la répartition et la tension des filaments à l'aide du même type de liens ;

- lorsque les poseurs sont arrivés à l'extrémité du site, la mèche est coupée, divisée en cinq petites mèches qui seront fixées, comme au départ, à cinq petits piquets.

- on procède de la même façon pour les bandes suivantes juxtaposées.

3-2 Pose de la grille

3-2-1 Poteaux

Cette grille d'un mètre de large va être mise en position verticale et maintenue ainsi sur de gros piquets en bois. De nature assez grossière, ce sont le plus souvent des branches simplement sciées. Leur diamètre est de 7 à 12 cm, c'est-à-dire suffisamment résistants pour ne pas rompre sous les fortes poussées exercées sur la grille par des vents violents de 100 km/h et plus. La solidité d'ensemble de ce brise-vent dépend donc de plusieurs facteurs :

- résistance à la traction de la grille notamment dans le sens chaîne ;
- résistance à la rupture des poteaux ;
- de la profondeur d'implantation des poteaux (résistance à l'arrachage) ;
- de la longueur des intervalles qui est proportionnelle à la charge ou à la poussée que doit subir chaque poteau.

L'expérience nous a montré que les poteaux décrits ci-après donnent satisfaction en toutes circonstances que ce soit en sable sec ou humide :

- diamètre 7 à 12 cm, en bois relativement dur ;
- longueur 1,70 m dont 1 m en support de grille et 0,70 m de profondeur d'implantation ;
- intervalles 4 m.

3-2-2 Fixation de la grille

Devant ces poteaux solidement implantés et du côté vent dominant on fera passer la grille assez tendue, cette tension la positionnant à son emplacement définitif. Sur ce même côté du poteau (face la plus rectiligne) on applique alors une baguette qui sera clouée en 3 ou 4 points coinçant ainsi la grille entre poteau et baguette. On procède de la même façon pour les poteaux d'extrémités mais avant clouage on enroule la baguette de plusieurs trous de grille. Comme dans tout dispositif employant des grilles textiles ou même des tissus, en tension, il faut que la fixation soit bien répartie sur l'ensemble des fils constitutifs afin qu'il n'y ait pas ruptures successives de fils. C'est la raison d'être de cette fixation par baguettes qui dans le cas de vent opposé au vent dominant va permettre à la grille d'y prendre appui sur toute sa largeur sans risque de déchirures.

3-3 Mise en place de la végétation

Le fléau de l'ensablement sévit dans des pays en voie de développement pour la plupart. Ne disposant donc que de peu de moyens, l'arrosage systématique et par conséquent le semis sont impossible à réaliser. On procède donc à des plantations de jeunes plants d'arbres adaptés au climat. Ils sont plus résistants que le semis mais nécessitent cependant au début un minimum d'arrosage localisé. Cette plantation réalisée juste avant la saison des pluies accroît évidemment les chances de réussite. Comme dans la méthode traditionnelle on procède à la plantation des jeunes sujets avec des intervalles de 2,50 m environ. Ensuite a lieu la pose du réseau de fibres qui recouvrent les plants. Leurs deux ou trois petites branches sont passées manuellement au travers du voile et mises à l'air libre. Un arrosage individuel a lieu immédiatement après.

Lors d'un premier projet F.A.O. Sénégal les résultats ont été spectaculaires. L'objectif a été atteint : le sable recouvert ne bouge plus autour des plantes qui ne sont plus déchaussées et emportées par le vent. 80 à 90 % des plants témoins non protégés par ce réseau textile ont été emportés par le vent alors que sur la zone expérimentale la totalité des plants est restée en place et a eu une forte croissance malgré une situation particulièrement exposée aux vents alizés et au vent du sud. Soulignons ici la complémentarité entre le réseau textile, qui maintient en place sable et végétation et cette dernière qui au début tout au moins contribue à la bonne répartition des filaments sur le sol.

4 - TEMPS DE POSE

4-1 Temps de pose de la mèche

Il varie en fonction d'éléments divers tels que :

- la préparation rationnelle du travail à effectuer : planification, qualité du personnel, approvisionnement du matériel principal et acessoire ;
- l'expérience du personnel ;
- la topographie du site ;
- la longueur totale à couvrir (moins de reprises).

En tenant compte de ces considérations on peut donc donner les approximations suivantes : 2 hectares -par jour de 8 heures de travail effectif - par équipe de 6 hommes.

4-2 Temps de pose de la grille

Contrairement à celle de la mèche, la pose de la grille textile est facile car elle est semblable à celle de toutes les sortes de grillage. Aussi est-elle connue des ouvriers et ne demande pas d'adaptation particulière. Temps moyen (un poteau et sa position de grille) 5 à 7 mn.

5 - METHODES CONCURRENTES - AVANTAGES ET INCONVENIENTS

5-1 Recouvrement

5-1-1 Les branchages

C'est la méthode traditionnelle réalisée à l'aide de branchages étendus sur le sol et disposés en lignes perpendiculaires au vent dominant pourvoyeur de sable. Ces lignes sont plus ou moins denses et éloignées (1 m) entre elles suivant les disponibilités de l'organisme responsable. La consommation est très variable suivant les sources d'information : 30 t/ha nous paraît un chiffre assez raisonnable.

Avantages et inconvénients : les branchages représentent une matière première bon marché, leur coût correspondant parfois simplement à celui de la main d'oeuvre d'exploitation. Par ailleurs en tant que production nationale il n'occasionne pas de sortie de devises. Cependant la lutte la plus urgente contre l'ensablement se situant dans des pays pauvres, au climat très sec et par conséquent où le patrimoine forestier est en péril, l'exploitation irrationnelle du bois sous quelque forme que ce soit est actuellement une calamité. Prenons comme exemple le Soudan : aux environs de Khartoum on ne trouve du bois pour la cuisson des aliments qu'à partir d'une centaine de kilomètres alors qu'il y a 10 ou 15 ans il fallait (quand même) parcourir quelques dix kilomètres.

De plus pour en revenir à des considérations purement économiques on doit signaler pour l'avoir constaté que le lieu de récolte est souvent très éloigné du lieu d'utilisation, d'où des transports longs, coûteux et peu sûrs dans leurs régularité d'approvisionnement : donc coûts de matière élevés, retards de livraisons et non respects des prévisions.

5-1-2 Le paillage

Il est utilisé dans les pays possédant une agriculture céréalière importante. Ce procédé mécanisé consiste à répandre de la paille sur le sable et à l'enfouir partiellement de façon à ce que certains brins de paille se redressent plus ou moins vertivalement en formant de mini brise-vent alors que d'autres à demi enfouis font office de couverture.

La consommation de paille par hectare, que nous avons constatée dans le nord de la France et dans la région de Marseille, est de l'ordre de 7 T/ha. Cependant cette quantité peut doubler car cette première couche est souvent emportée par la tempête : soit 14 T/ha.

Dans l'optique du grand problème mondial de la désertification cette technique n'est guère intéressante et ne peut être retenue pour les raisons suivantes :

- la paille en quantités industrielles est une production de terres riches, provenant de la grande agriculture industrialisée, toutes choses qui n'existent pas dans les pays soumis à l'érosion éolienne ;

- c'est un produit qui, dans le cadre des économies d'énergie, devrait tendre à se raréfier même si la production reste stable. D'après les prévisions la paille aurait un avenir rémunérateur dans d'autres applications telles que les granulés pour l'alimentation du bétail et la production d'énergie nouvelle (Bio-masse) ;

- enfin c'est un produit à faible valeur marchande et il n'est donc pas envisageable de l'exporter.

5-1-3 La projection de bitume

Le bitume a l'avantage d'être bon marché dans les pays producteurs souvent exposés à l'ensablement. Par contre il présente de nombreux inconvénients qui sont bien confirmés par la rareté des réalisations :

- efficacité limitée car il craquelle et peut donc être emporté s'il n'est pas en couche suffisamment épaisse ;

- difficile combinaison avec la mise en place par l'homme d'une végétation ;

- application pratiquement impossible dans les zones éloignées des voies de circulation ;

- son coloris noir est inesthétique et dans les proportions où il est utilisé il n'est pas bio-dégradable. Les quantités nécessaires sont importantes : 10 à 100 m^3/ha.

5-1-4 La pulvérisation de liants chimiques

Elle nécessite 300 à 1 000 kg/ha d'extrait sec, soit plusieurs tonnes par hectare de solution. Son avantage serait peut-être sa simplicité d'application mais il présente des inconvénients suivants :

- coût élevé ;
- croûte de surface après séchage très fragile et ne supportant pas le passage des piétons, d'animaux et encore moins de véhicules ;
- réalisations jamais rencontrées dans nos nombreux contacts.

Pour l'avoir expérimenté nous-mêmes nous savons que cette fragilité est très préjudiciable à une bonne protection du sol car lorsqu'il y a rupture de la croûte le vent s'engouffre par dessous et peu à peu évacue le sable. La cavité de plus en plus grande ainsi formée est "soufflée" par les grands bents. Il y a éclatement de la pellicule sèche de liant puis destruction progressive de celle-ci.

5-2 Barrières

Dans les techniques concurrentes faisant office de barrières on retrouve presque exclusivement le bois et en solution intermédiaire la végétation seule, c'est-à-dire sans l'aide de matériaux divers.

5-2-1 Le bois

Il s'agit pour la plupart des formules de brise-vent confectionnés différemment, c'est-à-dire de protections verticales réalisées à l'aide de branchages avec ou sans support, le tout ayant une hauteur utile de 0,50 m à 1 m approximativement. Ils sont appelés suivant le cas ou la région : cordons, fascines ou palissades, etc. et demandent environ 1 à 4 T/ha de branchages ou de palmes. Les cordons comportent des piquets assez rapprochés (≅ 1 m) entre lesquels on tresse horizontalement les branchages. Les fascines sont des genres de haies plus ou moins denses faites à partir de fagots plantés dans le sable et souvent maintenus ainsi par des piquets et quelques fils de fer horizontaux. Parfois même il s'agit d'une simple rangée de branches avec leur feuillage, qui sont piquées toujours verticalement dans le sable à une profondeur de 20 à 30 cm (Tunisie), les intervalles étant égaux à leur diamètre moyen.

C'est ainsi d'ailleurs que sont réalisés les réticoli ou quadrillage du terrain en Tunisie et en Libye. Cette même rangée de branchages forme alors des carrés contigus de 5 à 10 mètres de côté, créant ainsi de petites enceintes très abritées dans lesquelles on arrive à faire pousser des arbustes ou granumées destinés à la fixation définitive. Ce quadrillage peur également être réalisé à partir de grille textile. Ce genre de protection peut également être réalisé à partir de planches à claire-voie qui sont surélevées successivement après ensablement mais c'est une solution coûteuse, aussi nous ne nous y attarderons pas.

Comme pour toutes les techniques à base de bois nous donnons des quantités de consommation qui doivent être considérées simplement comme des ordres de grandeurs car tous les facteurs élémentaires peuvent varier dans des proportions non négligeables : type de bois, essence, nombre de rangées par hectare, densité des branchages par unité de longueur de barrière. Ainsi pour les réticoli que nous avons vu en Tunisie on peut donner une évaluation de 5 T/ha.

Cordons et fascines sont utilisés au voisinage direct de la mer, c'est-à-dire en front de mer alors que les réticoli sont en général implantés dans des zones un peu plus calmes, à quelques centaines de mètres à l'arrière, où la végétation a davantage de chances de réussite.

Toutes ces solutions branchages sont efficaces lorsqu'elles sont correctement réalisées et entretenues ; leur ancienneté le prouve. Elles comportent cependant les deux défauts majeurs suivants :

- coût élevé en raison de la rareté du bois dans les pays concernés et plus généralement à cause de la main d'oeuvre très importante nécessaire au transport et à leur réalisation correcte et rapide ;

- deuxième inconvénient qui est sans doute le plus important car, comme nous l'avons déjà dit, il s'agit du mal chronique des pays des régions désertiques : le déboisement qui peut encore accentuer la catastrophique désertification ;

- enfin le volume et le poids de ce matériau représente un encombrement très important. Il demande par conséquent non seulement beaucoup de main d'oeuvre mais aussi un parc de véhicules très coûteux.

5-2-2 - La végétation seule

Nous avons déjà souligné la complémentarité réciproque de la végétation et du réseau de filaments mais il apparaît que cette végétation, sélectionnée parce qu'adaptée au milieu, devient parfois un concurrent. En effet cela se vérifie à partir du moment où elle est préférée à toute autre solution moderne, même si les techniciens ne sont pas assurés d'aboutir à un résultat positif. Par manque d'audace, d'initiative ou par ignorance, ils rejettent l'aide des produits nouveaux non traditionnels.

C'est ainsi que les jeunes plants d'arbres (hauteur 20 cm - intervalles 2 m à 2,50 m) ne sont pas assez efficaces tant qu'ils sont insuffisamment développés pour arrêter l'érosion ou l'invasion du sable. Il leur faut incontestablement une aide qui peut leur être donnée par les géotextiles qui ont été présentés ici. Sans cette aide les résultats sont très souvent négatifs. On le sait et on compte donc sur la chance ...

6 - ESSAIS ET REALISATIONS

6-1 Chronologie

Lors de la découverte de cette technique en 1964 nous avons réalisé quatre essais au cours des premières années sur de petites surfaces. Malgré des résultats très positifs nous avons abandonné ce projet car le principal utilisateur potentiel tenait à conserver les méthodes traditionnelles, les matériaux nécessaires à celles-ci lui étant fournis gratuitement par la forêt toute proche. Le compte rendu d'essai du Centre National de Recherche Forestière disait textuellement : "Nous sommes parfaitement d'accord sur la tenue très satisfaisante du voile employé en tapis horizontal sur la dune ..." mais il ne nous intéresse pas pour un emploi généralisé car cette technique "s'avère infiniment plus coûteuse, même lorsqu'elle est plus efficace ...".

Motivés par l'attitude de l'O.N.U. face aux problèmes de la désertification et compte tenu de cette réponse très favorable sur le plan purement technique, nous avons repris cette étude en 1975, d'autant plus que les prix étaient alors très nettement mieux adaptés. Au cours de cette année-là nous avons fait quelques essais de mise au point en France dans le but d'aborder les problèmes africains avec une technologie éprouvée et des prix compétitifs et adaptés.

Les résultats étant toujours positifs, au cours des cinq années suivantes nous avons fait dix-sept essais d'un hectare en moyenne chacun : sept ont été effectués en France dans le but de perfectionner les détails de cette technique et dix en Afrique dans un but de promotion auprès des administrations nationales et des organismes tels que la F.A.O. et des bureaux d'études.

Les contacts furent toujours difficiles à établir mais devant les résultats positifs d'un premier essai, l'administration responsable, semblant peu intéressée au départ, nous demandait quelques mois après de renouveler cette expérience, toujours à titre gratuit disant que cet essai-là "serait suivi beaucoup plus sérieusement ..." etc. Cela n'a pas toujours été possible en raison de la restriction des budgets affectés à nos déplacements et essais en cette période de crise.

6-2 Résultats

Les différents pays visités se situent dans la partie ouest et surtout nord de l'Afrique. Par ordre chronologique ce sont : Tunisie, Maroc, Sénégal, Algérie, Egypte, Libye. Dans leur majorité, les résultats, toujours difficilement obtenus, ne sont pas chiffrés et sont souvent transmis verbalement. Nous prendrons donc comme exemple ceux de la Tunisie, du Sénégal et de Marseille-Fos (France) car les résultats ont fait l'objet de courriers et de rapports.

6-2-1 France - Port autonome de Marseille-Fos

Il s'agit d'un essai du réseau textile seul et le compte rendu a été effectué par le Laboratoire Régional des Ponts et Chaussées de Toulouse.

Sous une vue générale de la zone traitée, alors qu'un fort vent s'est levé, on note "Immédiatement après la zone traitée il n'y a pas de vent de sable", montrant ainsi l'efficacité du traitement.

Par la suite l'ingénieur responsable a constaté la levée régulière de notre semis de luzerne, sauf dans la zone témoin et concurrente traitée par paillage.

6-2-2 Tunisie

Les conclusions sont données par un ingénieur de l'Institut National de Recherches Forestières ayant participé à cette réalisation "Les fibres ont stabilisé effectivement le mouvement de sable. Cela est visible aux abords des parcelles traitées et des parcelles non traitées ... Le cordon brise-vent en grille synthétique a été submergé de sable en 6 semaines ... semble donc être efficace pour la stabilisation des sables".

D'un autre ingénieur tunisien : "La fibre et le cordon (grille brise-vent) ont bien joué leur rôle de fixateurs".

6-2-3 Sénégal (Essais d'octobre 78 et de décembre 78)

- Rapport des Eaux et Forêts (décembre 1979 - lieu Malika) :

Le réseau "fixe effectivement le sable en supprimant tout transfert de sable à partir de la zone protégée". "Les plants de filao situés à l'extérieur" du réseau "ont été en grande partie déchaussés".

- Rapport de la F.A.O. (décembre 1979 - lieu Kebemer) "Une fois les poches remplies, le sable reste emprisonné. Cela fait que les plants ne souffrent plus du phénomène de dessouchement qui est un des problèmes majeurs ...".

Avantages cités par la F.A.O. :

. grande facilité de transport, agréable à manipuler, consomme moins de piquets (que les panneaux de branchages), facile à stocker et protège contre des vents de sens perpendiculaires ou opposés ;

. pas de parc important de véhicules et par conséquent diminution importante de la consommation de carburant.

Ces deux essais du Sénégal, représentant une surface protégée légèrement supérieure à un hectare, ont été suivi d'une réalisation très satisfaisante d'une centaine d'hectares.

La grille brise-vent ne fait pas dans cet exemple l'objet de commentaires mais elle a été utilisée régulièrement avec succès sur de très grandes longueurs.

CONCLUSION

Il est dommage l'on ne prête pas toujours une attention suffisante au problème de la progression du sable et que la lutte contre l'ensablement dans beaucoup de pays ne soit pas suffisamment structurée et soutenue par un plan d'ensemble.

Cependant les résultats, que tôt ou tard nous arrivons à connaître, sont dans leur grande majorité très positifs. Ils pourraient d'ailleurs l'être encore davantage grâce à des améliorations de détail encore possibles. En son état actuel cette méthode reçoit une large approbation de la part des ingénieurs et techniciens l'ayant réellement expérimentée sur le terrain. Ils déclarent que ces produits et notamment le réseau textile ont un très grand avenir dans la lutte contre l'ensablement.

Il serait donc souhaitable que cette opération soit reprise et poursuivie sous le patronnage d'un grand organisme disposant de l'autorité et des moyens nécessaires. Cela permettrait d'apporter tout d'abord une aide certaine au tiers monde et ensuite de trouver un débouché supplémentaire non négligeable à l'industrie textile doublement en crise aujourd'hui. Rappelons pour terminer que notre enquête portant simplement sur l'Afrique et le Proche-Orient nous a signalé qu'en dehors des grandes surfaces de désert sableux il y a des millions d'hectares susceptibles d'être protégés et aménagés. A raison d'une moyenne de 50 kg/ha de mèche, cela correspond à des quantités impressionnantes de matière textile.

Nous profitons de cette circonstance pour remercier les organismes et ingénieurs qui nous ont apporté leur concours par la mise à notre disposition de leurs connaissances, de leurs conseils, de leur participation active ou de leur aide en personnel et matériel.

Ces remerciements s'adressent tout particulièrement à :

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The Use of Honeycombed Geotextile Lap to Combat Erosion

Emploi de nappes géotextiles à structure alvéolaire pour la lutte contre l'érosion

Les dégradations superficielles des talus consécutives à l'érosion ou à "des glissements de peau" constituent un souci permanent pour les services chargés de l'entretien du réseau routier ou autoroutier. L'apparition sur le marché d'un dispositif original constitué d'une nappe géotextile à structure alvéolaire permet d'envisager une solution économique aux nombreux problèmes de protection superficielle. La présente communication traite de l'expérimentation de ce procédé effectuée en Octobre 1980 sur un talus dégradé et des constatations déduites à la suite d'une année d'observation qui démontrent son efficacité comme moyen de protection contre le ravinement, l'érosion et la desquamation de la surface des talus.

Surface weathering of embankement slopes due to plake type slides are permanent preoccupation of the Highway Départment maintenance crews. An original process recently launched on the market and including a honeycombed geotextile lap allows to solve the numerous problems encountered and that in an inexpensive way. The present paper deals with both the experimenting of this method in October 1980, on a weathered embankement slopes, and with the verifications done during one year observations on the field which prove the efficiency of this method as a mean of preventing gullying erosion and embankement slopes surface exfoliation.

I INTRODUCTION

Pour lutter contre les dégradations de talus dues essentiellement aux phénomènes d'érosion hydraulique externe, on disposait à ce jour de méthodes classiques qui consistent à agir sur la pente des talus (soit en la diminuant, soit en édifiant des terrasses), sur le compactage des talus (dans le cas de remblais) et sur la mise en végétation rapide.

Ces méthodes permettent en général de réduire l'érosion, celle-ci étant proportionnelle (1) à la pente du terrain et inversement proportionnelle à l'importance du couvert végétal. Elles ne sont toutefois pas suffisantes ou inapplicables dans certaines conditions de sols et de climat. Récemment sur le marché français est apparu un dispositif constitué d'une nappe alvéolaire composée de bandes de géotextile nontissé et soudées entre elles de manière à former un réseau d'alvéoles hexagonales. Ces alvéoles sont remplies de terre où la végétation peut s'implanter progressivement. Les parois des alvéoles empêchent le ruissellement d'entrainer la terre. Ce procédé a fait l'objet d'une expérimentation sur un talus très dégradé, en région méditerranéenne. Elle a consisté à suivre le comportement de la nappe en faisant varier la dimension des alvéoles, leur hauteur et la nature du matériau de remplissage.

Dans cette communication sont principalement développés les résultats obtenus au cours de la mise en place (déformation et déplacement des alvéoles) et après plusieurs mois d'application (aspect du talus, mouvements de la nappe et évolution du couvert végétal).

II DESCRIPTION DU PROCEDE

Le procédé étudié qui répond à la dénomination commerciale d'ARMATEX (R) est formé d'une nappe alvéolaire régulière constituée de bandes de géotextile soudées par points entre elles (fig.1)

Fig. 1 - Vue de détail des alvéoles

Le géotextile utilisé est un nontissé aiguilleté constitué de fibres continues polyester, imprégné par des résines pour conférer une certaine rigidité à la nappe (tableau 1).

Tableau 1 - Caractéristiques du géotextile

Masse Surfacique	320 g/m2	
Résistance à la traction	110 daN ± 15	Essais Norme Afnor G07001
Allongement à la rupture	40 % ± 8 %	

La forme hexagonale retenue pour les alvéoles résulte d'essais préliminaires portant sur la déformabilité tolérable et les conditions de fabrication de la nappe.

La mise en place consiste à déployer la nappe sur le talus à protéger, à l'ancrer en tête de talus, à l'épingler latéralement puis à remplir les alvéoles de matériau (terre végétale, par exemple).

III EXPERIMENTATION

III.1 Objectif de l'expérience

La zone choisie pour l'expérimentation est un talus autoroutier très dégradé, affecté par des glissements superficiels qui ont nécessité sur l'ensemble de la zone une reprise du talus et la mise en place d'une protection par masque drainant en enrochement. Cette technique sûre et éprouvée s'avère toutefois très onéreuse et d'utilisation parfois limitée. C'est pourquoi la solution plus économique apportée par la nappe géotextile à structure alvéolaire se devait d'être expérimentée afin de juger par comparaison d'un autre procédé de son efficacité (stabilité de la nappe, déformabilité du treillis de résistance au ravinement, etc ...).

Dans ce but, l'expérimentation envisagée se proposait de suivre la déformabilité de la nappe lors de sa mise en oeuvre, de vérifier sa stabilité dans le temps et son comportement aux sollicitations extérieures (écoulement des eaux, etc ...).

III.2 Modalités suivies

Après réglage du talus dont la hauteur est de l'ordre de 8 mètres et la pente géométrique moyenne de 78%, deux planches d'essai ont été délimitées sur le terrain en ne faisant varier pour simplifier l'opération de suivi qu'un seul paramètre : la dimension de l'alvéole.

C'est ainsi que la première aire d'essai(planche n°1) est constituée d'alvéoles hexagonales de 0,20 m de côté et formée de bandes de 0,10 m de hauteur. La deuxième planche expérimentale (planche n°2) est formée d'alvéoles hexagonales de 0,30 m de côté et de 0,10m de hauteur.

Le matériau choisi pour le remplissage est celui présentant les caractéristiques mécaniques les plus défavorables sous l'angle de la stabilité : la terre végétale. Le sol utilisé est en effet caractérisé par une granulométrie régulière 0/30 mm, un passant à 80 µm de 38 % et un indice de plasticité de 7.

Fig. 2 - Vue d'ensemble du site expérimental

Une troisième aire d'essai a été implantée sur le site sans mesures particulières, en utilisant pour la nappe géotextile des alvéoles hexagonales de 0,20 m de côté et de 0,07 m de hauteur et en matériau de remplissage des sous produits de carrière (granulométrie 0/40 mm, passant à 80 µm ≃ 18 %, Ip = 9).

III.3 Déroulement de l'expérimentation

Après avoir été déployée la nappe géotextile est ancrée à la partie supérieure du talus (fig.3) et épinglée latéralement afin de limiter la déformation de l'ensemble pendant chaque phase de remplissage des alvéoles.

Fig. 3 - Dispositif d'ancrage en crête de talus

Le déversement du matériau s'effectue, dans le cas de l'expérience étudiée, à partir de la crête du talus. Les opérations de remplissage et de réglage sont assurées manuellement le plus symétriquement possible.

Pour chaque planche, cinq lignes graduées sont matérialisées permettant le repérage des alvéoles en coordonnées (fig.4).

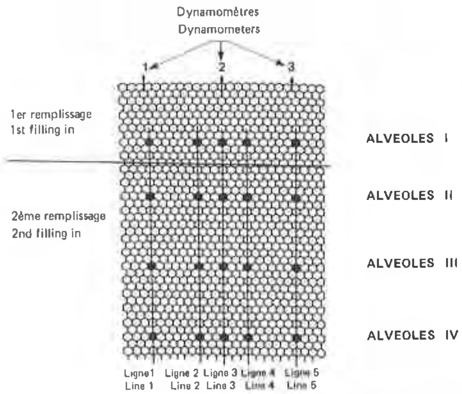

Fig. 4 - Principe du dispositif de mesure

La déformabilité de la nappe déployée déterminée par rapport à des repères fixes positionnés aux parties inférieure et supérieure du talus, est exprimée par la mesure du déplacement du centre de gravité d'alvéoles témoins à l'issue de chaque phase de remplissage et après la mise en place du dispositif.

Les efforts de traction développée dans la partie supérieure de la nappe sont par ailleurs enregistrés à l'aide de dynamomètres.

III.4 Comportement de la nappe lors de sa mise en place

Lors du remplissage des alvéoles les déplacements de la nappe ont été observés notamment dans sa partie centrale.

Fig.5 - Déformation de la nappe en cours de remplissage

Ces déplacements augmentent au fur et à mesure du chargement et dans les deux cas étudiés atteignent 0,55 m dans la partie inférieure de la nappe alvéolaire.

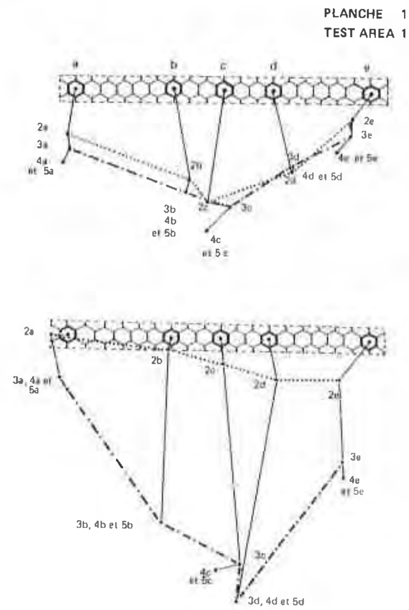

Fig.6 - Déplacement des alvéoles I et IV sur la planche numéro 1

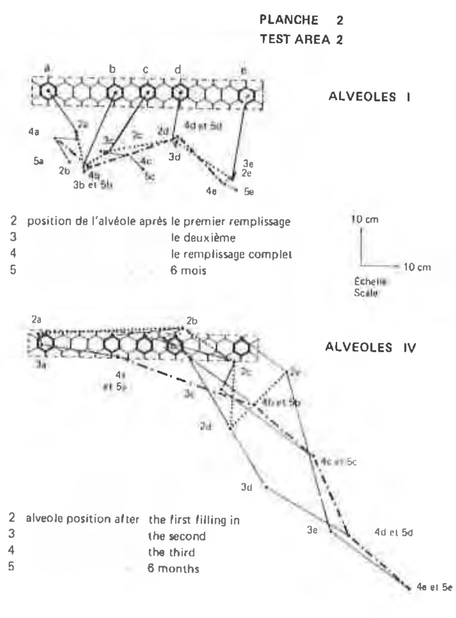

Fig.7 - Déplacement **des** alvéoles I et IV sur la planche numéro 2

Sur la planche n° 1, la nappe épinglée latéralement est légèrement tendue afin de favoriser la reprise rapide des efforts consécutifs au déversement brutal du matériau de remplissage, étalé symétriquement par bandes transversales à partir de l'axe vertical de l'aire d'essai. Les déformations des alvéoles sont peu importantes et les déplacements de la nappe se produisent suivant l'axe médian comme le montrent les schémas de la figure 6.

Sur la planche n° 2, la nappe épinglée sans tension latérale, est remplie par bandes de manière dissymétrique. Si la déformation des alvéoles demeure limitée, le déplacement du treillis géotextile produit au fur et à mesure du remplissage est surtout conséquent dans sa partie inférieure ($\varepsilon \approx 0{,}60$ m) (fig.7).

La comparaison des deux aires d'essai montre, comme il était prévisible, que la déformation est fonction de la dimension des alvéoles (fig.8) mais aussi du procédé de fixation latérale et de la méthode de remplissage.

Les forces de traction enregistrées dans l'axe médian des deux nappes étudiées augmentent progressivement pendant la phase de remplissage et tendent vers une limite correspondant à la mobilisation de la résistance au cisaillement du sol utilisé. Les valeurs obtenues identiques dans les deux cas, varient de 2 daN avant remplissage à 14 daN après remplissage.

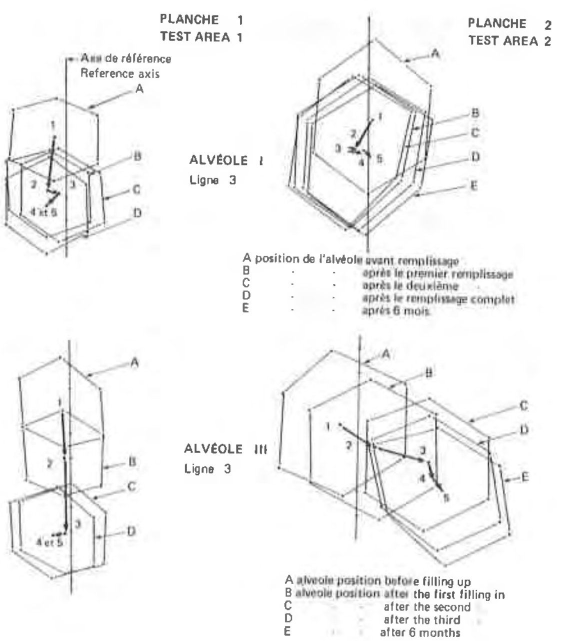

Fig. 8 - Shémas comparatifs de déformation des alvéoles sur les deux planches d'essai

III.5 Comportement de la nappe après sa mise en place

Les constatations effectuées pendant une année et notamment après de très violents orages, démontrent que le dispositif est très peu affecté par les éléments extérieurs. Les déplacements sont négligeables quelles que soient les dimensions des alvéoles et le matériau utilisé.

Elles mettent en évidence une meilleure résistance au lessivage des nappes constituées d'alvéoles de dimensions réduites et remplies de matériaux de caractéristiques mécaniques élevées et une implantation de la végétation largement facilitée par la configuration alvéolaire du treillis.

En effet, une année après la mise en place de la nappe, il est difficile de distinguer la présence du géotextile dans la zone revêtue de terre végétale (Fig. 9).

Fig.9 - Vue d'ensemble de la zone d'essai un an après sa mise en place

V CONCLUSIONS

A l'issue de cette expérimentation, il apparaît que l'utilisation de nappes géotextiles à structure alvéolaire constitue un procédé simple et efficace de protection contre l'érosion des talus.

Le fonctionnement de l'ARMATEX (R) a été analysé d'un seul point de vue mécanique. Sa fonction essentielle est de bloquer le processus d'érosion régressive en empêchant la formation de ravines de longueur supérieure à celle d'une alvéole. Son efficacité est d'autant plus grande que la dimension de l'alvéole est plus petite.

Les parois des alvéoles peuvent avoir aussi étant donné leur nature (nontissé de polyester), une fonction hydraulique et constituer ainsi un réseau de drainage très dense et très efficace dont l'action, qui n'a pas été étudiée dans cette première expérimentation, est certainement très importante.

Enfin, ce procédé comparé aux méthodes traditionnelles du fascinage, a l'avantage d'être facile à mettre en oeuvre et d'un coût modéré. Comme pour tous les géotextiles, les dépenses de transport sont peu élevées, les éléments à transporter étant légers (2 à 300 g/m2 pour des alvéoles de 20 cm de côté et 10 cm de hauteur) et peu encombrants.

REMERCIEMENTS

Nous tenons à remercier d'une part l'Inventeur du procédé, Monsieur VIGNON J.F., et son équipe, et d'autre part, la Société des Autoroutes du Sud de la réalisation des planches expérimentales.

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Living Sheets on Steep Slopes
La stabilization végétal de talus très inclinés

Steep slopes covered and stabilized with natural landscaping materials in conjunction with geotextiles were constructed in order to determine their effectiveness and economy. A trial pit was dug with the sides set at steep slopes and three segments were formed for testing purposes. The first segment was left to weather normally as a control reference: the second was stabilized with geotextile bags filled with crushed limestone and strengthened with living willow shoots: the third was faced down the slope with geotextile matting staked into position and sprayed with a seeded mulch. Over a period of seasons the slopes were monitored and it was seen that the young willow shoots had grown strongly and that their segment of the slope had remained stable, whereas the other two segments had suffered weathering and surface slipping. The use of the geotextile bags in conjunction with the willows had thus enabled a significant improvement in the stability of the soil slope to be achieved. Larger tests are planned, formulae will be drawn up and further research will seek optimal aspects.

La stabilisation de talus très inclinés à partir de couverture et texture végétales se révêlerait efficace et économique. Des essais ont été effectués dans une fouille à parois très inclinées, divisées en trois tranches: la première laissée telle que, servit de témoin pour mesures de contrôle. La seconde stabilisée par des boudins de géotextile à remplissage de calcaire concassé renforcé de plants de saule. La troisième stabilisée par une couche de terre rapportée armée d'un treillis synthétique et ensemencée. A la fin du premier cycle végétal on a pu observer le bon développement des jeunes plants de saule, la tranche est demeurée stable, tandis que les deux autres tranches montraient des traces d'érosion et glissements. L'emploi de boudins géotextiles combinés avec le saule a influencé fortement la stabilité du talus. Des essais plus étendus sont prévus dans le but de déterminer avec précision les conditions optimales de mise en oeuvre.

1 SCOPE

The complexity of embankment construction projects becomes increasingly handicapped through the difficulty of proving appropriate surfaces for them. Moreover, the public is increasingly demanding that modern roads make less impact upon the environment and that, where large through streets occur, noise protection embankments or walls are provided or that the streets are set in cuttings or roofed over.

From these requirements there comes an economical demand to seek to develop the inherent strength of slopes and to achieve two objectives: to permanently stabilize the slopes without extensive building operations and to minimize the surface areas required.

It was decided to investigate the possibilities which presented themselves in Baden-Württemberg if one is required to produce steep slopes in weathered soft rock of Keuper age. Following a suggestion by the first author, the concept of a living sheet was developed. Consultations took place with engineer biologist Professor Schiechtl, Innsbruck, and Dipl.-Ing. Härle of the Highway Department, Baden-Württemberg. The authors thank both these gentlemen for their valuable suggestions.

2 PHYSICAL SOIL ASPECTS

Normal experience indicates that the Keuper - which predominantly contains clay minerals such as Illite, Corrensit and Chlorite and has a small-fissured, stratified structure with deposits of sandstone banks and Dolomite layers - will, when first cut, stand almost vertical to a height of 15 - 20 metres. However, under weathering action, strenght is lost and the material moves forward so that fractures occur in increments and screes form at the foot of the slope.

The cut face is only stable as long as the cohesion of the materials is able to resist tension forces within the soil body. Due to the horizontal relaxation of the soil following excavation of the hill, fissures are formed vertically and falling rain is allowed to enter so as to further accelerate the weakening of the material.

In order to favourably influence the maintenance of the soil cohesion through building techniques it is necessary to insulate the freshly exposed cut face against the atmosphere, as is regularly done in building excavations by means of shotcrete.

Figure 1 is a photograph of the claystone structure as is often met with. The seepage water flows out only sporadically because the material is generally impervious to water. The fissured matter has only a relatively insignificant strength and allows itself to be easily loosened and broken in the hand. The consistency fluctuates between stiff and semi-firm, the water content being predominantly below the plastic limit.

If the weathering is not prevented, then the soil deteriorates into a clay slurry with a shear angle of 15° to 20° and the effective cohesion decreases from an initial value of 50 - 100 kN/m² down to practically zero.

Fig. 1 Structure of the claystone

However, before reaching the minimum strength, the slope fails.

3 INVESTIGATED SOLUTIONS

In a test pit of 5 m depth, Figure 2, two 20 m long slopes with an inclination of 2:1, corresponding to a slope angle of 64°, wre formed. This appeared to be the steepest slope workable in the use of living sheets, described below. The soil succession here was a 1.7 to 2.0 m coverlayer thickness of gravelly to stoney loam, underlain by 0.3 to 0.4 m of thick chalkstone band underlain by that which was shown in Figure 1, namely stiff to semi-firm claystone (Formation: Black Jura, Lias α).

The soil mechanics parameter are:

$$w = 11 - 14\ \%,\ w_L = 32 - 37\ \%,\ w_p = 14 - 19\ \%,$$
$$\phi' = 22°,\ c' = 80\ kN/m^2$$

whereas the soil parameters of the cover layer are:

$$\phi' = 20°,\ c' = 14\ kN/m^2.$$

If a uniform layer of unit weight 21 kN/m is applied then the slope remains stable, as the effective cohesion is kept above 10.5 kN/m². The slope lengths were arranged into three segments (Figure 2): an unstabilised control segment 1, at which the pregressive weakening of the earth slope was studied; segment 2 which was stabilised with HaTe bags in a form developed by IGB; segment 3 which was stabilized by the use of Enkamat mats and green planting.

Figure 3 shows cross sections through the slope stabilised using the IGB method and demonstrates that two distinct geometrical shapes were chosen for testing. Moreover, the placing of a small berm provided advantages for access to the willow branches during trimming.

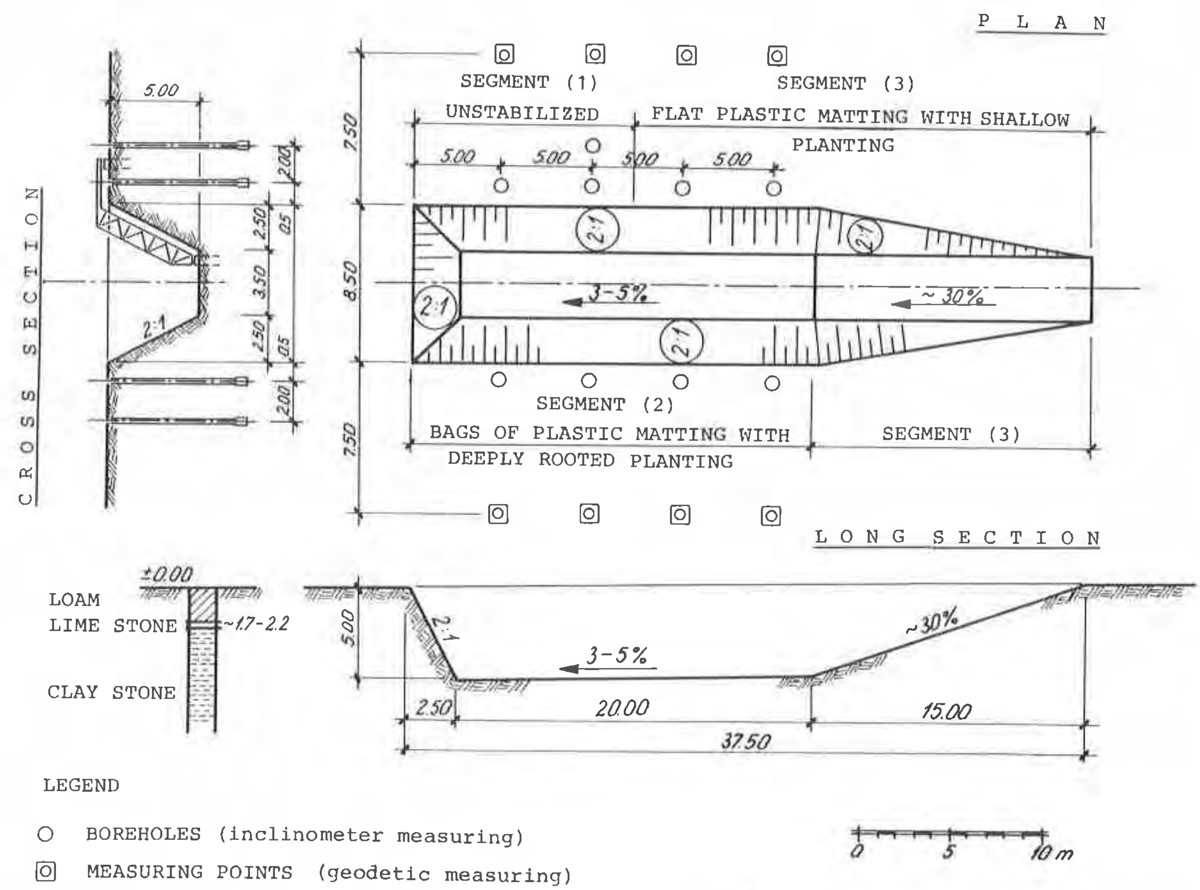

Fig. 2 Test pit

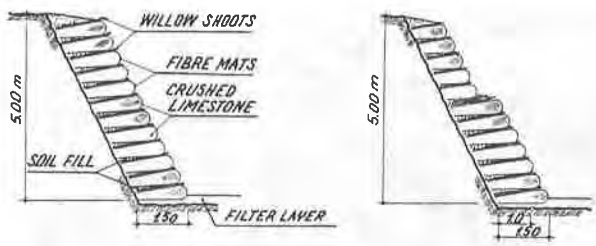

Fig. 3 Slope stability in cross sections

Figure 4 shows as cross section of the construction in greater detail whilst Figure 5 is a photograph of the matting. 4.0 m wide mats were arranged to form courses of 1.5 m width, each course being filled to a depth of 40 cm with size 0 - 45 mm crushed limestone. The upper surface of each course was laid to a slope of about 10° to the horizontal (required by the living sheets) and was covered with about 10 cm of bedding soil in which the willow shoots were laid at 0.1 m centres in April 1980.

The installation procedure is illustrated in Figures 6 to 8. Formation of the bag shape was facilitated by use of a piece of half section pipe. Consolidation was effected by four passes of a small vibrator.

To stabilize the ENKA mats, Figure 9, a 20 mm thick plastic fabric was hung over the slope and spiked and was then sprayed with a mexture of straw mulch, adhesive and grass seed. The method of installation is well known and simple and was able to be done by the Institute people themselves.

4 ACCOMPANYING SURVEY

The question of which accompanying survey - apart from the laboratory soil mechanics tests - was reasonable for the control of the work was not easy to answer, because appearance was the only real control. In addition to the geodetic measuring (see Figure 2), 10 measurement points were chosen for inclinometer measuring, in order to follow possible movements of the slope. Of course, continuous meteorological data were recorded. To our knowledge, there were no testing procedures known for the control of progressive weakening of an unsafe earth

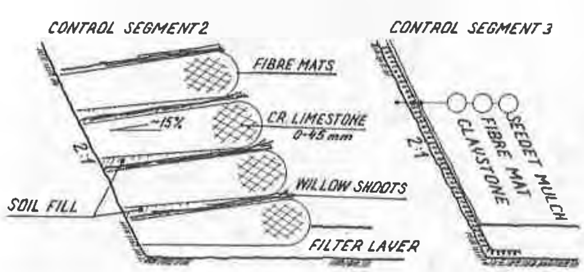

Fig. 4 Details of the slope stability in control segments 2 and 3

wall. We therefore built ourselves a travelling framework, as seen in Figure 10, to enable us to take point soundings on the slope.

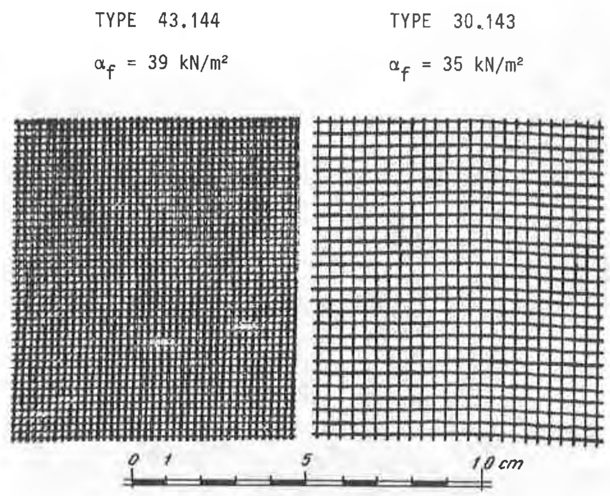

Fig. 5 Structure of the HaTe mats

Fig. 6 IGB stability slope, partly constructed

Fig. 7 IGB stability slope, partly constructed

Fig. 8 IGB stability slope, partly constructed

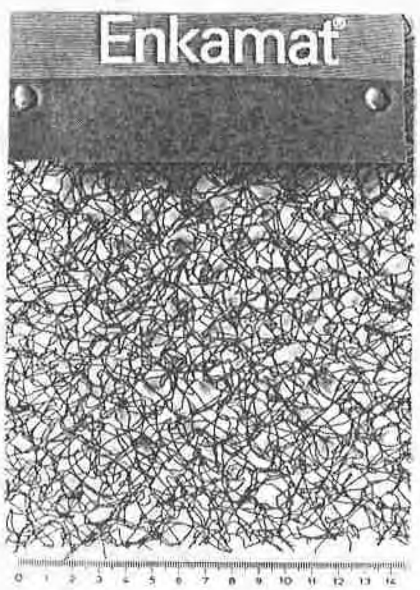

Fig. 9 Structure of the Enkamat mat

Fig. 10 Travelling framework for control measurements on the slope

5 RESULTS TO DATE

In spite of the fact that the planting was not carried out in the optimum season, the plants grew satisfactorily and after only two months had reached the stage shown in Figure 11, where generally the plastic fabrics were not visible outwardly.

Large differences were exhibited in the stability of the construction types. whist in the IGB segment no measurable slips were recordet and the slope remained unchanged over a full summer-winter period, the surface of segment exhibited slipping and an example of this is shown in Figure 12. It became loosened through the considerable rainfall experienced in 1980. We see here a clear advantage of the solution using bags, which hold a sufficient volume of water for the use of the plants yet allow seepage to prevent the development of eccessive water pressures behind the vegetation.

A drainage system was installed in the valley floor to collect run-off.

6 FURTHER WORK

According to the way that the IGB procedures showed satisfactory practical results, the following matters have yet to be investigated.

6.1 In ggreement with the Highway Department of Baden-Württemberg the method will be used to construct a 10m high slope over a length of about 100m.

6.2 In order to gain further information prior to the construction of 6.1, a test load will be carried out on

Fig.11 Condition of the IGB stability slope after two months

Fig. 12 Slips in control segment 3

the crest of the existing slope to study the conditions of failure under the actions of loads greater than self weight.

6.3 The support frame for the installation of the bags will be extendet to be applicable for any slope height between 3 and 20m.

6.4 Technical improvements will be examined whereby possible optimisation and increased use of mechanisation can be used in the construction.

6.5 Fot the new construction type a soil static stabiloiy verification will be produced.

6.6 Tests will be made of other plant types:essentially, thewillow was chosen for its resistance to salts and its ease of maintenance, for the construction of higher slopes, however, there are possibly other suitable plant types.

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Investigation on Long-Term Behavior of Geotextiles in Bank Protection Works

Recherches sur le comportement à long terme des géotextiles utilisés dans la protection des berges

A two years prototype investigation started in 1980 on the long-term behaviour of geotextiles when applied as a filter in bank protection works. Many structures with a minimum age of 7 years, and representing various combinations of circumstances (different types of geotextile, soil, aquatic environment and hydraulic conditions) are involved. The investigation has been set-up primarily to gain knowledge on the hydraulic and soil mechanical behaviour of the geotextile in combination with underlaying soil. Therefore a number of undisturbed ground samples, including the geotextile, are taken in each of the selected sites.
The following laboratory tests have been performed :
(a) permeability; both for the ground samples and the geotextile alone, (b) sieve analysis of thin layers of the soil at various depths below the fabric, (c) largest pore sizes of the geotextile and (d) physical-chemical characteristics of the geotextile.
Their results are interpreted by means of a number of factors, enabling judgement of the behaviour of the geotextiles.

Des recherches in-situ sur le comportement à long terme des géotextiles utilisés dans la protection des berges se sont déroulées entre 1980 et 1982. Un grand nombre de constructions, agées d'au moins 7 ans et représentant différentes combinaisons de circonstances (types de géotextile, sols, environnement aquatique et conditions hydrauliques) sont étudiées. Des recherches ont été entreprises principalement en vue d'étendre les connaissances concernant le comportement hydraulique et géomécanique du textile et du sol sous-jacent. Dans ce but, un certain nombre d'échantillons des sols avec le textile ont été relevé dans chaque site sélectionné. Les expériences suivantes ont été réalisées en laboratoire : (a) perméabilité; tant pour les échantillons de sol que pour les textiles seuls, (b) analyses granulométriques de fines couches du sol à différentes profondeurs sous les textiles, (c) caractéristiques des mailles des textiles et (d) caractéristiques physico-chimiques des textiles. Les résultats de ces expériences sont interprétés à l'aide d'un certain nombre de paramètres, permettant ainsi un jugement sur le comportement des géotextiles.

1. INTRODUCTION

The "Nederlandse Vereniging Kust- en Oeverwerken" (K&O) started in the seventies a study on standard designs for bank protection works. Its objective is to give standard solutions for the following restricted cases :

- constructions of limited length, for which case normal design procedures would be relatively too expensive,
- constructions being attacked so little, that a minimum construction is sufficient for the given circumstances.

Moreover, costs of bank protection works can be assessed on base of the standard designs in pre-feasibility and feasibility studies.

During the above study it appeared that little was known about the long-term behaviour of geotextiles, being an important part of modern bank protection works. The same lack of knowledge became obvious during the International Conference on the use of fabrics in geotechnics, Paris, 1977.
Both events led to the set-up of a field study to investigate the long-term behaviour of geotextiles when applied as a filter in bank protection works. A pilot investigation, carried out in 1978, brought about that this aim could be attained by setting-up a more extensive investigation (1).
This two years prototype investigation, which started in 1980, is described in this paper. After a brief discussion on the main features of bank protection works and the application of geotextiles in bank protection works, the set-up of the investigation and execution of fieldwork are described. Next, the laboratory tests and their results are presented, followed by the preliminary conclusions drawn so far from the study results.

2. MAIN FEATURES OF BANK PROTECTION WORKS

A bank protection must be able to withstand all relevant forces endangering the bank stability. Such forces originate from hydraulic phenomena taking place in the waterway, and can be divided into two types (2) :

a) External forces : acting mainly on the surface layer of the bank protection; being shear and pressure forces due to currents and waves in the waterway.

b) Internal forces : acting on the body of the bank; being pressure forces caused by groundwater flow. Besides the Ground Water Table (G.W.T.), the groundwater flow in the bank is largely influenced by the water level in the waterway and its fluctuations.

Both the external and internal forces thus include (quasi) stationary as well as non-stationary components. Stationary forces are associated with river or canal run-offs, flood waves, tidal motion and seepage flow. Rapidly varied forces are caused by translatory and surface waves and ship induced water motion. With the progressive increase in traffic and ship sizes, the latter is increasingly becoming the major factor governing the design of bank protection works (3).

A bank protection can be either pervious or impervious. The mode of functioning and the mechanisms involved are completely different for both types. In view to the

application of geotextiles as a filter, the discussion in this paper is restricted to the pervious type.
To be able to resist the external current and wave forces, the top layer of the bank protection is usually composed of coarse or very coarse grains. Between this layer and the bank material, one or more layers - called filter layer(s) - are applied. These filter layers are very important; in their absence the bank material can easily erode away amongst the grains of the top layer. The transport would take place due to groundwater flow, which includes here the flow through the grains of the top layer. Depending on the hydraulic phenomena involved, stationary and/or cyclic groundwater flow takes place, having components both parallel and perpendicular to the plane of the bank. For a more detailed discussion of the mechanisms involved, reference is made to (2).

3. APPLICATION OF GEOTEXTILES IN BANK PROTECTION WORKS

Traditionally, granular filter layers have been applied in bank protections, the number of layers being dependent on the size amplification between the bank material and the required top layer. Recently in the Netherlands, geotextiles have been succesfully applied as a replacement of the granular filter layers, not only in bank protection but also in bottom protection works. A typical cross-section of a bank protection including a geotextile is shown in figure 1.

To fulfil its function properly, either the granular or geotextile filter should comply with the following two requirements (4) :

1. It must be more permeable than the bank material, in order to avoid the development of any appreciable uplift pressures.
2. It prevents the bank grains from motion under the effect of expected hydraulic gradients of the groundwater flow (sandtightness).

The Delft Hydraulics Laboratory has developed standard methods to test the above characteristics in the case of a geotextile. These tests, which are described in (5), form a part of the hydraulic tests in the present study.

The geotextile filter has the advantage of being cheaper, easier and quicker in execution than a granular filter. Its main disadvantage, however, is that it is liable to be clogged, not only by silt and other fines in the water or soil, but also due to biological or chemical processes (6). This aspect, and its effect on the functioning of the geotextile, is one of the main items of this study.

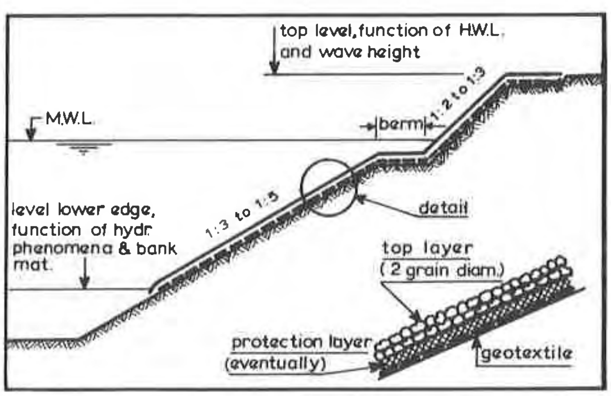

Fig. 1 Typical cross-section of a bank protection including a geotextile filter.

Fig. 2 View of a site during sampling

4. SET-UP OF INVESTIGATION AND EXECUTION OF FIELDWORK

The aim of the investigation is to gain knowledge on the long-term behaviour of geotextiles when applied in bank protection works, particularly with regard to phenomena as ageing, blocking and clogging.
Ageing is a function of composition and characteristics of the raw material used in manufacturing the geotextile water quality (pollution), forces acting on the geotextile and time.
Blocking and clogging are functions of properties of the geotextile (thickness, pore sizes), hydraulic circumstances in the waterway, water quality (silt), composition of the subsoil and time. Their effect may cause a considerable change in the hydraulic gradients and result in an appreciable uplift pressure directly under the bank protection.

The extent of ageing is determined by physical-chemical tests of the geotextile and by comparing the results with the original properties. The extent of blocking and clogging is determined by hydraulic and soil mechanical tests on undisturbed ground samples including the geotextile above it. These laboratory tests are described in section 5.
A similar investigation has been carried out by Heerten (7). However, the geotextile and the subsoil were sampled apart. Due to the ralatively large variations of geotextile and subsoil characteristics, it is important to take samples containing both.

To perform the tests, samples are needed of :
- the geotextile for the physical-chemical tests;
- the geotextile and undisturbed ground samples for the hydraulic and soil mechanical tests.

In the K&O-investigation, samples were taken from 33 sites alongside Dutch canals, rivers and coast. The minimum age of the structures sampled is 7 years. The variables influencing the behaviour of the geotextile in bank protection works are numerous. The most important of these, as well as a broad classification, made in the context of the present study, are given below :

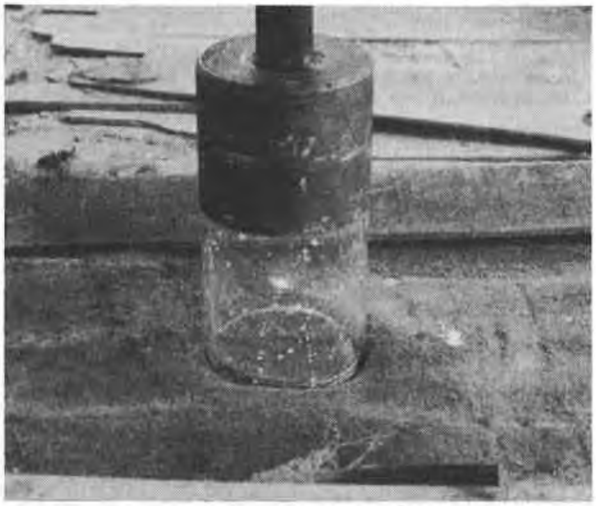

Fig. 3 A plexiglass sampling tube is pressed through the geotextile into the ground.

- Type of geotextile (all woven) : (a) polypropylene, polyethylene (b) polyamide.
- Grain size of the bank material : (a) fine (b) coarse.
- Aquatic environment : (a) salt (b) fresh.
- Hydraulic phenomena acting on the bank protection, including ship-induced water motion : (a) light (b) heavy.
- Location of the fabric : (a) about 1 to 2 m above water level (b) more or less at water level (during sampling). Taking of undisturbed samples under water was not feasible.

The sites selected for sampling were chosen such that all possible combinations of above classes are covered.

The sampling procedure can be divided into three parts :

a) Removing of the surface layers :
First of all the surface layers, covering the geotextile, must be removed. This has to be done very carefully; the geotextile may not be damaged. Mostly a crane is used for the execution of this job. The last layer is removed by hand. The surface cleaned amounts to 20 up to 25 m².

b) Taking of the samples :
The ground samples are taken by means of a cone penetration apparatus, mounted on a mobile frame (see figure 2). This apparatus presses a plexiglass sampling tube (height 200 mm, diameter 100 mm) through the geotextile into the ground (see figure 3). Prior to this, the geotextile is cut loose circularly (also diameter 100 mm) by a thermal device. Thus, undisturbed samples with a piece of the geotextile on top of it are obtained. Fifteen ground samples are taken at each site. After removing a large piece of the geotextile (about 10 m²), the ground samples are carefully excavated, making sure that they remain undisturbed.

c) Repairing of the construction :
A new geotextile is laid down. The surface layers are restored.

5. LABORATORY TESTS AND THEIR RESULTS

5.1. Hydraulic and Soil Mechanical Tests

The following tests have been performed at the Delft Hydraulics Laboratory and the Delft Soil Mechanics Laboratory :

i) Permeability tests

A special apparatus has been constructed in which the plexiglass sampling tube can be fitted (see figures 4 and 5). Six holes for connecting 3 mm piezometric tubes are bored, one above the level of the geotextile and 5 below it; at 5, 35, 65, 95 and 125 mm distance.
The piezometric head (h) is measured at the above 6 points at 3 different discharges. The results illustrate the variation of the hydraulic gradient in the bank section under stationary conditions (see figure 6) and provide information on the coefficient of permeability (k) of the various layers directly under the geotextile. The latter is computed according to Darcy's law (i.e. assuming laminar flow).

$$v = k\ i \qquad (1)$$

The coefficient of permeability cannot be computed for the upper 5 mm layer soil, since the head difference between points 1 and 2 (see figure 4) includes both the head loss in that layer and across the geotextile.

The head loss over the geotextile is measured in a similar apparatus (see figure 5), where the fabric piece alone is fixed in a socket. In order to count for pre-Darcy regimes (relatively large pores of geotextile), the head difference (Δh) - filter velocity (v) relationship takes the general form :

$$\Delta h = a\ v^m \qquad (2)$$

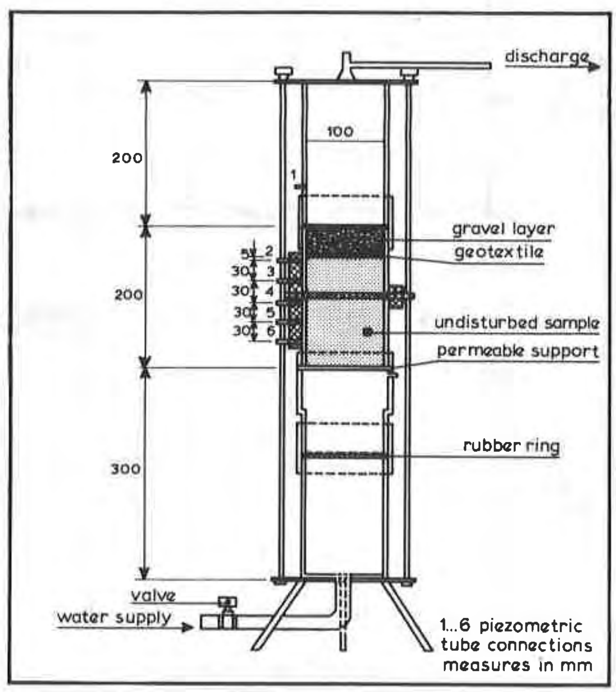

Fig. 4 Permeability apparatus for ground samples

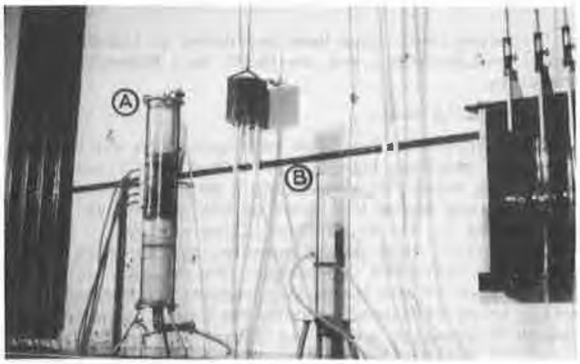

Fig. 5 Permeability apparatus for ground samples (A) and for geotextile alone (B)

where :

a = resistance factor

m = exponent, characterizing the flow regime (= 1 in case of laminar regime, 2 for turbulent regime and in between for transitional regime).

The head difference corresponding to v = 0.01 m/s is taken as the characteristic measure for the geotextile permeability (notation Δh_1) (4).

The fabric piece on top of the ground sample is first tested. The resulting Δh_1 is characteristic for its state in the bank protection during sampling. Thereafter, the geotextile piece is cleansed by laying it in an ultrasonic bath for about 30 minutes. The cleansed geotextile is re-tested, yielding Δh_1^*, which is considered representative for the original state of the geotextile. The above tests have been performed on 5 of the 15 undisturbed samples taken at each site.

ii) Sieve analysis of bank material at various depths

A sieve analysis of 4 thin layers has been carried out on another 5 of the undisturbed ground samples. The layers have a thickness of 5 mm each, being taken between 0-5 mm, 5-10 mm, 10-15 mm and 150-155 mm under the geotextile. The analysis took place by means of standard sieves ($\geqslant$ 37 µm) or by deposition in the cylinder of Attenberg or in an areometer.

The tests (i) and (ii) aim to provide information on possible changes in the composition of the bank material directly under the revetment, as well as changes in the permeability of the geotextile. Two examples of the results obtained are shown in figure 6, giving the readings of the 5 samples used in both tests. The piezometric heads are given as a percentage of the head difference

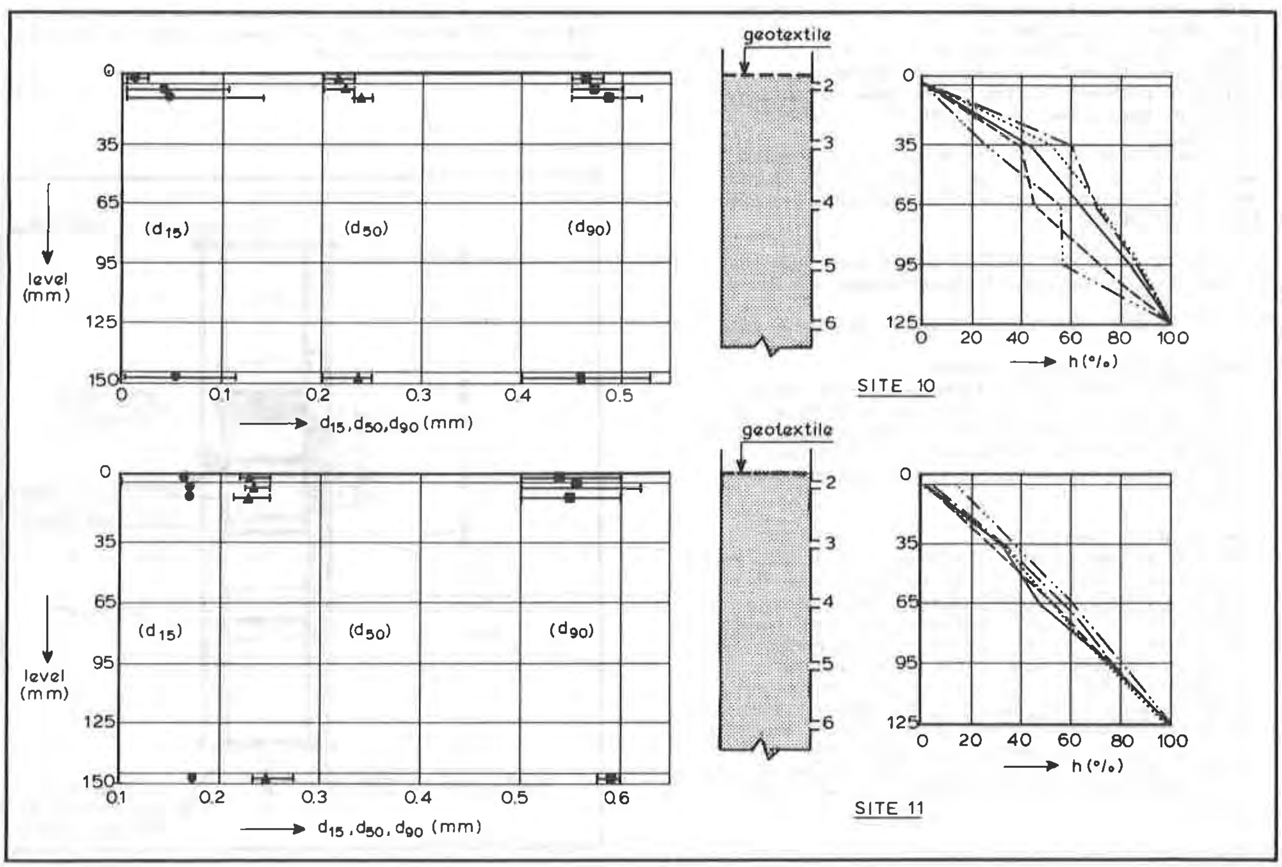

Fig. 6 Two examples of tests (i) and (ii)

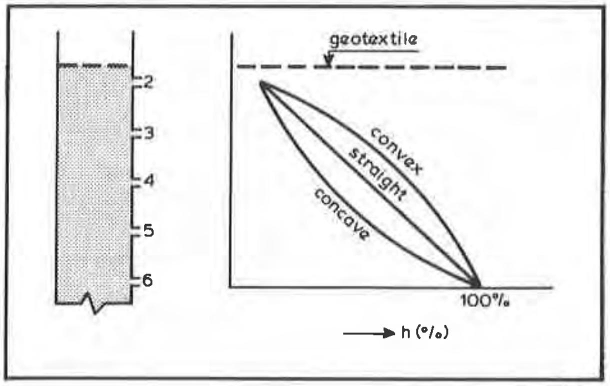

Fig. 7 Form factor hydraulic gradient

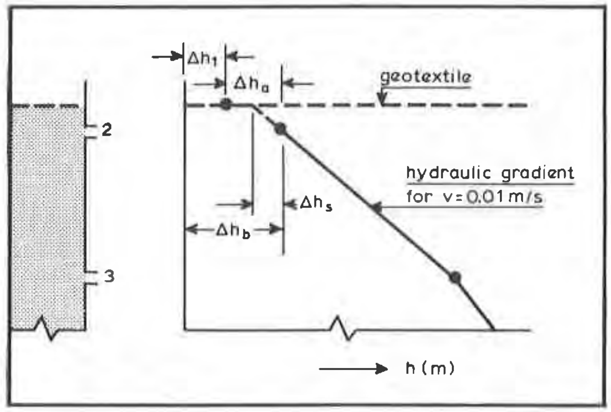

Fig. 8 Basic quantities for calculating factors F1 to F4 (computed for v = 0.01 m/s)

between points 1 and 6. Both the mean and the variation (amongst the 5 samples) of the characteristic grain diameters d_{15}, d_{50} and d_{90} are shown in the left figure.

The behaviour of the geotextile in the bank protection is judged on base of the following 5 factors, to be determined from the test results :

a) Form factor hydraulic gradient (see figure 7), giving an overall description of the variation of the hydraulic gradient as a function of depth and hence possible changes in the composition of the soil.
b) $F1 = (\Delta h_s - \Delta h_a) / \Delta h_s$ (see figure 8), providing information on the variation of the permeability of the upper 5 mm soil layer relative to the 30 mm layer directly below it.
c) $F2 = (\Delta h_1 - \Delta h_1^*) / \Delta h_1^*$ (see under (i)), yielding the degree of reduction in the geotextile permeability, due to blocking or clogging.
d) $F3 = \Delta h_b$ (in % relative to h at point 6, see figure 6), indicating whether appreciable uplift pressures can be feared. Taking into consideration possible changes in the composition of the subsoil, the permeability design requirement of the geotextile (section 3) can be translated as follows. The hydraulic gradient directly under the toplayer (arbitrarily taken here over the geotextile and underlaying 5 mm soil $= i_g$) must not show an abrupt increase with respect to the average gradient in the subsoil (i). An acceptable i_g/i ratio depends on the prevailing groundwater flow and submerged weight of the top layer(s). In the present investigation it has been arbitrarily taken = 2 to 3, which means that there is no fear of unfavorable uplift pressures as long as F3 < 10 % (depth ratio = 1:25).
e) $F4 = \Delta h_1 / \Delta h_b$ (see figure 8), indicating whether an uplift pressure is mainly due to geotextile clogging or due to blocking of the underlaying subsoil.

The above factors as calculated for the two examples shown in figure 6 are given in table 1, together with the subsequent judgement.
The agreement between the conclusions based on the above factors and the results of the sieve analyses (ii) is - in general - moderate. A difficulty thereby is the relatively large scatter in the results.

(iii) Pore sizes of the geotextile

A standard sieve test (5) has been applied to determine the largest pore sizes of the geotextile samples taken at the various sites. Because the geotextile with a sand fraction on top of it is vibrated, the O_{90} and O_{98} determined can be considered to apply for the cleansed geotextile, i.e. in its original state. They are to be compared with the characteristic grain diameters of the subsoil.

For the 33 sites investigated, an analysis - relating the results of tests (i), (ii) and (iii) to the parameters influencing the behaviour of the geotextile - is being currently made.

Table 1. Factors judging long-term behaviour of geotextile

site	sample	Factors					Judgement
		Form factor	F1	F2	F3	F4	
10	1	convex	0.9	34.7	0.4	1.2	* Limited layer under the geotextile has become less permeable (except upper 5 mm).
	2	convex	0.9	169	0.5	1.5	* Factor F1 does not agree with sieve analysis results.
	3	convex	~1	214	~0	---	* Geotextile clogging is considerable.
	4	convex	~1	289	~0	---	* Nevertheless no fear for appreciable uplift pressures.
	5	straight	0.7	319	1.1	1.3	
11	1	straight	0.8	~0	0.8	0.6	* Generally no changes in the subsoil.
	2	straight	0.6	0.6	1.7	1.0	* Locally geotextile moderately clogged.
	3	straight	0.4	11.9	2.9	7.5	* Locally fear for appreciable uplift pressures, mainly due to blocking of subsoil.
	4	straight	-2.7	~0	14.6	0.1	
	5	straight	0.6	~0	1.6	0.3	

5.2. Physical-chemical Tests

Tests have been performed at the Fibre Research Institute TNO, Delft (physical tests) and at the Plastics and Rubber Institute TNO, Delft (chemical tests). The results of these tests are compared with the original properties, in order to evaluate the ageing of the geotextiles.

(iv) Physical tests

According to Dutch standards, tests are performed determining the number of threads per 100 mm, yarncount, crimp, breaking strength and elongation at break.

(v) Chemical tests

Tests are performed in order to determine the type of polymer, ratio of weight polypropylene / polyethylene (polypropylene tape fabrics may contain a percentage polyethylene), quality and quantity of additives and thermal resistance. These tests are described in (8).

(vi) Original properties

Comparing the results of above tests with the original properties gives knowledge about possible degradation of the geotextile and the expectation of its long-term behaviour. The tracing of the original properties is sometimes obstructed by the absence of data.
Physical properties have been traced for 70 % of the geotextiles sampled. To determine the chemical properties of the raw material, "archive" geotextiles have been tested, i.e. geotextiles, having the same quality and being as old as the geotextiles sampled. These geotextiles have been stored in an archive. In this way, a good picture is formed of the composition of the raw material, mostly not changing for a number of years. The chemical tests on the archive geotextiles are still running.

Table 2 shows some results of the physical tests of a polypropylene, a polyethylene and a polyamide geotextile. Also the original properties, so far as known, are presented. The mass is calculated from the properties determined.

From the results obtained so far, it appears that geotextile clogging with materials containing iron compounds shortens its lifetime.
Both the physical and the chemical test brought out that some geotextiles were manufactured from yarns of different composition. The location of the geotextile (above water level or more or less at water level) does not lead to differences in degradation of the properties of the geotextile.

Table 2. Results of physical tests

Properties	Geotextile					
	polypropylene		polyethylene		polyamide	
Year of construction bank protection	1970		1969		1969	
	now	original	now	original	now	original
Mass per unit area μ (g/m²)	120	116	212	220	110	92
Breaking force F_T (N) *						
warp	26.8	34.0	26.9	30.2	12.7	13.7
weft	20.6	27.8	27.2	27.0	10.5	13.7
Elongation at break (%)						
warp	9.0	16 - 18	26.3	34.5	20.1	20.4
weft	6.5	16 - 18	21.2	23.5	21.0	20.4

* F_T now : According to International Standard ISO 2062

6. CONCLUSIONS

The conclusions drawn so far are :

1. Changes in the composition of the subsoil usually extend some 40 to 70 mm below the geotextile in bank protection works, sometimes even down to 120 mm or more.
2. The largest changes take place in a thin layer (order of magnitude 5 mm) directly under the geotextile.
3. At the majority of sites investigated, the geotextile was found to be clogged, sometimes to a very large extent. This aspect should therefore be taken into consideration in the design.
4. Appreciable uplift pressures may result due to blocking of the upper soil layer. Clogging of the geotexile has not been the reason in any of the sites investigated, because its permeability is and remains larger than that of the subsoil.
5. Geotextile clogging with materials containing iron compounds shortens its lifetime.
6. The location of the geotextile (above or at water level) does nog affect the degradation of its properties.

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The Development and Application of Geotextiles on the Oosterschelde Project

Développement et application de géotextiles du project de l'Oosterschelde

A number of estuaries are located in the south western part of the Netherlands. Since the flood disaster of 1953 all these sea inlets but one have been closed (the Delta Project). The last inlet is the 9 km wide Oosterschelde, which for environmental reasons will be shut off from the sea only in heavy storm conditions by a storm surge barrier (the Oosterschelde Project). For the large areas of scour protection of the Delta Project and specially the Oosterschelde Project new techniques have been necessary. This paper reports the development and the application from the conventional mattress, the semi-conventional mattress and the permanent ballast mattress to the fixtone mattress, the block-mattress and the gravel-sausage mattress. The storm surge barrier in the Oosterschelde will be built up from prefabricated elements, which will be placed in the channels by special barges. The piers will be placed on prefabricated filter mattresses, built up from geotextiles and holding natural filter material. Around the foot of each pier special gravel bags are suspended. These last two constructions are only possible because of the use of geotextiles.

Au sud-ouest des Pays-Bas se trouvent un nombre de bras de mer, qui après le raz de marée en 1953 furent fermés. Le dernier bras de 9 km de large est l'Oosterschelde qui pour raison de sauvegarder le milieu naturel, sera fermé par un barrage anti-tempête. Des nouvelles solutions comme protection des fonds sont exigées pour le Plan Delta et spécialement pour les travaux dans l'Oosterschelde. Ce rapport traite le développement de ces nouveaux revêtements, depuis le matelas de fascines croisés, par le matelas à simple couche de fascines, jusqu'au carapace souple, comme le paillasson en empierrement asphaltique, le matelas de blocs en béton et le matelas de bourrelet en gravier. Le barrage anti-tempête dans l'Oosterschelde sera construit par la pose d'éléments préfabriqués dans les chenaux au moyen de bateaux spéciaux.Les piles seront posées sur les matelas filtre, composés de géotextiles et remplis de matériaux naturels filtrants. Des bourrelets spéciaux bourrés de gravier sont mis autour du pied de chaque pile. Ces deux dernières solutions étaient uniquement possible grâce à l'application de géotextiles et le développement de techniques totalement nouveaux.

1. PROJECT REVIEW

1.1 Introduction to the Delta Project

In the history of the Netherlands there have been many floods: this has been inevitable, as so much of the country - particularly in the southwest - is low-lying. The rivers Schelde, Maas and Rhine flow through a combined delta to the sea, and the coastline is complex with deep seawater channels (Fig. 1).
In the past flooding has been controlled by a system of dikes, however this was inadequate in February 1953, when gales and a spring tide combined to breach the dikes in many places in a disaster reminiscent of the floods of earlier centuries. The number of deaths rose to 1,850 and the losses in livestock, buildings and agricultural land were beyond quantification.

A special commission quickly set to work to discover what measures were necessary: it devised a plan, called the Delta Plan, which involved closing off the Veerse Gat, Haringvliet, Brouwershavense Gat and Oosterschelde estuary. Working in that order, from the smallest to the largest, has enabled the engineers to learn by experience. The closure of these inlets shortens the coastline of the Netherlands by seven hundred kilometres.

1.2 The Oosterschelde Project

The Oosterschelde Project was started in 1967 with the building of harbours, with quays, jetties and yards. The actual construction of the 9 km dam started in 1968. Three construction islands, Roggenplaat (1969), Neeltje Jans (1970) and Noordland (1971), the Geul dam section (1972) and Schouwen (1973) abutments were built on the shallower parts of the river bed. By the end of 1973 5 km of dam had been completed. According to the plans, the remaining three channels, Hammen, Schaar and Roompot, should have been "plugged" with quarrystone and slag, covered with concrete blocks, gravel and sand between 1974 and 1980. The intention was to dump the concrete blocks from cableways.

1.3 Safety versus the environment

In the meantime, by the early Seventies, the plans to close the Oosterschelde had become the subject of heated discussions. A number of groups, opposed to complete closure, began to state their views more and more vigorously. They argued that the area could be sufficiently protected by raising existing dikes, and that the Oosterschelde should be kept open in order to preserve the existing tidal environment with its shellfish beds. Supporters of complete closure pointed out that this was the only way to guarantee the complete safety of the land in the area. The discussions led the Minister of Transport and Public Works to appoint a committee whose terms of reference were to report on all aspects of safety and the environment connected with the Oosterschelde Project. The committee issued its report on 1 March 1974. The conclusion was that in the interests of safety and of the environment, it would be best to build a storm surge barrier across the Oosterschelde which could be closed when necessary. Following the publication of this report, the DOS Consortium and the Public Works Department put forward other technical proposals for the design of the storm

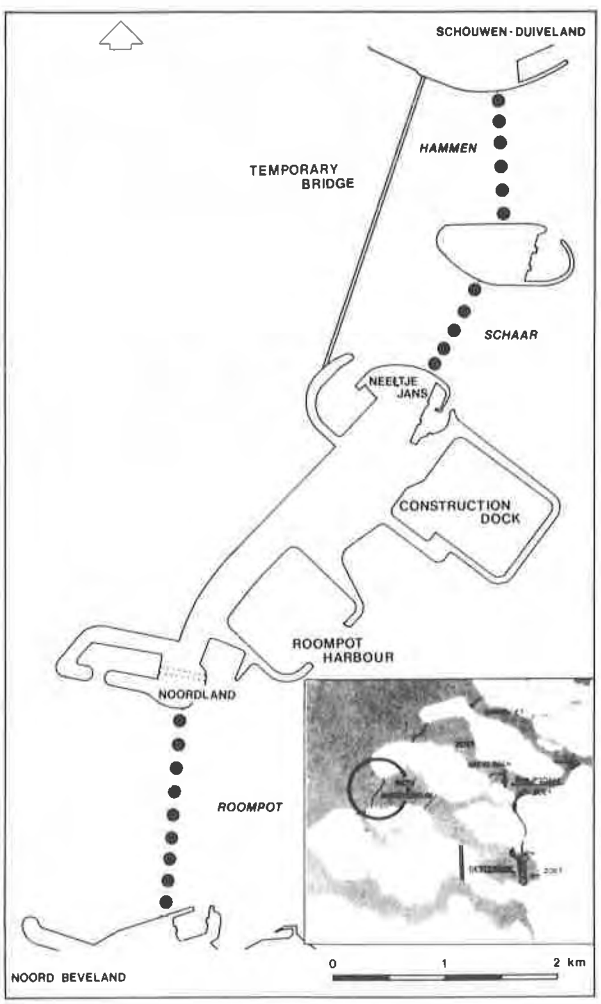
surge barrier, taking into account three conditions laid down by the Government:
a. it must be technically feasible
b. it must be completed by 1985
c. the costs must be within certain limits

Designs were made for a triple storm surge barrier in the remaining channels, Hammen, Schaar van Roggenplaat and Roompot. The designs all had one thing in common: the barrier was to be prefabricated because construction cofferdams could not be built in the channels, since they would have temporarily closed the estuary to the detriment of the environment (Fig.1).

1.4 The selected design

The design finally selected, was for a storm surge barrier consisting of monolithic piers, the seabed between them being raised by a sill construction of quarry stone and threshold beams. The piers, the sill and the beams together form the frame within which steel sliding gates can be raised and lowered. During normal weather conditions the gates will be kept raised so the water can pass freely through the barrier and thus preserve the tidal environment. During gales the gates will be lowered (Fig.2).

Fig. 1 : The Oosterschelde Project.



The size of opening is designed to allow an average tidal range at Yerseke of 2.70 m (77% of the existing range). An effective opening of 14,000 m^2 is needed to achieve this. The storm surge barrier will be over 2,800 m long, with 63 closable openings and a total of 66 reinforced concrete piers: 16 in the Hammen, 17 in the Schaar van Roggenplaat, and 33 in the Roompot. The piers have baseplates of 25 x 50 m, their heights vary between 35 m and 45 m and they have a maximum dry weight of 18,000 kg. The distance between the piers, centre to centre, is 45 m.

2. SCOUR PROTECTION

2.1 Development

For centuries scour protection in the Netherlands has been made of wood (willow, osier wood). However for the Delta Project which requires enormous areas of scour protection, the use of protecting mattresses of wood was not feasible. First of all there was a shortage of wood, furthermore the special skilled labour to make the wood mattresses was not available in such a short time. Also the stones required to sink the mattresses are very expensive and sinking is labour intensive. Finally ship-worms flourish in the Delta waters and the mattresses would be attacked by these worms reducing their durability considerably. Therefore it was necessary to develop new types of more suitable scour protection.

Initially, at the end of the late Fifties, an attempt was made to reduce the use of wood in mattresses by using plastic layers and steel wire-mesh; only the fascines were still of wood.

This mattress was rolled on to a tube, shipped to the exact position, and lowered down onto the sea-bottom by unrolling and sinking by dropping stone onto it. These mattresses are the first types which were sunk under control as opposed to the former types the sinking of which was not controlled. In addition these mattresses are the first types of scour protection which used geotextiles. Because there was insufficient knowledge about the permeability of the geotextiles this type of mattress did not work effectively because it was impermeable to water.

The next step in the development was the use of relatively light nylon geotextiles instead of plastic layers and steel wire-mesh. For the Veerse Gat closure a nylon scour protection was used consisting of a nylon geotextile fabric to which are connected nylon geotextile tubes filled with sand. These mattresses were placed, using a vessel. The vessel towed the mattress over the scour as the scour protection was unrolled.

2.2 Semi-conventional mattresses

However this nylon scour protection, referred to in the previous section, was not entirely satisfactory and on several occasions the water permeability was insufficient. Therefore the search for a more suitable geotextile continued. Eventually a geotextile was found, made from a heavy polypropylene woven fabric weighing about 750 g/m^2. For this mattress a small amount of wood was used as stiffening and to give it buoyancy. The wood also protects the geotextile during stone dumping operations. In addition the method of placing also completely changed. The new procedure worked faster, and was much more accurate. The mattress was towed by a tug between two pontoons. A heavy weight, placed on one end, pushed the mattress down to the sea-bed. Then a stone-dumping machine was positioned above the mattress and stone was dumped on to the mattress so that the whole protection was correctly placed. This method is also possible if there is a small current flow. To resist the forces during the sinking procedure a geotextile is required with a breaking load of 150 kN/m; the conventional wooden mattresses had only 20 kN/m breaking load. The edges of the mattress are protected against instability in currents by concrete blocks weighing 800 kg/m.

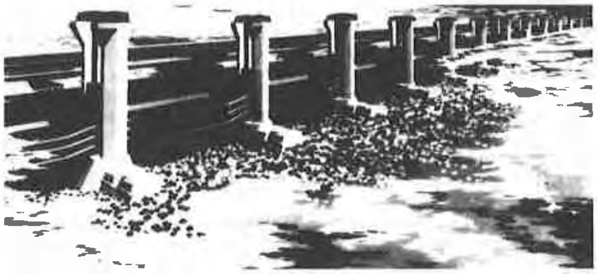


Fig. 2 : An artist impression of the Oosterschelde Storm Surge Barrier.

This construction worked satisfactorily and is still used on a large scale for instance as a shore protection for the artificial islands in the Oosterschelde Project. In recent years the construction has been further improved by testing and evaluation, so that it now has good water permeability, is impermeable to sand and is very durable in water.

2.3 Bottom protection with permanent ballast

The semi-conventional mattress was much better as a filter than the conventional protection of osier wood. The geotextile in the mattress combines great strength and filter properties. But the ship-worms which flourish in the Oosterschelde, also attacked the wood in the semi-conventional mattress. Other materials are required, particularly if the mattress is to be used for more than one year and on slopes. Also a lot of labour would be required for the large areas of scour protection in the Oosterschelde as the semi-conventional mattress is labour intensive. Therefore the Delta Department of the Ministry of Public Works and the contractors searched for another form of scour protection which would not have these problems. Starting with the semi-conventional mattress and using the same type of polypropylene geotextile they developed a scour protection with permanent ballast which involves no osier wood and only on some occasions a small amount of stone. This protection is completely factory-made and is placed by a special vessel.
Three new types of bottom protection with permanent ballast have been developed:
- the fixtone mattress
- the block-mattress
- the gravel-sausage mattress

The first two types are used on the Oosterschelde Project.

2.3.1 The fixtone mattress

a. General description

The mattresses are 17 m wide and will vary in length from 150 to 200 m. They are composed of fixstone applied to a permeable polypropylene geotextile in two layers, total thickness 0.12 m. The mesh size of the filtercloth is suited to the average sand-grain diameter in the Oosterschelde, $O_{90} \leq 0.150$ mm. The mattresses are weighted at the ends with concrete blocks so that the edges will be able to follow the configuration of the sea-bed. Since the mattresses are pulled forward during the production process and stresses are exerted on them while they are being laid on the sea-bed, they are provided with 18 cables and wire-mesh reinforcement. The diameter of the cables is directly related to the stresses that will be exerted on the mattresses and these stresses in turn depend on the depth of water in which the mattresses are to be laid. The cables are attached to the reinforcement by clamps. The concrete blocks and cables are connected to the permeable sheeting (Fig.3).

b. Fixtone

The product fixtone is 80% crushed stone (20-40 mm) and 20% asphalt mastic. The composition of asphalt mastic is 60% sand, 20% limestone filler, 20% asphalt bitumen. The fixtone is made in two stages in the same mixing plant; first the asphalt mastic is produced after which it is mixed with the crushed stone at a temperature of 110°C.

c. Fixtone mattress manufacture

Manufacture of the mattress takes places on board the of vessel. Sand and crushed stone are brought to the vessel the "Jan Heijmans", in hoppers from stock piles. Reels of geotextiles, 4.75 m wide, and the wire-mesh reinforcement are transported to the floating factory and there placed in bearings.
The manufacturing procedure is as follows. The strips of geotextiles are unwound from their reels and the edges are fastened together. The sheeting is then drawn through under a spiked drum until a length of about 6 m rests on the production platform. When the position of the geotextiles on the platform has been checked, the end ballast of concrete blocks is placed on the leading end of the geotextile. The concrete blocks are then attached to the geotextile and to the steel-wire cables of the mesh reinforcement.
The balance beam, used when placing the concrete blocks, is then put on the geotextile and fastened at one end to the steel wire cables and at the other to one of the winches on the vessel. By pulling on this bar, while the spiked drum continues to exert pressure on the geotextile, it is possible to tighten the geotextile and the wire-mesh reinforcement over the entire width of the mattress. When these operations have been completed, a start can be made on applying the fixtone.

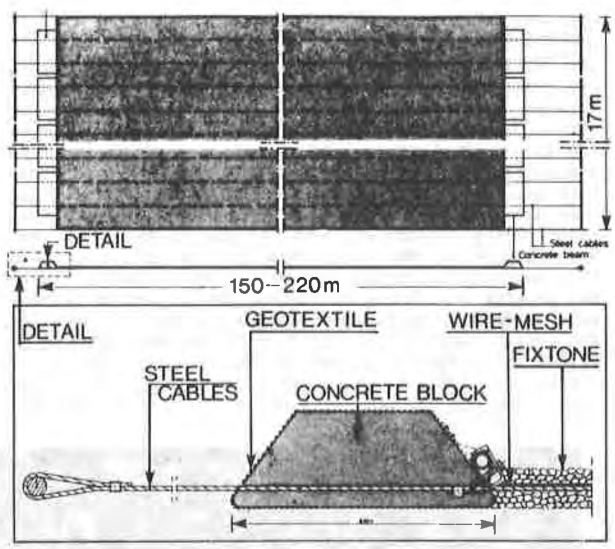
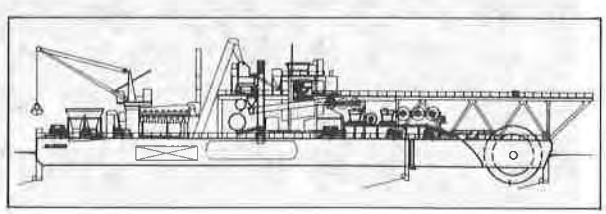


Fig. 3 : Shape and construction of the fixtone mattress.

Fig. 4 : The "Jan Heijmans" - mattress layer.



Fixtone is applied in two layers, the bottom layer by using the spreading trolley nearest midship, and the top layer by using the other spreading trolley, the latter after the wire-mesh reinforcement has been placed on the first layer. The spreading worms attached to the trolleys are fed with fixtone from the mixing plant. During these continuous operations the mattress is drawn forward along the production platform by the winch, using the bar of balance beam. When, however, the leading edge of the mattress has nearly reached the sloping part of the platform the matting drum takes over from the winch. The bar is removed and replaced by the tail beam. The drum is rotated by jacks. In this way the mattress is drawn towards the drum and then wound on to it. When the end of the mattress has nearly reached the sloping part of the production platform, the end ballast of concrete blocks is placed on the geotextile. When the last fixtone has been spread on the sheeting the mat is wound further onto the drum. The maximum speed at which the mattress moves during production is 1.5 m per minute, corresponding to a drum speed of 3 to 4 revolutions per hour.

The braking force exerted on the sheeting by the spiked drum during production of the mat varies between 450 and 600 kN. The horizontal movement of the mat on the platform is thereby adjusted to the rotary movement of the mat on the drum. The relationship between these movements is governed by the requirement that the mat must not sag between the sloping part of the production platform and the drum to such an extent as to cause a crack in the fixtone. A minimum radius of curvature of 4 m has accordingly been specified.

It takes about 10 hours to make and roll up a 17 x 200 m^2 fixtone mattress weighing 850,000 kg, 4 hours being required to make and spread the fixtone and 6 hours for incidental operations such as conveying materials and applying the edge ballast.

d. Mattress laying

The mattresses are laid at low water at about the turn of the tide. Then the variations in the direction and magnitude of pressures exerted on the mattresses by the current while they are being laid in position will not be too great. At least two hours before the tide turns at low water the "Jan Heijmans" (Fig.4) must therefore have been manoeuvred into the correct position, using the hauling system of anchors, wires and winches and the position-finding facilities, and the anchor beam must have been secured to the wire cables of the mattress. If at this moment the current velocity of the outgoing tide is not greater than 0.5 m per second and the set of the current is considered favourable, the mattress is unrolled. The correct position of the anchor beam must then be obtained by manipulating the winches and its position is checked with the echo-sounding equipment. When the anchor beam is in the correct position on the sea-bed, the mattress can be unwound; while this is being done, the vessel is pulled to the speed of unrolling, since at this stage of the work the radius of curvature of the mattress must not be too small and excessive force must not be exerted on it, because this might result in damage to the mattress or the anchor beam being dragged from its position on the sea-bed.

When the tail beam has reached the bottom, this beam is released pneumatically from the mattress and pulled up again.

The vessel is then hauled backwards until the forepart has been manoeuvred into a position more or less vertically above the anchor beam. The anchor beam is then disconnected from the mattress by means of a release cable after which the beam is pulled up. Depending on weather conditions, swell and waves, it takes about three hours to lay a fixtone mattress. The entire operation is monitored continuously by echo-sounding equipment.

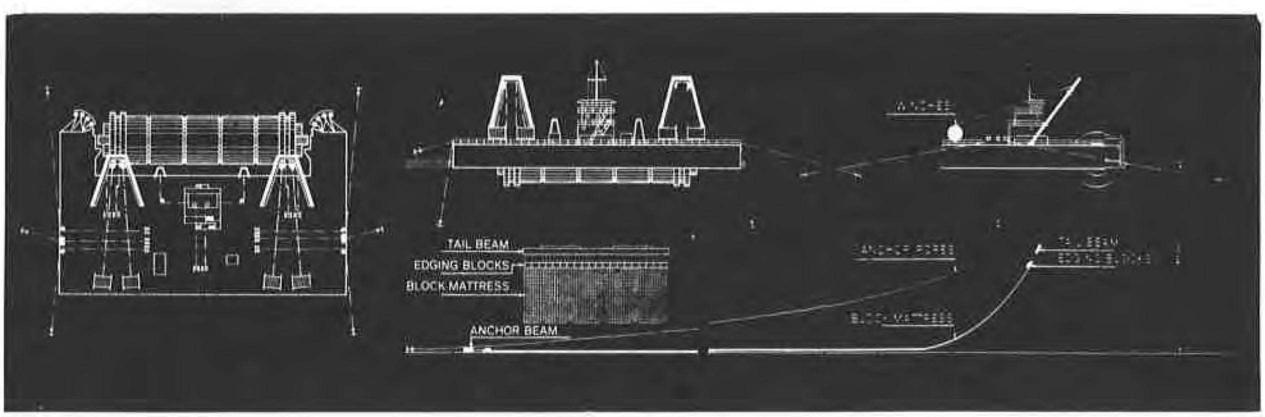
2.3.2 The block mattress

a. Mattress composition and manufacture

The base of the block mattress is formed by a geotextile (polypropylene) specially developed for the purpose. The geotextiles have been subjected to extensive tests and the result is a geotextile weighing about 1200 g/m^2 having a tensile strength of about 250 kN/m. In addition, this fabric satisfies all requirements as regards permeability to water, impermeability to sand ($O_{90} \leq$ 0.300 mm) resistance to wear and biological and chemical resistance. The block mattress factory was built at the Sophia workharbour, on the island of Noord-Beveland. At the beginning of the production line six rolls of the geotextile, adjacent to each other, are reeled off and immediately sewn together automatically, thus forming a strip 30 m wide. In the next phase plastic pins are forced into the material in a pre-arranged pattern. The pins serve to anchor the concrete blocks that are to be placed on the mattress at a later stage. Thus, each block mattress, with an area of 6,000 m^2 is produced with 72,000 pins. These operations are carried out in eight minutes.

The mattress is then moved a distance of 2 m and the production cycle starts all over again. The mattress - complete with pins - is passed underneath an array of block moulds 2 x 30 x 0.17 m^3, which are then lowered. At the same time the moving floor, made up of pallets, slides underneath the matting. The block mattress is supported by the moving floor for the duration of the manu facturing process. A pouring machine moves in transverse direction from the concrete mixing plant to fill the

Fig. 5 : Block-mattress laying.



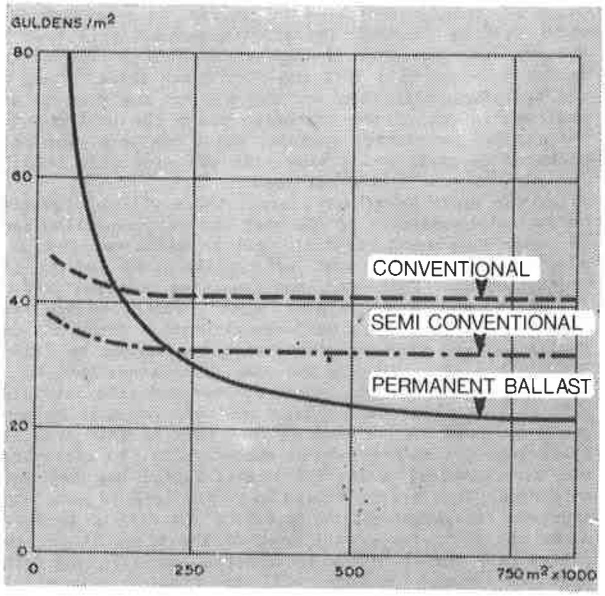


Fig. 6 : Scour protection.

moulds. High-frequency vibrators - placed in the moulds - ensure that the concrete acquires its proper density. As it moves back, the machine tops up the moulds. When all moulds have been raised the mattress is moved along another 2 m to a 60 m long steam room, where a temperature of between 50°C and 60°C and a very high humidity are maintained. It takes about four hours for the concrete to acquire the necessary strength.

After the block mattress has left the steam room, the moving floor is taken away. It takes 15 hours to make one mattress, 200 m long, 30 m wide, studded with 18 thousand concrete blocks and weighing 12 hundred tons in all. An expansion pit takes up the slack which forms due to the difference between the rate at which the mattress is manufactured and that at which it is wound onto the floating drum (measuring 10 m in diameter and about 42.50 m in length). Steel ropes are used to secure the heavy anchor beam to the end of the mattress. Supported by a hydraulically operated steel floor the mattress with the anchor beam attached, is wound onto the drum. After the anchor beam has been locked to the drum, the cylinder with block mattress is towed away and an empty cylinder takes its place. In this way a reasonable continuity is assured both in manufacture and in the sinking operations.

b. Mattress laying

A special method has been developed for sinking the mattresses on location and this can be used in depths of water down to about 40 m. The cylinder with the block mattress is towed to an accurately positioned pontoon and locked in a kind of trap. Immediately after the start of the ebb-tide the mattress is unwound from the cylinder in a direction parallel to the current. Initially the cylinder is rotated with the aid of winches. As soon as the mattress hangs down far enough, the lowering operation is commenced. At the same time the pontoon is shifted at a maximum speed of 4 m per minute in the same direction as the current. The actual sinking operation takes about an hour and a half. A heavy bar, accurately positioned on the bottom by means of anchors, ensures that the block mattress is placed on the bottom in the specified position.

During the sinking operation the position of the suspended part of the mattress between the roller and the seabed is constantly checked by means of a Profiling Sonar. In this way it is possible to correct any deviations from the required position that occur through the current striking at an angle or on account of the mattress being lowered either too slowly of too quickly. At the end of the block mattress a tail beam is fitted (Fig.5). If everything has proceeded according to plan, the anchor beam and tail beam are detached , secured on the empty cylinder and towed to the factory.

c. General

Quite a lot experience has been obtained in the manufacture of the block mattresses and from sinking operations. Results have come up to expectations, but research is being continued with unabated effort. For example, the technicians have recently succeeded in improving the geotextile even further by adding an extra non-woven geotextile. This has made it possible to control the sand permeability of the filter to an even higher degree of accuracy than hitherto, so that it can be adjusted to local conditions and specific requirements.

2.3.3 The gravel-sausage mattress

The gravel-sausage mattress has a geotextile base to which sausage-like geotextile tubes filled with gravel are attached. This mattress is easy to reshape and is therefore useful also on irregular bottom conditions. The geotextiles used are the same as those used in the fixtone mattress and the block matttress described above. For several reasons the gravel sausage mattress is not yet used in practice on a large scale.

2.3.4 Cost aspects of scour protection

Because of the very large areas of scour protection, required for the Oosterschelde Project the cost per square metre is very important. On Fig.6 you can see an indication for the price of several types of scour protection. For conventional and semi-conventional scour protection the starting costs are low and they can be used for small projects. But when more than about 250,000 m^2 are required permanent ballast scour protection is cheaper. This is because of the investments in special equipment, factory/s and vessels. On the Oosterschelde Project a total of about 5 million m^2 of scour protection was required and so the new permanent ballast scour protection provides a very economical solution.

3. SCOUR PROTECTION FOR THE STORM SURGE BARRIER

For the storm surge barrier in the Oosterschelde, the sandy bed must be protected to prevent it being washed away before, during or after placing of the piers and also during closure operations. About 4.5 million m^2 of block mattress scour protection are required. In the last 6 years new demands have arisen for the polypropylene geotextile of the block mattress, and it now has been developed with an expected lifetime of up to 200 years. More details about this subject are given in the paper "The long-term thermo-oxidative stability of polypropylene geotextiles in the Oosterschelde Project" (1). The strength must be garanteed and the quality also has to be to a high standard. Details on the mechanical research on geotextiles are given in the paper "Analysis and experimental testing of the load-distribution in the foundation mattress" (2). Over the past few years protective mats have been placed on either side of the axis of the barrier to form a strip between 450 and 650 m wide.

4. CONSTRUCTION IN THE MOUTH OF THE OOSTERSCHELDE

4.1 General

A storm surge barrier is to be built in the three channels Hammen, Schaar van Roggenplaat and Roompot at the mouth of the Oosterschelde. The best way to construct the barrier is to place concrete piers in the channels, the base of each pier being firmly embedded in rubble, and then to close off the remaining parts of the wet cross-section by means of sliding steel gates. Construction work has now begun: the piers and sill beams are being constructed in a special dock at the mouth of the Oosterschelde, from there they will be transported to their final positions in the three channels.

4.2 Foundations

The sea-bed beneath the storm surge barrier must have an enormous bearing capacity and therefore the materials of the sea-bed, fine sands, silts and muds which are poor foundation materials, are being removed by dredging and replaced by better quality sand. To further strengthen the sea-bed it must then be compacted to a depth of up to 15 m by a specially built vessel, the Mytilus (Mussel). The vessel has four steel vibrator tubes which compacts the loose sand and thus increases its bearing capacity. To make sure that the base on which the concrete piers rest, will withstand the ravages of time, an extra layer is to be added in the form of prefabricated filter mattresses, which are manufactured in the Roompot harbour and placed in position by a special pontoon, the Cardium (Cockle).

4.3 Function of the filter mattress

The mattresses consist of a filter construction of three graded layers and are impermeable to the sand of the sea-bed. They form a carefully composed transition from the sea-bed to the layers of rubble which will be placed around the base of the piers. The mattresses will hold the underlying sea-bed sand in position despite the pressures to which the barrier may be exposed, and at the same time they will provide sufficient drainage for water This latter function is of prime importance, ensuring that the upper layers of sand are not washed away should the barrier be exposed to severe current action or heavy waves. The mattresses, approximately 42 m wide and 200 m long, weigh about 5.5×10^6 kg each. In order to prevent the mats from being damaged before and during the installation of the piers, a second smaller mat, 31 x 60 m is used immediately below the base of the pier. In view of the very high standard of requirements regarding both quality and positioning of the filters, it was decided, after various ways of production had been considered, to prefabricate the mats and to transport them to their location.

The same procedure, used before when the block mattress were so successfully laid, will be followed. The filter mattresses will be assembled on shore and then wound on to a gigantic floating cylinder positioned in front of the assembly plant. A more detailed description of the filter mattress is given in the paper "The Oosterschelde filtermattress and gravel bag, two large-scale applications of geotextiles" (3) and on the mechanical requirement, in the paper "Analysis and experimental testing of the load-distribution in the foundation mattress"(2).

4.4 Laying filter mattresses on the Oosterschelde sea-bed

The filter mattress will be placed onto the sea-bed in water, with a maximum depth of 35 m, by the specially built rig, the Cardium. This vessel is also equipped with a dust-pan suction nozzle which will dredge and level-off the Oosterschelde sea-bed to the correct depth immediately prior to mattress laying.

4.5 Pier positioning and installation

A lifting pontoon, the Ostrea (Oyster), is being built to lift, transport and position the concrete piers. The piers are being constructed in the dry, in a special dock which will be flooded. The lifting pontoon will then be brought into the dock through an opening in the circularing dike and will lift the pier a few metres clear of the bed. Tugs will then tow the pontoon and pier to its position in one of the channels, where the pontoon will be attached to another pontoon, which has been carefully anchored in position. At low tide the pier will then be lowered onto the filter mattress.

After the piers have been placed, there will be cavities between the underside of the pier and the foundation bed. In order to ensure stability and to minimise pier displacement, the piers must be rigidly connected to the foundation bed. Therefore, any remaining cavities will be filled, from inside the pier, with a cement bonded filling material such as a sand-cement mortar. Since the cavities between pier and foundation bed cannot be filled immediately after placing the pier, provisions have to be made to prevent the settlement of sand and silt particles into the cavaties. In addition the cavities must be properly enclosed with a kind of form work so that grouting operations can be carried out successfully. To meet these two requirements, a special gravel-filled bag has been developed. This gravel filled bag (sausage) is made from a porous sand-tight synthetic fabric and will be attached along the perimeter of the foot of the pier. It will be filled with gravel in the construction dock. During transport the gravel sausage will be tied-up temporarily against the pier for protection. After placing the pier on the foundation bed, the fastenings will be released, causing the gravel bag to fall onto the foundation bed. In this way, the two functions of the bag, seal and form work are realised. This item is discussed further in the paper "The Oosterschelde filter mattress and gravel bag, two large-scale applications of geotextiles" (3).

Finally the pier base will be firmly embedded in several layers of rubble. Once all the piers are in position, the various parts of the bridge, the gates and the upper beams and girders will be added and the pier dam will be complete.

5. CONCLUSIONS

For the Delta Project and in particular the Oosterschelde Project a number of completely new scour protection methods have been developed and implemented, all using geotextiles. Two methods, the filter mattress and the gravel bag, are only possible because of new geotextiles. Much research has been carried out using both large scale and laboratory tests. Totally new design methods had to be developed because in most cases it was necessary to design the scour protection and other constructions from first principles based on the functional requirements. With this approach it was possible to meet the needs of the local requirements and the circumstances. In order to verify the quality of the geotextiles used and other products a new certification system was developed, based on an internal quality control system of the manufacturer. Good experience has been acquired on the use of this system.

On the basis of this approach to design, execution and quality control the use of the relatively cheap geotextiles will lead to effective constructions all over the world.

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The Oosterschelde Filter Mattress and Gravel Bag

Le matelas de fondation et bourrelet de gravier de l'Oosterschelde

Main feature of the Oosterschelde Storm Surge Barrier is that the construction will be in open sea channels using prefabricated elements. The foundation of the piers must prevent erosion and water pressure generation in the sandy soil. A layered granular filter has been chosen, which is assembled into a mattress away from the site. This mat is transported and sunk on location. A variety of geotextiles was necessary for the mattress. A description is given of the choice of geotextiles and the composition and transport of the mattress.

The space between pier and mattress will be filled with a mortar. To protect this cavity against sand and silt, a seal is made around the pier base, which also acts as mortar formwork. The seal consists of a geotextile unit ballasted with gravel. The design, research and assembly of this gravel bag is also described.

The mattress and the gravel bag are specific applications of geotextiles, which perhaps will find a continued use in civil engineering.

Charactéristique principale du barrage anti-tempête de l'Oosterschelde est l'empilage en pleine mer par éléments préfabriqués. Comme le sol d'assise des piles est sableux, il y a une condition d'éviter l'érosion et les sous-pression sous l'ouvrage. Les matelas de fondation adoptés se composent de couches granulaires formant filtre. Ailleurs ces couches sont reliées entre elles de façon à constituer un matelas, transporté et immergé à son emplacement. La réalisation de ces matelas necessite une gamme de géotextiles. Une description sera donnée de la composition du matelas.
La cavité entre la pile et le matelas sera injectée de mortier. Autour du pied de la pile un bourrelet est confectionné afin d'empêcher la venue de la vase et du sable Il sert aussi comme coffrage du mortier. Ce bourrelet consiste en toiles de géotextiles bourré de gravier. La conception, la recherche et la fixation de ce sac seront décrites. Les deux solutions adoptées forment l'objet des applications spécifiques de géotextiles, qui peuvent avoir un large emploi dans le domaine du genie civil.

1. INTRODUCTION

The filter mattress and the gravel bag have in common that they will be used at the junction between pier and sandy soil. For the rest they are completely different items, with different purposes.
The filter mattress is described in this paper. The mattress is also subject of other papers of these proceedings (1),(2),(3).
The contribution concerning the gravel bag will be limited mainly to this paper. Therefore design and research are mentioned here in addition to the detailed description (1), (3).

2. THE FILTER MATTRESS

2.1 Brief description of the system

The sill and the foundation bed have been designed in accordance with the filter principle using granular materials, that is to say in layers, the size of material in each layer being coarser than the underlying layer until a stone size is reached which is sufficiently large to resist the current. Each layer is physically impermeable to the layer below, although water is able to pass through freely. The purpose of the foundation bed, as a part of this system, is to hold the fine Oosterschelde sand in place, despite the high water pressure gradient across the pier base. The impermeable filter below the pier is composed of, successively, a layer of sand, grain size 0.3-2 mm, a layer of fine gravel grain size 2-8 mm and a layer of gravel grain size 8-40 mm. On top of this gravel layer layers of stones are dumped .

The filter materials below the pier are not resistant to the current and it is very difficult to place them in the required layer thickness particularly in the Oosterschelde, where the soil is moving constantly and there is considerable natural sand transport. For this reason it was decided to prefabricate the filters in the form of a mattress on shore and then to place them on the sea-bed with a special pontoon.

The mattresses had to be sufficiently thick to be able to distribute the loads involved satisfactorily. However for practical handling reasons the thickness had to be limited and in fact two mattresses are to be placed on top of each other. The upper mattress also protects the lower mattress in the installation stage. There are 66 lower mattresses, each 200 m by 42 m by 360 mm thick, composed of, successively, 3 layers, sand 110 mm, fine gravel 110 mm and gravel 140 mm. The 66 upper mattresses which will lie immediately below the base of the piers are 60 x 31 m by 360 mm thick, however unlike the lower mattresses, they are composed of three layers of gravel. The weight of the filter mattresses is approximately 600 kg/m^2 the total weight being, for a lower mattress, 5×10^6 kg and 1.1×10^6 kg for an upper mattress.

The foundation beds are prefabricated by packing the granular materials into a system of geotextiles. In addition to having to meet the same hydraulic specifications as the foundation bed, the composite mattresses have also to meet high mechanical handling requirements, for which the development of new materials has been necessary. Because of the large quantity, approximately 700,000 m^2, filter mattress assembly has taken about 2 years and because of the quality required, it was decided to use an automated

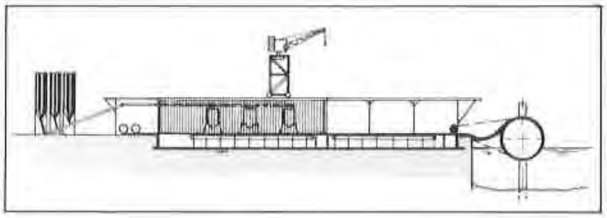


Fig.1 : Longitudinal section-mattress assembly plant.

production process. The filter mattresses are being assembled in a plant, then wound on to a floating cylinder and transported by a pontoon. The sea-bed is dredged and leveled to the correct depth and then the filter mattress is laid.

2.2 Fringe benefits

The fringe benefits, often established after prolonged research for the geotextiles used in the filter mattress, are defined by the hydraulic properties of the connected filter layers, the filter mattress placing operations and the method of production.
The hydraulic properties of the mattress are protection to the underlying sand and the water permeability, which have to meet high requirements in connection with the quality of the mattress. The operations with the filter mattress, such as winding onto the cylinder and sinking are dictated by its mechanical properties (2). The method of production has fixed the size and lay out. It will be obvious that the development of geotextiles to meet all specifications has demanded large-scale research.

2.3 Detailed specification of the filter mattress(Fig.2)

2.3.1 Supporting fabric (Fig.3)

The function of the supporting fabric of the filter mattress is to take the stresses and strains during winding onto the cylinder and the sinking operations. Because of the water depth of 35 m, in which the mattresses have to be sunk, and the high current force an ultimate design load of 800 kN/m is necessary in the longitudinal direction and 80 kN/m in the widthwise direction. For this purpose a polypropylene fabric has been developed, reinforced lengthways with steel wires, diameter 2.7 mm, and having a design load of 800 kN/m. This fabric is supplied on cylinders in lengths of 204 m for lower mattresses and lengths of 64 m for upper mattresses. The width is approximately 4.90 m. In order to achieve the required dimensions 9 widths are stitched together. In order to create enough strength the stitching seam is made with an Aramide stitching thread, which combines a small linear mass with a high strength.

Both outer widths are bent, in part, to shape the sides of the mattress. Polyester cord is used in this part to take the required strain, and not steel wires. During one week approximately 12,000 m^2 of this fabric, weighing approxemately 4,400 g/m^2, is produced with 3 special looms.

Fig. 2 : Lower mattress section.

To transmit the forces of the mattress to the steel structure of the cylinder, the front and back of the mattress are fitted to heavy steel beams, referred to as the tail and head beam. To connect the mattress to the tail beam wedges of synthetic resin are fitted on to the steel wires in the supporting fabric, a connection device similar to that used with steel cable sockets. The connection for the head beam has to cope with less load and therefore a wrapping clamp is adequate (Fig.4).

2.3.2 Intermediate fabrics

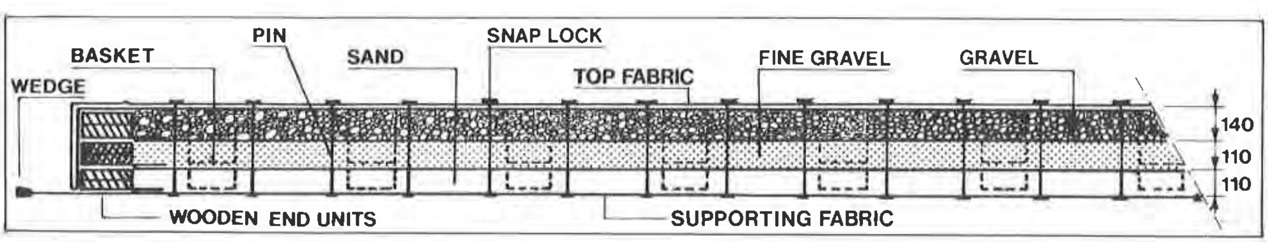
In order to keep the three filter layers apart in the mattress intermediate fabrics are used between each layer. In order to meet the sand impermeability requirement a spun-bonded fabric is inserted on top of the supporting fabric. The intermediate fabrics have to meet the same specifications of water permeability as the filter layers. Between the sand and fine gravel it is sufficient to use a polypropylene spun-bonded fabric, between fine gravel and gravel, however, a gauze fabric is used composed of polyethylene monofilaments. These geotextiles are supplied in lengths of 1000 m with width varying from 1.50 to 5 m. In order to achieve the required dimensions the lengths of geotextiles are placed next to each other with an overlap of 500 mm. In all approximately 2,300,000 m^2 of spun-bonded fabric is necessary.

2.3.3 The vertical dividers

Vertical dividers are inserted to prevent the filter material from shifting in a longitudinal direction and to guarantee the thickness of the filter layers during the winding and sinking operations of the mattress. U-shaped baskets are turned inwards to enclose a spun-bonded fabric, the height of a basket being equal to the minimum layer thickness of the filter material. In all approximately 1,500,000 m^2 of gauze fabric and 800,000 m^2 of spun-bonded fabric are necessary.

2.3.4 Top fabric

The top fabric is the outer cover of the filter mattress. So that the filter mattress can follow the curve of the cylinder this fabric has to meet in addition to a water permeability requirement high strain requirements. The fabric,specially developed for this purpose, can take 25% strain in the longitudinal direction with a strength of 100 kN/m and at the same time 15% strain in the width with a strength of 80 kN/m. The fabric is composed of polyamid warp yarns and polyester weft yarns woven in an open-work bond. It is wound onto cylinders, in widths of 4.90 m and lengths equal to the lengths of the filter mattress. The full width of the mattress is obtained by stitching sections of the fabric together. The total amount is approximately 750,000 m^2. On the side of the mattress the top fabric is connected with the supporting fabric by stitching.
To enclose the filter layers at the front and the back side of the mattress wooden end units are fitted to which the various geotextiles are stapled. These wooden units are of the same thickness as the layers of filter material, and are joined to each other with nails and to the supporting fabric with bolts. In order to facilitate under waterinspection of the mattress a pattern is painted on top of this fabric.



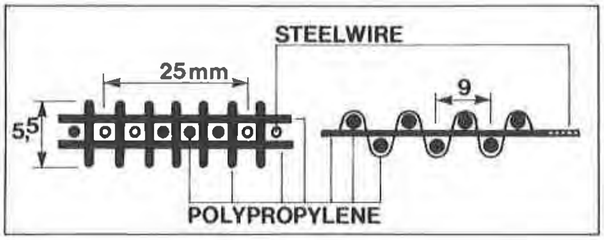


Fig. 3 : Supporting fabric structure.

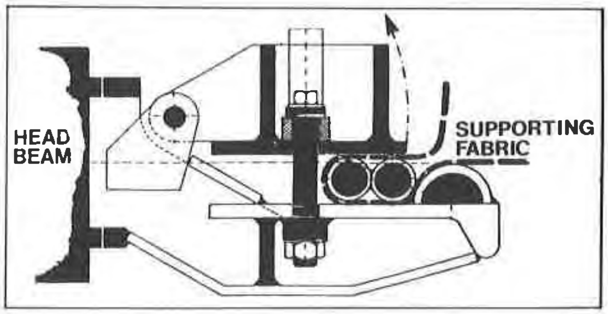


Fig. 4 : Wrapping clamp

2.3.5 Vertical joints

Steel pins are used as vertical joints between the supporting fabric and the top fabric to consolidate the layers together into a compact mattress. These have a diameter of 6 mm, and have on one end a nail head and on the other end a sharp point; a washer is used on the lower side. The connection on the upper side consists of snaplocks, with a minimum gripping force of 5 kN. The pins are also used to prevent the baskets moving. Eight pins are used per m^2, so in total about 6,000,000 pins and associated snaplocks and washers are required.

2.4 Detailed description of the production process(Fig.5)

2.4.1 General

The assembly plant consists mainly of a production hall and a discharging wharf. The production of the mattress in the assembly plant is independent of the mattress-laying process in the Oosterschelde and two transport cylinders are used for this purpose. The production capacity is determined by the progress required at the construction site of the barrier. This means, in fact, that on average one filter mattress must be completed every 40 hours. During the assembling process the mattress is moved underneath stationary machinery. For this purpose the floors of both assembly plant and discharging warf consist of 9 synchronised chain conveyors driven by electric motors. The assembling process takes place at ground level and has been phased in such a way that a two metre strip of mattress can be assembled every ten minutes. The whole mattress is then moved forward a further two metres by 9 coupled chain conveyors at the same height towards the discharging wharf. The conveyors inside the production hall can operate independently of those on the discharging wharf, so that the assembling process can be disconnected from the operations taking place on the discharging wharf. The operation machinery of the plant and discharging wharf is housed in a large concrete basement which also contains some machinery for parts of the assembling process. The time required to assemble a mattress of 200 m is 24 hours; the time to handle the same mattress on the quay is about 10 hours.

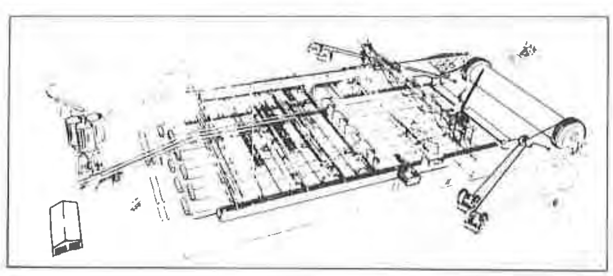
2.4.2 The production hall

The process starts with the pouring of the wedges independently of the production process. This operation is carried out in the wedging shed which is situated apart of the assembly plant. From here rolls of supporting fabric are then taken to the west side entrance of the plant by crane and mounted ready to be decoiled. The 9 widths of the supporting fabric are then stitched together to an overall width of 42 m by 8 sewing machines housed in the basement. To avoid storage within the plant the baskets are manufactured at the same time as the filter mattresses in the basket hall, north of the mattress assembly plant. The basket hall houses three machines which mould the steel netting and cut it to size. Both top edges of the U-shaped baskets are turned back 180° to enclose a geotextile. After having been placed in position, ready for use, the baskets are transported to the assembly plant by 4 basket transporters and placed in the correct position on the mattress. Three such operations operate parallel to each other.
The granular materials are transferred from the storage depots to a hopper by a front loader and transported to the appropriate silos via a movable chute and a conveyor. Three independently operating conveyor systems feed the materials into three moving distributors and four stationary hoppers, the latter being used particularly for the sides of the lower mattresses. When the material has been distributed on a two metre strip of mattress, the moving distributors are refilled, which takes only two minutes, whilst the mattress moves on. The adjustable vibrating chutes below the hoppers ensure an even layer of filter material, unaffected by the inevitable sagging of the supporting structure and conveyors. The sides of the various fabrics which, during the assembling process, have been bent vertically are stitched together with sewing machines before the pins are injected through the mattress. Widths of the top covering are stitched together as they are being unrolled vertically. The top covering is then transferred to a horizontal position and both ends are stapled on to the wooden end units.
The equipment for injecting the pins consists of an upper and an under carriage. The under carriage runs on rails in the basement and hydraulically injects 75 pins every 5 x 2 m area of the filter mattress. A detection and positioning system ensures that the injected pins do not hit the steel wires in the supporting fabric. The synchronised upper carriage, running on a steel structure above the mattress, meets the 75 pins, fixes bowl-shaped snaplocks on to them and cuts off the sharp ends. By this method it is possible to inject, secure and cut 700 pins during the 10 minute period when the mattress is stationary.

2.4.3 Discharging wharf

As soon as the beginning of the filter mattress approaches the fixed quay floor at the end of the second chain conveyor, it is connected to the tail beam by means of the fitted wedges. The tail beam itself is coupled to the filter mattress cylinder by steel cables. In the final stages of winding there is only a small part of the mattress remaining on the quay. To prevent this end of the

Fig. 5 : Mattress assembly plant.



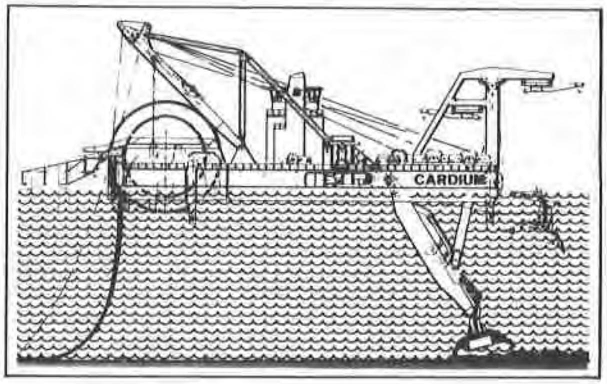


Fig. 6 : Pontoon Cardium

mattress from suddenly sliding off the quay, and to control the whole winding process, a braking system has been devised. This system consists essentially of a heavy steel bridge construction. Two heavy winches have been positioned on the bridge, each capable of delivering a braking force of 1500 kN.

Balancing and loading winches.In order to keep the cylinder in balance during winding two balancing winches have been installed. The winding operation itself is carried out by two loading winches. The winding process proceeds intermittently, each two metre length of mattress, which is completed and fed through every ten minutes, taking only one minute to be wound on to the cylinder. When the mattress is being wound on to the cylinder, the load on each of the two winch cables increases progressively to a maximum of 2250 kN, relative to the balance, mattress radius, mattress weigth and water level.
The whole plant and quay is bridged by a portal crane, which has a lifting capacity of 1200 kN, and is used mainly for operations on the quay, such as the handling of the head beam, which weighs 1100 kN and the tail beam, which weighs 850 kN. After the mattress has been wound the floating cylinder is disconnected from the discharging warf and transported by tugboats to the Cardium.

2.5 The Cardium (Fig.6)

A special vessel has been built for placing the mattresses. This special vessel, the Cardium, with a length of 72 m and a width of 80 m will simultaneously smooth out the sea-bed by dustpan dredging, and place the mattress by unwinding it from the floating cylinder. The cylinder is connected temporarily to the rear end of the vessel, where it is accomodated in a recess. The cylinder has an unloaded diameter of about 15 m and a characteristic width of 46 m between "shields". The cylinder is prevented from untimely unwinding by wires arranged around it.
At the front of the vessel, a recess accomodates a suction ladder with dustpan suction heads and dredge pumps. Eight mooring winches are normally available to position and shift the vessel, the eight mooring lines being attached to piles driven into the sea-bed.
After this operation the foundation bed for the piers is finished. By the time the job is completed at the end of 1983, a new method of foundation techniques involving a great amount of geotextiles will have been developed and intensively tested on the site and will be ready for use elsewhere in the world.

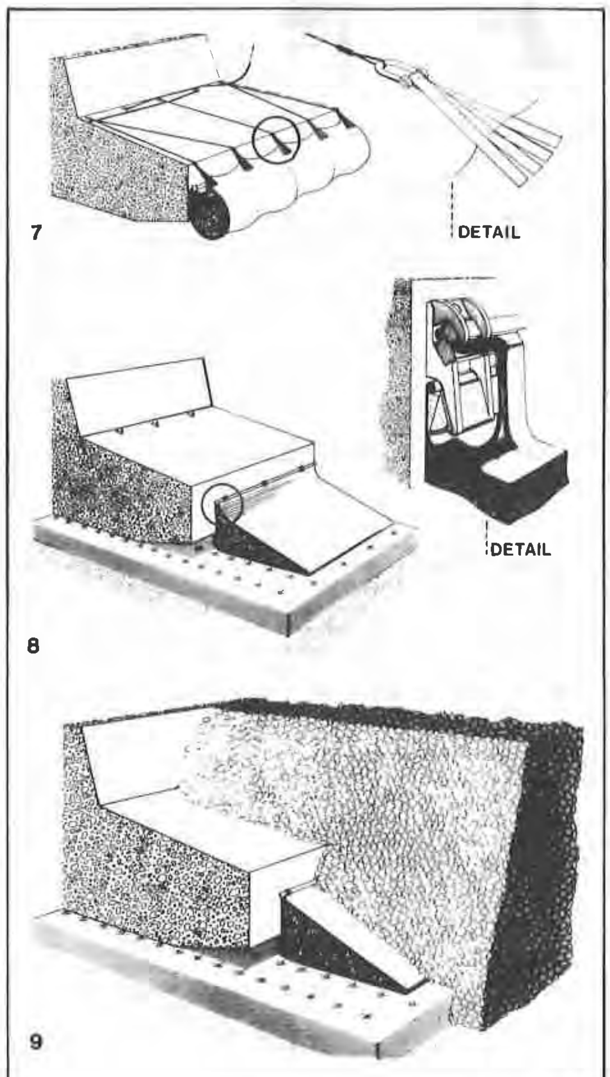
3. THE GRAVEL BAG

3.1 Brief description

After placing the piers on the foundation bed open spaces will remain between the base of the piers and the filter-mattress. This cavity has to be filled with a sand-cement mortar in a later stage of the construction. The gravel bag is necessary to keep that cavity temporarily open.

The phenomenon "gravel bag" (Fig.10B) requires some initial explanation. It is basically a strip of manufactured fabric, 100 x 8 m, that is folded into two along the width and is filled with 160 x 8 m^3/m coarse gravel. This "sausage" of fabric and gravel is positioned round the pier, firmly connected to it and tied up onto it (Fig.7). This gravel bag is a rather vulnerable structural element that is to be attached at the end of the construction of the enormous prestressed concrete piers. After inundation of the construction dock and transport to the site, the pier is placed at its final location. Immediately after placing the spaces remaining under the piers must be protected against penetrating sand, silt and attaching sea-organisms. Therefore the gravel bag is to be unchained from the tied-up situation, so that is falls out onto the

Fig. 7 : Gravel bag tied-up - suspension loop detail.
Fig. 8 : Gravel bag unrolled - pier connection detail.
Fig. 9 : Gravel bag dumped, with sill.



foundation mattress and closes the cavity around the pierbase (Fig.8).
In the following construction stage the sill is dumped and only then the joint between pier and mattress can be completed by injection of sand-cement filling from inside the pier.In this stage the gravel bag acts as mortar formwork (Fig.9).
All 66 piers are to be provided with such a gravel bag.

3.2 Surround seal;functions and conditions

Generally it is necessary to provide a seal around the pierbase. This will have a twofold function: as a seal against penetrating sand and silt and as a mortar formwork.
The first requirement is very essential. When sand or silt deposits in any extent under the piers there is an unreliable element in the foundation. Under the influence of water head and wave loading the pier will exercise large static and cyclic loadings on the foundation. This results in large water pressures and in static and dynamic gradients of many hundreds of percents(400-600%). Sand and silt can thereby be washed away which could result in rotation and translation of the pier and seizure of the gates.
The formwork function prevents mortar leakage and at the same time guarantees the reliability of the underbase filling process and the quality of the filling layer.

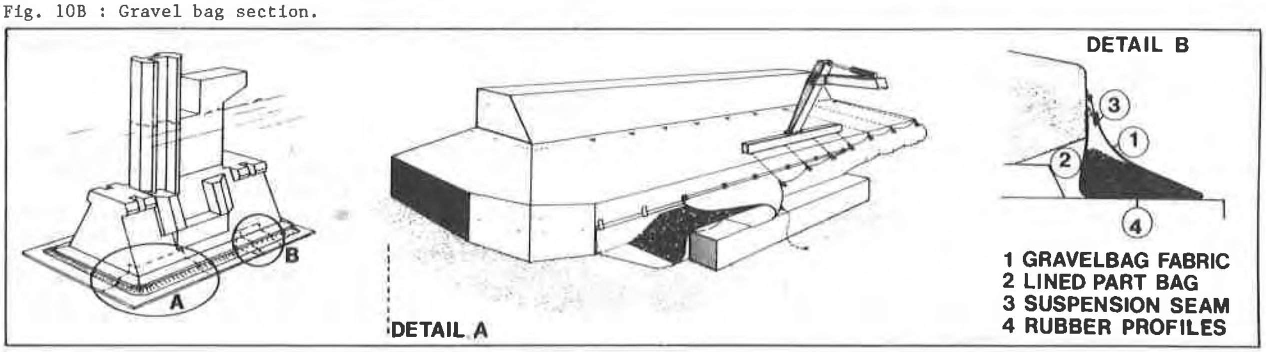
In detail this means also that the connection of the surrounding seal to the pier and mattress must fulfil these requirements, thus a siltproof seal against a rough concrete surface and against a mattress surface which will not be a model of flatness. Finally there is the time requirement that the system has to function immediately after placing the pier. Otherwise it must be sufficiently durable until the underbase mortar filling is executed; set at 5 years, including the necessary margin.

Besides these requirements there are many more boundary conditions, of which the important ones are:
- the height to be sealed can vary between 0.4 and 1.1 m, with local variations from 0.15 -1.35 m.
- the filter mattress may not be damaged by the sealing construction.
- limitations of weight and space in view of transport.
- the system has to withstand currents during transport and after release (2.5 m/sec).
- because of the durability requirement no holes must occur in the sill.

3.3 Detailed description

The main characteristics of the gravel bag are sealing obtained by the fabric; siltproof, and also as porous as possible. The contents of the bag has a weight function and enables the bag to withstand currents. The bag is pushed homogeneously against the bottom and the loose filling prevents the bridging of the fabric as much as possible over abrupt irregularities in the mattress.
The filling itself would not withstand the currents expected, therefore the bag must be closed carefully.
The durability requirement is very limited. The line has been taken that the textile can cease to functionduring the lifetime of the piers which is more than 200 years The grain size of the granular filling should therefore be a part of the filter formation of the sill, in which every layer is inpenetrable to the underlying material. Gravel 8-40 mm has therefore been selected.
The size of the gravel bag is ample, because a large height has to be spanned and at the same time a large connection with the mat is necessary. During transport the gravel bag would hang about 3.0 m below the pier. This however, cannot be allowed, it would be too vulnerable and uncontrollable. Therefore it was decided to tie the bag up against the pier. On the top edge of the pier base pins have been fixed 1 m apart, to which the bag can be fastened. Tying up is achieved using suspension loops sewed to the fabric. Wire ropes connect the loops to the pins.
In the submerged situation the tying up has to be undone. This is effected with the help of long steel ropes, which are lead under the tying up ropes and then upwards. The gravel bag is unfolded around the pier by pulling the ropes.
The connection with the pier consists of a double clamping device for both longitudinal sides of the fabric. Each side is doubled over a 30 mm thick rope with a considerable overlap and fastened with wooden wedges under the clips; 1 m apart (Fig.8).
The rope is tightened by pulling the fabric and pushes the fabric against the pier. A maximum tightness developes against the pier and the fabric is fastened without being exposed to peak loads.
To prevent any imperfections in tightening between the gravel bag and the foundation mattress a rubber profile has been developed which closes the last openings. The profile is tied on to the gravel bag.
In the actual choice of fabric the "permeability" requirement was less important than the siltproof requirement. Therefore the gravel bag is lined at the pierside with an almost waterproof textile.
In practice, based on the forces, a polyamide fabric has been chosen with interwoven strengthened strips 0.5 m wide, 1 m apart, at the same frequency as the suspension loops.
The fabric is woven 5 m wide, in which the five strengthened strips are provided by using three times the number of the warp threads. The direction of the strips is transverse because of the force direction. The unfolded width of the gravel bag is 8 m.
For manufacture, pieces of 8 m length are cut, on which the loops are sewed and the lining connected. Then half of the gravel bag is laid out and assembled on a large floor. The cross-seams and the expansion-seams are sewed on and the ropes connected with the longitudinal sides and the rubber strips fastened. This stage is completed by zig-zag folding of the bag and rolling it up.

Fig. 10A : Pier with gravel bag assembly.
Fig. 10B : Gravel bag section.



For the assembly procedure on the construction dock (Fig.10A), wooden boxes are installed at a fixed distance from the pier of 1.3 m. The gravel bag is unrolled. The clamps and lathes of the pier connections are fastened, the inner brim of the gravel bag clinched to the pier, the fabric pushed smoothly into the gutter between pier and boxes with the remainder of the fabric hanging over the boxes. The gutter is then filled with gravel and levelled. The outer brim of the gravel bag is then clinched and with the help of a crane with balance beam a part of the gravel bag is lifted and pulled in the direction of the pier, after which the ropes can be attached onto the pins of the pier.

3.4 Design aspects

The separate loading situations have been considered for the design in the successive stages. From here the critical loadings are deduced.

The assembly in the dry dock is a first stage with critical loadings. The forces develop from the dead weight of the filling, about 1400 kg/m. The lifting force of about 20 kN, on the gravel bag via the loops is important. This force is halved after connection onto the pins of the pier. The load-distribution is clearly a plane strain situation, especially during the lifting, because only three loops are lifted at the same time pulling up the adjacent part of the gravel bag.

The first important loading in the underwater stage occurs during untying. Because of the delayed fall of the loose granulated filling and the resilience of the elastic fabric it is difficult to analyse this arithmatically. Therefore prototype tests were necessary. In one of these tests an impact load occurred in a loop by a hitch when discharging the suspension wire. The disconnection of that loop gave a chain reaction, in which several loops broke. The fabric appeared to be completely undamaged, only the stitching was severed, so that the hitch was acceptable. The loading force had apparently exceeded the limit of load of 50 to 60 kN. Precautions concerning a premature and unintended discharge are therefore absolutely necessary.

The next critical loadings concern dumping sill material from the water surface. In this process the unrolled gravel bag is hit directly by stones of 8-40 kg and after that a static loading is built up to a σ_V is about 80 kN/m^2 and a σ_H is 20 kN/m^2.
After unrolling, the full capacity of the fabric is utilized. The falling stones set the fabric tight from below through which it is stretched. After this the static load has to be taken up, which overloads the fabric and the pier connection. Therefore an expansion-seam is incorporated into the outside of the gravel bag (Fig.10B). The strength of this has to be sufficient unrolling the bag but smaller than the strength of the pier connection.
Such expansion-seam is not necessary for the fabric at the pierside. The design elasticity garantees a small radius of curvature such that the maximum force is only 25 kN/m.

From these considerations and the tests described below it appears that there will be continuous loads of 10-25 kN/m and loadings of 25 kN on the loops, which have to be taken up locally in the fabric. The necessity of a rather large elasticity has been proved. In addition the expansion-seam will give extra capacity.

Naturally a decrease in strength will occur due to absorption of water in the filement; by ageing of the material as result of UV-radiation (maximum of two years); and the alkaline environment; and by creep as result of durable loading, all had to be taken in account. Also processing details and vulnerability had to be taken in consideration

In connection with these remarks the following primary requirements have been stated:

- linear strength in both principal directions	150 kN/m
- linear strain limit in both directions	150 %
- strength, loop at 45° with the fabric	50 kN
- tensile strength on pier connection at 30°	25 kN/m
- fabric sand tightness	$O_{90} \leqslant 300$
- silt tightness of lined fabric	$O_{90} \leqslant 10$
- strength sewing-seams with regard to fabric	70 %
- strength stitchings with regard to fabric	50 %
- strength expansion-seam	$\geqslant$ 20 kN/m and $\leqslant$ 25 kN/m
- lining strain equal to fabric strain.	150 %

3.5 Research

The design process has been accompanied by prototype tests through out from the very first until completion. All the time the practical possibilities of processing, the realisation of the pier connection and the critical loading situations have been reviewed. The tests were executed in dry conditions, with the assumption that this overloading would amply balance the reductions in strength and other influences on the specifications of the gravel bag as result of water absorption, ageing and creep. This method has been necessary for determination of the development of such a new structural element and its details. Initially tying up gave unexpected problems. The filling consisted of 1 m^3/m sand 0.3-30 mm. The internal friction gave large resistance to lifting the bag and to pulling it to the pier. When the contents were replaced by gravel 8-40 mm, the friction seemed to be so much smaller that simpler methods were possible.
The development of the suspension loop was also an important detail for research. The force of 50 kN at 45°with the fabric posed strong requirements to the connection. Cross-seams or zigzag-seams for the strings and the fabric formed connections which were too inelastic. Only the longitudinal seams proved to be sufficiently elastic, so that the stitches were equally loaded and reliable. In order to ensure that both strings came under load simultaneously small adaptions were made to the form of the loop. Particular examples of the consequences of the present plane strain condition were formed by the expansion-seam in the fabric with the strengthened strings and the lining on the ordinary fabric.
The research on the UV-durability of the fabric and yarns indicated a small reduction of strength (10%). The same applies for the water absorption (15%). The results of creep tests indicated changes of a few percent, particularly the stitching of the loops.

3.6 In conclusion

Because of the vulnerability of the gravel bag system a spare bag has been designed. When diver-inspection of the unrolled gravel bag indicates a possible mal-functioning, the bag can be installed.
The gravel bag has been chosen from various alternative designs, the criteria being the technical aspects mentioned in Section 3.2 and the costs.
The total costs of supply, manufacturing and assembly of the fabric and all other parts amounts to approximately U.S. $250/m.

4. References

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Analysis and Experimental Testing of Load Distribution in the Foundation Mattress

Analyse et verification expérimentale de la distribution des forces dans le matelas de fondation

In this study the specifications of the fabrics forming the foundation mattress have been derived from the load distribution in the mattress during production, winding on to the cylinder and the sinking operation.
For the overall load distribution a theoretical model approach is described; for a more precise analysis of the local strain distribution in the fabric near the pins an experimental approach is described. The tests on a 5 m wide tension dynamometer showed that a fabric, with steel wires and polypropylene split-fiber yarns in the warp and polypropylene split-fiber yarns in the weft, meets all the specifications for the load bearing fabric.
A winding test with a 3 m wide, 90 m long section of the foundation mattress is described. In this test strains in the fabrics and in the pins, as well as the surface geometry of the top fabric were measured. This test showed no damage during winding and a sufficient safety margin present in all the construction elements of the mattress. Based on the results of the study the design of the mattress and the specifications of the construction elements were accepted.

Dans cette étude les spécifications des tissus formant le matelas de fondation sont dérivées de la distribution des forces dans le matelas, pendant la production, enroulement sur le cylindre et l'opération d'enfoncement. Pour la distribution des forces en général une méthode usant d'un modèle theorique est décrite. Pour une analyse plus exacte de la distribution des tensions locales dans les tissus près des chevilles une méthode experimentale est donnée. Des essais sur une dynamomètre, 5 mètres de largeur, ont montré q'un tissu, avec fils d'acier et fils polypropylene en chaîne et fils polypropylene en trame, satisfait tous les spécifications pour le tissu portant les poids. L'enroulement d'une section de matelas de fondation, 3 m de largeur et 90 m de longeur est décrit. Dans cette essai les tensions des tissus et des chevilles, ainsi que la géometrie du tissue du surface sont mésurées. Pendant l'enroulement du matelas aucun dégât est constaté. Les résultats des mesurages ont prouvés q'une marge de sûreté suffisante existe pour tous les éléments du matelas. Basé sur le resultat de cette étude le projet est accepté.

ANALYSIS AND EXPERIMENTAL TESTING OF THE LOAD DISTRIBUTION IN THE FILTER MATTRESS.

1. Introduction.

The construction and production of the filter mattress are discussed in Reference 1.
In this paper the problem of finding fabrics suitable for the filter mattress is discussed. The specifications for the fabrics have to be determined in relation to the stresses and strains in the fabric during production, winding on to the cylinder and the sinking operation of the filter mattress. The specifications for water permeability and sand impermeability are directly related to the properties of the filling in the mattress and of the sandy soil of the "Oosterschelde".

2. Loading Conditions of the Mattress.

2.1. During production the mattress is supported by a conveyor and therefore the loads are negligible.
2.2. During transportation between the discharging wharf and the cylinder the load in the mattress depends on the shape of its catenary curve. The shape of the catenary is known since the radius of curvature of the mattress is limited to 5 m in order to avoid excessive bending strain. By applying the formula for the catenary $y = y_0 + a \cosh [(x-x_0)/a]$ and with the weight of the mattress being 720 kg/m^2 the load of the mattress can be shown to be 80 kN/m (Fig. 1).
2.3. On the cylinder the bending strain in the top fabric is found to be 5% (Fig. 2) assuming that no change occurs in the strain of the bottom fabric.
2.4. The load during sinking to a maximum depth of 35 m can be calculated from Fig. 3 taking into account that the weight of the mattress under water is 430 kg/m^2 and

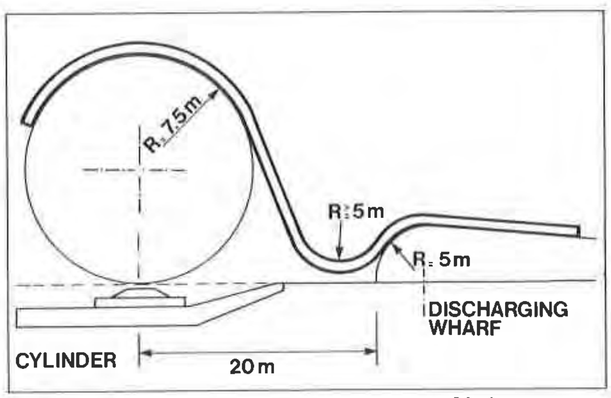


Fig. 1. Winding of mattress on cylinder.

the centre of the floating drum is 5 m above the water surface. The load has been found to be 200 kN/m.
Apart from the weight-load there is an additional load caused by the water currents. The current force at a maximum current speed of 2 m/sec during sinking is estimated to be 100 kN/m. The maximum static load during sinking will therefore be 300 kN/m.
At the end of unwinding slipping of the mattress over the cylinder surface can occur, as the difference between winding tension (80 kN/m) and sinking tension (300 kN/m) is considerable. The effect of this slipping is estimated to be a dynamic load of 660 kN/m.

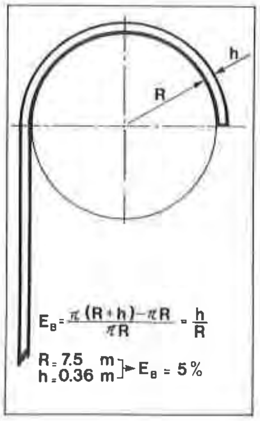


Fig. 2. Mattress on cylinder.

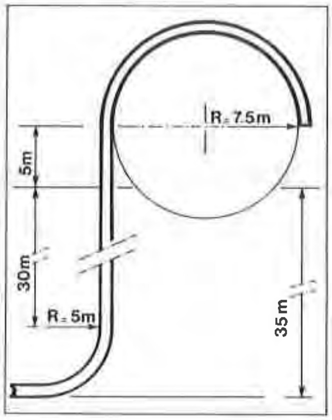


Fig. 3. Sinking of mattress.

3. Bottom Fabric.

3.1. Specification of the bottom fabric.

As it is assumed that the total weight of the filling material has to be supported by the bottom fabric, vertical hanging during the sinking of the mattress will be the critical phase.

In order to have a margin between the breaking strength and actual load, it was decided to specify a breaking strength of 800 kN/m for the bottom fabric. Experience with the polypropylene fabric of the blockmats, used as bottom protection, has shown that for a poplin weave the actual measured strength on a 5 m wide fabric was only half that calculated from a tensile strength test on a 20 cm wide test specimen. This effect is caused by the contraction in width of the fabric, due to crimp-interchange between warp and weft threads (Ref. 2). In the specifications for the bottom fabric it was stipulated that the widthwise contraction should be less than 0.5% and that the breaking strength of a 5 m wide fabric should be 4000 kN.

The breaking strain of the bottom fabric has to be limited, as the top fabric will have a 5% higher strain than the bottom fabric, when the mattress is wound onto the cylinder. A fabric having a high modulus and no widthwise contraction has the advantage that hardly any deformation of the filling material will occur, when the mattress hangs vertically. In addition the width of the mattress after being sunk to the bottom will be known exactly because there is no widthwise contraction.

3.1.1. In order to meet the specification for the widthwise contraction, the designers of the weaving mills, invited to make an offer, had to choose a straight warp weave.

In a straight warp weave the loadbearing warp threads have no crimp and consequently there is

a) no widthwise contraction of the fabric under load,
b) no bending strain of the warp yarns, resulting in a optimum efficiency of the yarn strength in the fabric strength.

To meet the specification for strength and breaking strain high modulus - high strength fibers are essential.

Two types of fabrics were offered:

a) a hybrid construction with steel wires and polypropylene split-fiber yarns in the warp and polypropylene split-fiber yarns in the weft,
b) high strength - high modulus manmade fiber yarns in a straight warp - straight weft construction.

3.2. Testing of the Bottom Fabric.

3.2.1. Testing of 20 cm wide fabric specimens.

First of all 20 cm test specimens were tested on a Zwick 1484 200 kN tensile tester of the Laboratory for Fiber Engineering of the Delft University of Technology.

Grips were developed, in order to be able to test these specimens and also to try to solve the problem of fastening the mattress to the sinking beam.

Two types proved to be successful:

a) a grip with friction bars (clasp type) (Fig. 4).
b) a wedge-shaped grip fastening the fabric with the help of a "filled" resin wedge cast to the ends of the fabric (Fig. 5).

In the friction bar-type the stress in the fabric at the grip is far less than in the middle of the test specimen due to friction between fabric and bars. In this way the total tension stress and compression stress in the fabric, pinched between the grips, is kept below the destruction resistance of the material, thus avoiding grip breakage. At the tip of the wedge-shaped opening of the wedge grip the compression load of the fabric is controlled by carefully adjusting the compression modulus of the "filled" resin wedge to avoid grip breakage. When the test results of certain fabric types met the specifications, 5 m wide fabrics were woven for testing.

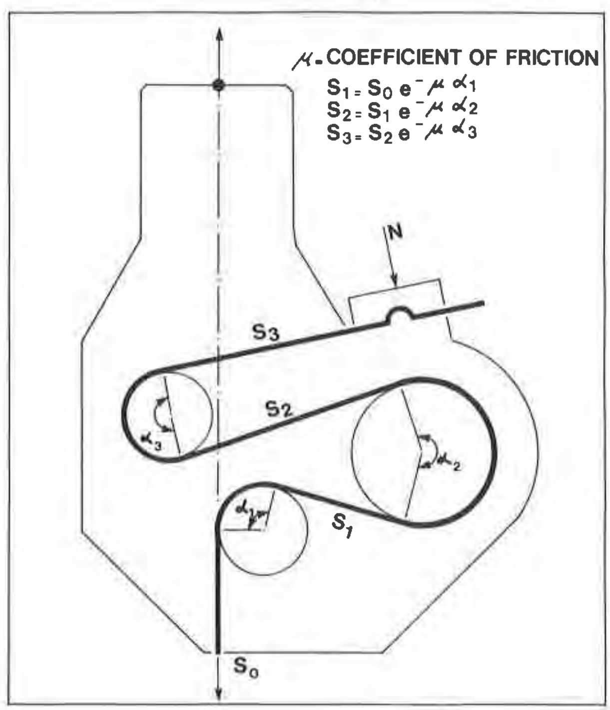
3.2.2. Testing of 5 m wide fabrics.

To test 5 m wide fabrics, a special testing apparatus was constructed by the "Deltadienst" on its testing site at the Oosterschelde. The wedge-type grip was used as a testing clamp. Load and strain in the warp direction, as well as the width of the test specimen were measured.

In a project like this an extra condition was, that for each step in design, a certain time limit had to be adhered to in order to complete the total construction by the set date. Although several types of fabrics did meet the requirements on the testing machine in the laboratory only the hybrid-steel-polypropylene warp met the 5 m wide specification within this time limit. In addition the 5 m wide wedge-type grip also proved to be successful.

It was decided therefore that the fastening of the mattress to the sinking beam would be done with the help of this type of grip. However as this type of fastening system is rather expensive, due to the high cost of the "filled" resin, for the fastening of the anchoring beam

Fig. 4. Clasp type grip.



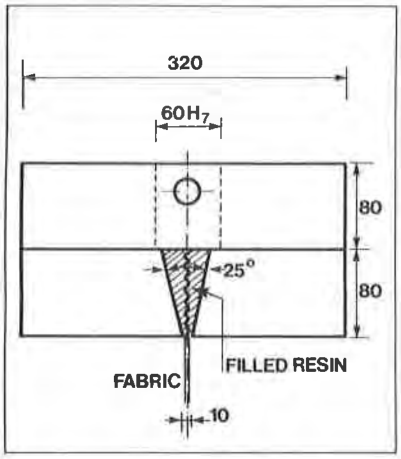


Fig. 5. Wedge shaped grip.

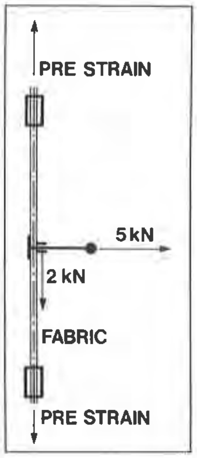


Fig. 6. Pin test.

to the mattress, the friction bar type of grip was in fact chosen.

4. Load Transfer between Pins and Bottom Fabric.

4.1. As the weight load of the filling material is transferred to the bottom fabric by the pins, the resistance of the fabric to this local strain had to be tested as no damage due to excessive local deformation of the mattress could be accepted.
The specification for the resistance of the fabric to the local pin stresses was derived from a theoretical model approach, discussed below (5.2.).
It was found that the resistance, parallel to the warp in the fabric, has to be 2 kN at a pin load of 5 kN perpendicular to the fabric, when the fabric is strained in the warp direction to 0.5% simulating the strain when the mattress is hanging vertically.

4.2. Pin force resistance test.
On the testing machine the fabric is prestrained and the pin is loaded perpendicularly to the fabric with 5 kN, then the pin is gradually loaded as near as possible to the fabric in fabric direction (Fig. 6). This load is recorded as well as the gap between the pin and the filling of the fabric (Fig. 7). It can be seen in this figure that the weft yarns of the hybrid type fabric can slip rather easily over the steel ropes of the warp. In this way several weft threads support the pin load.
The bending of the weft threads results in an angle between weft and warp threads such that the vertical component of the stress in the weft yarn is comparatively large. Moreover the pin force is distributed via the weft yarns over several steel ropes.
As the gap above the pins is covered by the washer, preventing the sand in the mattress from flowing through the gap, a maximum gap of 0.8 cm was specified. The hybrid type fabric was chosen for the definite design of the mattress as it was, at the time-limit, the only fabric meeting all the specifications, with an interesting price and low breaking strain which resulted in an as-low-as-possible breaking strain of the top fabric.
To be able to draw up the specifications of the top fabric the load distribution in the mattress must be known.

5. Load Distribution in the Mattress.

5.1. To get a better understanding of the load distribution in the mattress a theoretical model approach was used. The model consists of a bottom and a top fabric connected with separating walls, forming compartments. In the model the bottom fabric and separating walls are assumed to be of infinite stiffness as the pins are made of steel and the bottom fabric contains steel wires, having a very high modulus compared with a top fabric made of synthetic fibers. The connection of the separating walls to top and bottom fabric permits free rotation.
When the mattress is hanging vertically, the internal friction of the filling material will result in a load transfer of the weight of the filler to the bottom fabric and the connection between separating wall and bottom fabric.
In order to obtain maximum strain limits in the top fabric, when the mattress is in a vertical position, the internal friction of filling material is neglected. In other words we assume the filling to be a fluid having a density equal to the apparent density of the filling material.

5.2. Vertical Hanging of the Model Mattress.

5.2.1. To ensure a uniform filling of the mattress after it has been unrolled on the bottom of the Oosterschelde shifting of the filling has to be minimised. The compartment must therefore remain fully filled and therefore the volume must remain constant during deformation.
The distribution of pressure which the filling exerts on the walls of a compartment is given in Fig. 8a. For calculation of the deformation of the top fabric, the total pressure on it is assumed to be uniformly distributed (Fig. 8b). The deformed top fabric will then have the form of a part of the curved surface of a cylinder (Fig. 8c).
The geometrical condition of constant volume gives the relation between the rotation angle of the separating

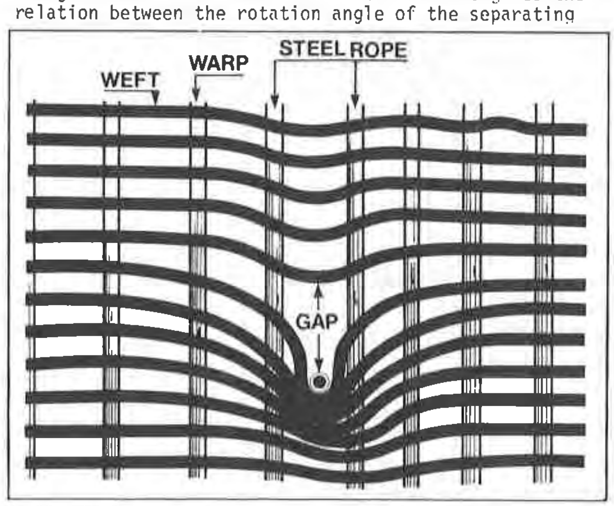


Fig. 7. Deformation of fabric.

Fig. 8. Pressure of filling on walls.

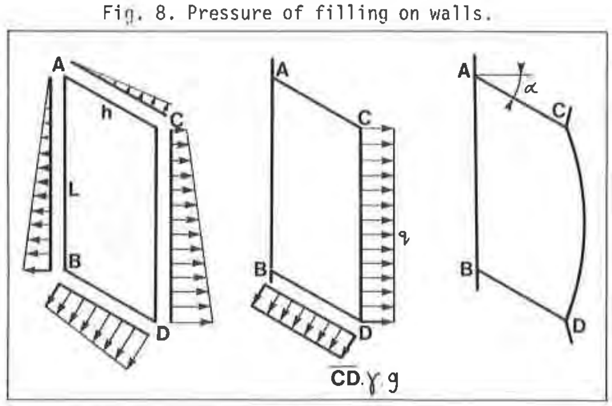


Table I. Results of the calculation of the stress and strain and fabric modulus for different rotation angles of the separating wall.

α rad	α °	R m	ε %	q kN/m²	P_H kN/m	per pin kN	S kN/m	modulus kN/m	Roving kN/m
0.1	5.7	20	0.004	7.1	4.738	1.7	143.575	36,000,00	0.024
0.2	11.5	5	0.07	7.9	5.199	1.9	39.386	5.570,0	0.092
0.3	17.2	2.2	0.37	8.6	5.650	2.1	18.833	5.080	0.208
0.4	22.9	1.4	0.95	9.1	6.088	2.2	12.914	1.370	0.367
0.5	28.6	0.9	2.4	9.8	6.508	2.4	8.074	370	0.570

wall and the radius of curvature of the top fabric. The stress in the top fabric can be derived from the condition of equilibrium of the top fabric (Fig. 9).

$P_H = 2\,S.\sin\phi$; $\qquad P_H = q.\overline{CD}$

$q = (\overline{AC}\,\sin\alpha + \frac{\overline{CD}}{2})\,\gamma.g. \qquad P_H = (\overline{AC}\,\sin\alpha + \frac{\overline{CD}}{2})\,\gamma.g.\overline{CD}.$

S = tension in top fabric
α = rotation angle of the separating wall
2ϕ = top angle of circle arc
E = Youngs modulus
γ = apparent density of the filling
g = acceleration due to gravity.
q = pressure on top fabric

The strain in the top fabric is derived from:

$$\varepsilon = \frac{\Delta l}{l} = \frac{2\phi R - \overline{CD}}{\overline{CD}} \qquad E = \frac{S}{\varepsilon}$$

5.2.2. Results of calculations.
In Table I the results of the calculation of the stress and strain and fabric modulus are given for different rotation angles of the separating wall. In reality this wall is formed by the fabric-lined cages in the mattress and the rotation angle has to be limited to α = 0.5 rad to avoid damage to these cages.
The calculated modulus values show that even at a rotation angle of 0.5 rad a relatively high modulus is required for the top fabric.

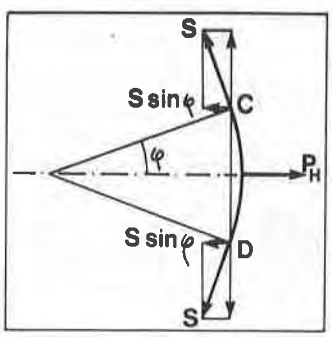


Fig. 9. Stresses in top fabric.

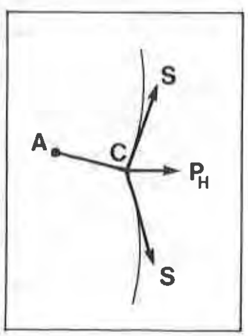


Fig. 10. Stresses at C.

Fig. 11. Stresses in separating walls.

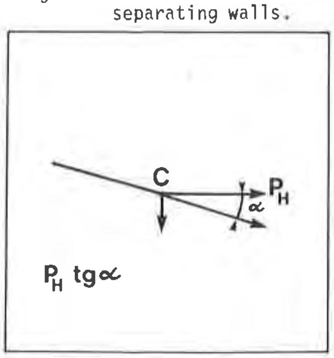
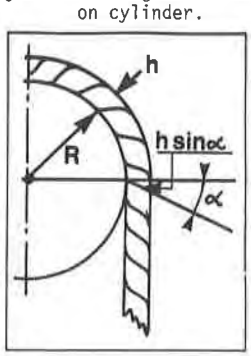


Fig. 12. Winding mattress on cylinder.



5.2.3. Stresses in the Separating Wall.
The stresses in the separating wall can be found with the help of Fig. 10.
At C the total horizontal reaction of the top fabric on the separating wall is equal to P_H. The relation between α and P_H is given in Table I.

5.2.4. Pin forces.
The horizontal pitch of the pins is 0.375 m. The horizontal force per pin, 0.375 x P_H, is given in Table I.
5.2.5. The specification for the pinforce resistance of the fabric is based on α = 0.5. The pin force perpendicular to the mattress is 2.4 kN (Table I). The pin force parallel to the mattress will be tan.0.5x2.4=0.3 kN. (Fig. 11).
Taking into account the dynamic effect described in(2.4), the specification of the pin force resistance of the fabric is chosen: 5 kN perpendicular to the fabric and 2 kN parallel to the warp of the fabric.
5.3. Load Distribution when the Compartment is on the Top of the Cylinder.
The stress in the top fabric is caused by the bending of the mattress plus the rotational displacement of the separating walls on both sides of the cylinder (Fig. 12).
The bending strain, ε_B, is given in(2.3) ε_B = 5%.
Rotational strain, $\varepsilon_R = \frac{h\,\sin\alpha}{\frac{1}{2}\pi R}$, when α=0.5 rad is ε_R=1.5%.

The maximum strain, when α = 0.5 rad is 6.5%.
When the compartment was about to be rolled on to the cylinder the strain was 2.4%. The higher strain in the top fabric caused by bending will result in a tendency to reduce the rotational angle of the separating walls.
5.4. When the compartment is at the bottom of the cylinder the full weight of the filling will be supported by the top fabric.
In a similar way as used in(2.3) the radius of the curvature can be calculated. Assuming the fabric to have a modulus 370 kN/m, we find R = 2.7 m and a strain of 5%. Compared with a normal bending strain of the top fabric of 4.8% only a slight increase in strain occurs and consequently there will be only a very low load in the separating walls.
6. Discussion.
The model approach shows that in the bottom fabric and the separating walls the stresses are a maximum, when the mattress is hanging vertically.
In the top fabric the strain is a maximum when the mattress is being wound on to the cylinder.
The model approach is essentially two-dimensional giving one dimensional stresses in the warp alignment of the fabric. In fact there is a plane stress condition, in addition to which there will be a stress concentration in the fabric close to the snaplocks on the pins.
6.1. Plane Stress Condition Approach.
The plane stress condition is generated as early as the final phase of the production of the mattress when the snaplocks are pushed on the pins. The snaplocks have to be pushed to 4 cm below the mattress surface to avoid contact with the bottom fabric when the second layer is wound on to the cylinder. In Fig. 13 the cross-sections of the mattress over the pins in both warp and weft direction are given. It is obvious that the warp strains are lower than the weft strains, as the distance between the pins is larger in the warp direction than in the weft direction.
Using the method given in(5.2.1),the warp and weft strains can be calculated.
We found ε warp = 1% and ε weft = 3%.

6.2 Plane Stress Calculation.
Calculations are based on a vertical cross-section geometry over the pins identical to that given in Table I. When the mattress is hanging vertically, the warp and weft stresses of the top section of a boss can be calcu-

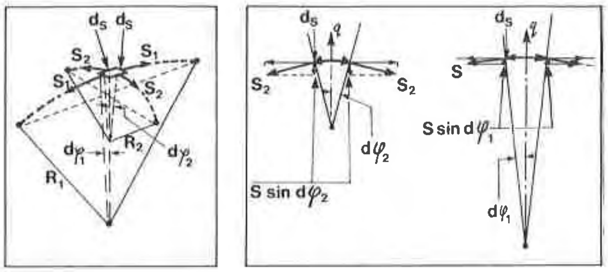


Fig. 13 + 14. Plane stresses in top fabric.

lated. Equilibrium conditions for the top section are: (Fig. 13 and 14):

$$q.(ds)^2 = 2\ S_1 \sin d\phi_1.ds + 2\ S_2 \sin d\phi_2.ds \quad (1)$$

$$ds = R_1\ 2\ d\phi_1 \qquad ds = R_2\ 2\ d\phi_2 \quad (2)$$

For small angles $\sin d\phi_1 = d\phi_1$ and $\sin d\phi_2 = d\phi_2$.

Equation (1) reads

$$q(ds)^2 = 2\ S_1\ d\phi_1\ ds = 2\ S_2\ d\phi_2\ ds. \quad (3)$$

Equations (2) and (3) give

$$q(ds)^2 = 2\ S_1 \frac{ds}{2R_1}.ds. + 2\ S_2 \frac{ds}{2R_2}.ds. \text{ and}$$

$$q = \frac{S_1}{R_1} + \frac{S_2}{R_2} \quad (4)$$

When we assume that for the top fabric the Youngs modulus in warp and weft direction is the same we find

$S_1 = \varepsilon_1 E$ (5) and $S_2 = \varepsilon_2 E$. (6)

Equations (4), (5) and (6) give

$$q = \frac{\varepsilon_1 E}{R_1} + \frac{\varepsilon_2 E}{R_2} \quad (7)$$

S_1 = fabric tension in warp direction
S_2 = fabric tension in weft direction
ε_1 = strain in warp direction
ε_2 = strain in weft direction
E = Youngs modulus top fabric
q = pressure on top fabric.

Based on a vertical cross-section geometry over the pins identical to that given in Table I, the warp and weft stresses and the fabric Youngs modulus can be calculated with the help of equations (5), (6) and (7).
R_1, ε_1 and q are given in Table I, R_2 and ε_2 can be calculated from the geometry.
For R_1 values of 0.9 m and 1.4 m, the stresses in warp and weft direction and the fabric Youngs modulus for the plane stress condition are given in Table II.
The result of the calculation shows that the weft yarns bear far more load than the warp yarns. Consequently the plane stress approach results in a far lower warp modulus of the fabric than found in the one directional stress approach. To be able to get an impression of the stress concentration close to the pins, the internal friction between the top fabric and the filling must be taken into account.

6.3. Influence of Friction.

When friction is present the deformation of the fabric is more localised in the region close to the pins. When the snaplocks are pushed 4 cm beneath the surface of the mattress and the deformation is limited in warp and weft direction to 10 cm, the calculated strain is 10%. Without friction we found a strain of only 3% in weft direction. Friction proves to be extremely important for the stress concentration at the pins and cannot be predicted exactly by a model approach, therefore experimental verification is necessary. In order to do these experiments a "best guess" specification for the top fabric was drawn up.
The local strain is chosen 6% in warp and 7.5% in weft direction. The bending strain for the warp from (5.3.) was 6.5%. Taking into account an uncertainty factor of 2, the "best guess" specification is:
warp breaking strain 2(6.5 + 6) = 25%
weft breaking strain 2 x 7.5 = 15%.
As the top fabric must have a relatively large permeability, the density of the fabric has to be as low as possible, consequently the rotation angle of the separating walls was chosen as high as possible (α = 0.5 rad).
A modulus of the fabric of 400 kN is chosen from Table II, which results in a warp breaking strength of 100 kN/m. A slightly higher modulus is chosen for the weft as this carries more load, 500 kN/m, resulting in a weft breaking strain of $\frac{15}{100}$ x 500 ≈ 80 kN/m.
The breaking strength and strain given above occur in a biaxial test, in a one directional stress condition of 25% warp-wise and 15% weft-wise.
A weaver was asked to develop a test fabric to meet these conditions.

7. Top Fabric Testing.

7.1. The strength of the fabric has been tested on the microprocessor-controlled biaxial tensile tester of the Delft University of Technology. The test is conducted in such a way that in the plane strain part of the test specimen, the weft strain, is 60% of the warp strain at any given moment.
7.2. The resistance of the fabric to the pin forces has been tested on a testing frame constructed for this purpose.
The fabric is strained 5% in both warp and weft direction, the loading of the pin is as described for the top fabric.

8. Experimental Strain Tests.

8.1. In order to simulate the actual strain condition of the fabric a box was constructed, 1.98 m long, 1.12 m wide and 0.36 m high. Two pieces of fabric were sewn together warpwise in the middle of which a Moire grid was attached to the fabric.
The fabric was mounted on the box which was filled with gravel. Pins with snaplocks were pierced through the fabric and the bottom of the box. In order to pull the snaplocks 4 cm below the original surface a force of 2 kN per pin was needed. The strain on the fabric was measured with the help of the Moire technique. The box was then placed vertically and the strains were measured. In order to find the maximum loading condition of the fabric the box was placed topside downwards on supports and strain readings were made. The results of these tests are given in Table III.

Table II. Calculated warp and weft strains when the mattress is hanging vertically.

α rad	R_1 m	ε_1 %	R_2 m	ε_2 %	E kN/m	S_1 kN/m	S_2 kN/m
0.4	1.4	0.95	0.47	3	1.33	1.26	4
0.5	0.9	2.4	0.31	7.3	38	0.91	2.8

Table III. Results of experimental strain tests.

	warp strain in %		weft strain in %	
	average	peak	average	peak
box horizontal topside upwards	½	4	¾	8
box vertical	1	4	1½	7
box horizontal topside downwards	2	8	3	8

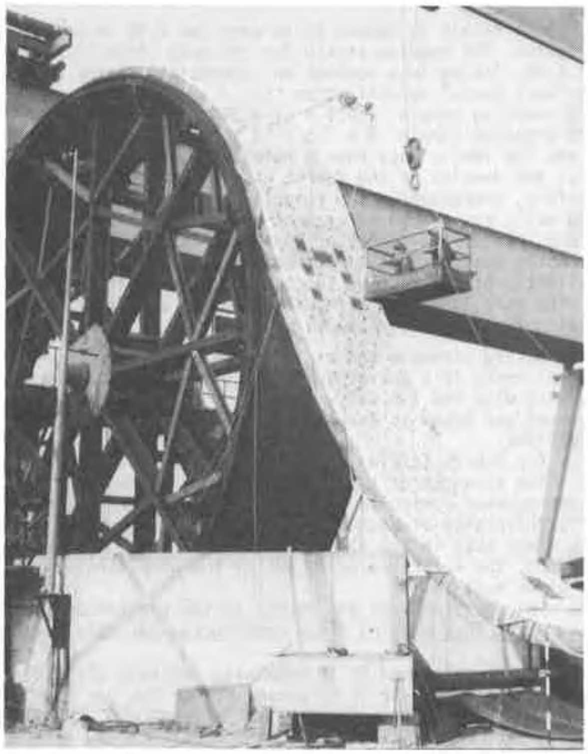


Fig. 15. Mattress Winding Test.

From the results of these tests it can be concluded that the strain readings agree very well with the "best guesses" given in (6.2.) It was decided therefore that a greater quantity of fabric should be woven and sections of the mattress 10 m long and 5 m wide should be made.

8.2. Experimental Tests on Sections of the Mattress.

The tests on these sections of the mattress were executed to find out if damage to the mattress construction elements would occur, when the loading conditions of the mattress were simulated. The section was suspended vertically and loaded at the bottom end, simulating unrolling in 35 m deep water.

The section was then bent to different radii of curvature both with the top fabric at the inner and outside of the curve. No damage whatsoever could be found in the mattress materials after all these tests.

Before a definite decision could be taken on whether or not the mattress would be suitable as a foundation for the piers, it was decided that a "full size" winding test should be executed.

9. Mattress Winding Test (Fig. 15).

9.1 For this test a 3 m wide test facility was constructed, in cross-section being an exact copy of the discharging wharf and winding operations of the mattress factory.

In order to get a double layer of mattress on to the cylinder, a mattress section 85 m long, 2.8 m wide and 0.36 m high was handmade.

In this mattress, a fabric made to the specifications was used and on both ends a beam was fastened to the bottom fabric by a resin-wedge clamp. The object of the winding test was to obtain information about the strains in the mattress construction elements at all the typical loading situations, during winding and unwinding of the mattress. For this purpose specially designed strain measuring devices were applied to the bottom and top fabric and the connecting pins. In addition pressure gauges were mounted in the sand layer of the mattress to measure soil pressures.

Three measuring sections were chosen, the first 10 m from the beginning of the mattress, the second at such a distance from the first section that, after winding the mattress onto the cylinder, it is situated on top of the first measuring section. The third section, 4 m from the end, is used to get strain measurements in the simulating of the unwinding situation.

To get information on the surface geometry of the mattress, stereo photographs were made with cameras mounted on a tripod, fastened to the mattress.

To measure the local strains in the top fabric around the snaplocks, Moire grids were fixed to the top fabric.

The winding tension in the factory can be governed by the torsional moment applied to the cylinder since the mattress is supported by a moving floor. In the test facility this floor was reproduced by a fabric between the mattress and the steel plates of the test discharging wharf, pulled forward by a separate set of winches.

The cylinder is rotated by means of steel wire ropes, wrapped around the cylinder and pulled by a 300 ton crane. Photographs were taken and the tension in the steel wire ropes was measured to verify the formula used to relate the shape of the mattress between the cylinder and the discharging wharf and the total force in the bottom fabric.

During the test the signals from the measuring devices were recorded.

9.2. Discussion of Test Results.

a) Pin forces: The pin force was 1.94 kN i.e. less than the 2.4 kN calculated from the model, neglecting internal friction in the filling material.
   As the holding force of the snaplocks is > 5 kN, the safety coefficient is greater than 2.
b) Strains in the top fabric: The average strain in the warp direction was 6.5%. The local strains were 8-12% in warp direction and 4-7% in weft direction, which corresponds rather well with the "best guess" approach. As the breaking strain in the fabric is over 25% in warp and 15% in weft direction, there is a safety factor of 2.
c) Stress in bottom fabric: The stress during normal winding was 86 kN/m, which corresponds rather well with the calculated 80 kN/m. The formula giving the relationship between the shape and the tension appears to be valid.
d) Rotation of the pins: The relative rotation of the pins during winding on to the cylinder appears to be very gradual, and does not appear to cause extra strain in the top fabric.
e) Soil pressures: The soil pressure measurements show that there is a relative movement of the particles more or less equalising stress concentrations in the soil.

Summarizing the results of the winding tests:
- no damage occurred during testing;
- there is a sufficient safety margin present in all the construction elements of the mattress.

Based on these results the design of the mattress and the specification of the construction elements discussed in this paper, were accepted.

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The Long-Term Thermo-Oxidative Stability of Polypropylene Geotextiles in the Oosterschelde Project

La stabilité thermo-oxydative à longue durée de géotextiles de polypropylène dans le project de l'Oosterschelde

4.5×10^6 m^2 block-mattress will be used for bed protection around the storm surge barrier in the Oosterschelde. The durability requirements for the polypropylene (PP)-bottom fabric in this mattress have been set extremely high (200 years). In pursuing these requirements, several improved antioxidant systems have been tested. These tests show that a high stability level combined with a high leaching resistance are the two prerequisites for antioxidant systems in the PP in order for it to be appropriate. Long-term tensile strength experiments have shown, that under practical circumstances the thermo-oxidative resistance of the material is the overruling, determining factor for the technical life of the PP-fabric. Non-woven spunbonded PP-sheet has been subjected to heat-ageing test at 50°-140°C, in its original state and in extracted state. On the basis of these results estimates have been made on technical life in bed protection (10^3 years). A geotextile has been selected for the gravel bag in the storm surge barrier on the basis of its weatherability.

Un matelas de blocs en beton de 4.5×10^6 m^2 sera employé pour protection de sol autour du barrage anti-tempête dans l'Oosterschelde. Les exigences de durabilité au tissu de fond de polypropylène (PP) dans ce matelas ont été placées extrêmement hautes (200 ans). A la poursuite de ces exigences, plusieurs systèmes d'anti-oxydants améliorés ont été éprouvés. Ces épreuves montrent qu'un haut niveau de stabilité, combiné avec une haute résistance au lessivage, sont les deux choses nécessaires auparavent pour des systèmes anti-oxydants dans le PP afin qu'il soit approprié. Expériences de résistance à la traction à longue durée ont montré que, dans des circonstances pratiques, la résistance thermo-oxydative de la matière est le facteur dominant et déterminant pour la vie technique du tissu PP. Du membrane de PP non-tissu a été soumis à des expériences de vieillissement à chaud à 50°-140°C, dans son état original et dans un état d'extraction. A la base de ces résultats évaluations ont été faites de vie technique de protection de sol (10^3 ans). Un géotextile a été sélectionné pour le bourrelet de gravier au barrage anti-tempête.

1. INTRODUCTION

During the development and execution of the Delta Works in the Netherlands, a rapid evaluation of bed protection methods has taken place (1). The necessity of protecting very large areas of the sea bottom together with an increasing scarcity of willow led to the development of synthetic fabrics for use as a base for prefabricated bottom protection constructions. In 1973 the so-called block-mattress was developed, being one of the trendsetting new types of bed protection. Because of soil characteristics the sand-impermeability of the block-mattress had to be improved; this was achieved by using a non-woven spunbonded polypropylene sheet on top of the PP-bottom fabric. In about the same period the so-called fixtone-mattress was developed. The aim of the work reported in this article has been to get a better insight into what factors determine the durability of the polypropylene geotextiles used in the bed protection units mentioned above. Requirements and generally practicable test methods had to be formulated, against the background of the Oosterschelde works. It appears, that once these mats are placed and in most cases covered by slag, gravel, quarry stone and/or asphalt mastic, the main risk for a proper long-term functioning is the possibility that the material will be unable to resist oxidative degradation. In addition the PP-fabric of the block-mattress should retain a minimum long-term mechanical strength under load because of the possibility of sudden soil displacements, leaving parts of the mat in a hanging, loadbearing position. Evaluation and testing of the various possible stabilizer systems form the subject of the present paper. For this purpose heat aging tests combined with leaching tests were used. These leaching tests were carried out over varying periods; this pre-treatment of test specimens will also be indicated by the term "extraction". In order to check the influence of mechanical load during thermo-oxidative degradation long-term mechanical strength experiments have been carried out at elevated temperature. As a representative of PP-fibres in various thicknesses, a non-woven spunbonded polypropylene sheet was selected for a more detailed programme of oven aging tests. Tests were carried out on the original as well as on the (sea water) extracted state, in order to establish a relationship between time to failure in heat ageing versus temperature. In addition, the relationship between time to failure in heat ageing and time of extraction at a moderately elevated temperature was also investigated. Polyamide fabric used for a gravel bag around the foot of the piers of the storm surge barrier in the Oosterschelde has been tested for weather-resistance over a limited period.

2. EXPERIMENTAL PROCEDURE

2.1. Oven-ageing tests

These tests have been executed in Heraeus Type T 5060 EK ovens, which have a cubic space ($600 \times 600 \times 600 mm^3$). Forced air circulation was not used. Air circulation velocity drastically influences the oven-ageing time to failure ("ovenlife"); when forced circulation is used, a substantially shorter ovenlife is observed than in the case of natural convective circulation (2). Therefore, in most standard tests forced air circulation is prescribed (3), also because then requirements related to temperature tolerances can be met more easily. A permanent difficulty in these tests, however, is the variation in air circulation velocity which occurs throughout the oven space, leading to a scatter in results. If a natural draught is applied, air circulation in ovens of this size adjusts itself in an experimentally reproducible way. According to our experience, even ovens of different design (namely arrangement of air outlets, etc.) can apparently be operated under practically equivalent conditions, and can consequently give equivalent results, provided that only a central part of the oven space is used. Tests were executed in a temperature range of 50° - 150°C, the temperature variation being not more than ± 0.5°C. Under these conditions the fresh air replacement ratio per hour is about 7 - 9 times the volume content. Test specimens (yarns 100 mm long, filters 40 x 150 mm^2) were suspended from glass clips in the centre of the oven, at a distance apart of 40 mm. In the case of the experiments on non-woven spunbonded PP-sheet test pieces were hung on a carrousel, which could be turned by hand around the vertical axis. At regular time intervals the test specimens were checked for embrittlement and the carrousel was turned by hand through 90°. In some cases premature embrittlement was observed due to gaseous dissociation products, originating from PP-residues on the hot oven bottom, which apparently catalysed the thermo-oxidative breakdown (4). This contact with the bottom was eliminated by using detachable aluminium foil trays covering the oven bottom and collecting the fallen residues, which could then quickly be removed during the daily check. The tests were terminated when the first of at least 20 specimens had become brittle.

2.2. Leaching tests

Leaching of test specimens in sea water, sampled from the Oosterschelde, was done in a 3l round bottomed flask, filled with sea water up to 2/3 of its volume. Test specimens were put into the flask, and completely immersed. A reflux cooler was connected to the flask and its content heated and refluxed for a fixed period. Afterwards the specimens were rinsed with cold water and dried in the air.

2.3. Long-term tensile strength tests

Tensile tests under constant load were done at 90° and 110°C. Two types of yarn were tested and coded A and B. From the yarns circular test specimens were made, total length 200 mm, the circle being closed with a suitable knot. The knotted specimens were suspended on the fixed upper connecting device of an Instron dynanometer drawing bench, in principle consisting of two metal pins, diameter 4.5mm, 13.5mm apart. Using a similar device connected to the moving traverse of the drawing bench the test specimens could be loaded and tested in tension. The raised temperatures were achieved by using an Instron dynanometer high temperature cabinet. Firstly, the reference breaking force F_T was determined.The traverse speed of testing was 50 mm/min. After that, a series of set percentages of this breaking force was applied as a constant load F and the time to failure was recorded. Tests with the lower loads ($<0.3F_T$) were executed in thermostat controlled oven cabinets with automatic time-to-failure recordings.

2.4. Artificial weathering test

In order to choose between two synthetic fabrics for use in the manufacture of gravel-filled bags around the foot of the piers of the storm surge barrier in the Oosterschelde (5) it was necessary to know the weatherability of the fabrics over a limited period (1.5 years). Test specimens, 50x250mm^2, were taken from the fabrics, and subjected to an artificially UV-ageing treatment in a Xenotest 1200 U accelerated weathering apparatus (Original Hanau Quarzlampen GmbH, Hanau, Western Germany). The test conditions were:

internal test chamber temperature : 25°C
black panel temperature : 47°C
filtering: 2 (instead of 3) "UV-Drittelschalen"
Artificial rain: 3 minutes/20 minutes running time
relative humidity: 60%
intermittent irradiation: 50/50

The light sources were 3 Xenonlamps, each 4500 W, situated in a selective (UV-reflecting, IR-absorbing) mirror system, surrounded by a quartz, water-filled cylinder. Around this cylinder the 3 "Drittelschale"-filters fit in their mountings. After the UV-ageing treatment, the specimens were subjected to a tensile test (distance between grips 140mm, speed of testing 100mm/ min., 23°C/50% R.H.). The breaking force, F_T, was determined, and compared with the corresponding findings for the unaged samples. As a basic requirement it was decided, that a reduction in strength larger than 20% would be unacceptable.

3. TEST RESULTS AND EVALUATION

3.1. General purpose polypropylene filter fabrics

For some 10-20 years polypropylene fabrics have been used in bed protection and embankment reinforcing units. Polypropylene can be considered as almost completely resistant to biological and chemical attack in sea water, and is therefore preferable to wooden or willow materials. For the Oosterschelde works, PP-fabrics for the block-mattress were initially woven out of yarns produced from fibre grade polypropylene homopolymer, containing 0.5-1.5% carbon black, about 15% (weft) to 20% (warp) polyethylene, and a rather moderate amount of antioxidants. These yarns were obtained by foil extrusion and water-bath chilling, then cutting in strips of a defined width, hot drawing, fibrillation, twisting (for warp) and winding. Water-permeability and sand-impermeability characteristics were produced by a special weaving technique. The required strength for the sinking operation was produced by optimising the drawing ratio and the linear mass of the yarns. However, how the strength of the yarns would change on the long term was unknown. This is of considerable importance since, under unfavourable conditions, the block-mattress, lying on the sea bottom,might still be loaded up to 10% of its strength (see Section 3.3). In this connection, the oxidation stability level must be sufficiently high throughout the technical lifetime. The general purpose PP-fabrics mentioned above generally showed an average ovenlife of 3 days at 150°C (6 days at 140°C). However, extraction with either sea water or a 2% Teepol GD 53 P solution at 100°C caused a reduction of the ovenlife to 30%. Some typical values are given in Table 1. These tests were carried out on a large number of samples (also on fixtone mattress fabric) with essentially the same results. Deviations in results were not more than 1 day. Warp and weft were also analysed with respect to the antioxidant content, the results being summarized in Table 2.

Table 1. Ovenlife (days) at 140°C, block mattress PP-fabric, warp and weft; before and after extraction in Teepol solution and seewater at 100°C.

	fabric	warp	weft
. unextracted	6	6	6
. extracted in Teepol solution - 7 days	2	2	2
- 14 days	2	2	2
. extracted in sea water - 14 days	2	2	2
- 28 days	2	2	2

Table 2. Weight percentage antioxidant* in warp en weft of the block-mattress fabric before and after extraction in sea water at 100 °C.

	warp		weft	
	Ionol	Irganox 1010	Ionol	Irganox 1010
. unextracted	.043 (±.002)	.0102 (±.0004)	.072 (±.004)	.0134 (±.0004)
. extracted, 8 weeks	<.001	.002	<.001	.004

* Ionol : 2,6-di-tert.-butyl 4-methylphenol
Irganox 1010 : penta erythrityl-tetrakis-[3-(3,5-di-tert.-butyl-4-hydroxy-fenyl)-propionate]

From these results it was concluded, that long-term leaching of protecting antioxidants had to be taken into account. Essentially the same conclusions were drawn from other studies, where similar fabrics, used in bed protections in the Delta Works for several years and dredged from the sea bottom, were analysed and compared with original material. In the planning of the storm surge barrier it was decided, that every prefabricated element, which in principle could not be replaced, should have an expected life of at least 200 years (1). On the basis of considerations which will be dealt with in Section 3.4, a residual ovenlife of 2 days at 140°C was then considered to be insufficiently compatible with this requirement. Furthermore, the revised concept of the storm surge barrier required an increase in the total area of sea bottom to be covered with block-mattresses from $1.5x10^6$ m^2 to $4.5x10^6$ m^2, reaching as far as 600m from the barrier axis on both sides. In this concept the bed protection will be exposed to tidal currents. This requirement therefore has constituted another reason for improvement of the antioxidants with respect to leaching.

3.2. Low-leach polypropylene filter fabrics

At the request of Rijkswaterstaat, a first low-leach stabilizer formulation, to be applied as a master batch, designated as PLZ 453, was evaluated by Shell Nederland Chemie. Ovenlife tests were done on 0.5mm sheet test specimens at 150°C instead of 140°C, giving the results summarized in Table 3.

The samples which were sea water extracted for 56 days were also analysed for antioxidants (Table 4).

Since these results indicated a substantial increase in leach resistance and thermo-oxidative resistance this system was introduced in split film fibre production, and the yarns produced (film thickness 60-70μm) were again tested (Table 5).

From these results it can be seen that, in comparison with the results for the 0.5mm sheet, the initial ovenlives are less, mainly due to the difference in thickness and the influence of processing and twisting. In this respect the difference between warp (fibrillated and twisted) and weft (flat tape yarn) is systematic, the warp value being the lower value. Also, the leaching effect is more pronounced, especially for the warp yarn. This stabilizing system, originally applicable for low-leach injection moulded parts,appeared to cause difficulties in foil extrusion. After the water-quenching step the foil showed water carry-over, which spoiled the stretching conditions. Therefore, another stabilizing system was formulated where Irganox 1010 and DMTDP were replaced by Ionox 330*, which is insoluble in water and does not show the disadvantages in processing mentioned above. The warp yarns produced were tested for thermo-oxidative resistance and leaching resistance, leading to the results given in Table 6. Experiments were done in two separate ovens of different design.

Table 3. Ovenlife (days) at 150°C,of PLZ 453 stabilized PP-sheet 0.5mm, before and after extraction at 100°C.

. unextracted	20
(. unextracted general purpose	3)
. Teepol extraction	
- 7 days	13
- 14 days	11
. Sea water extraction	
- 14 days	17
- 28 days	15
- 56 days	15

Table 4. Weight percentage antioxidant* in PLZ 453 stabilized PP-sheet, thickness 0.5 mm, before and after extraction at 100°C.

	Irganox 1010	DMTDP	Ionol
. unextracted	0.118 (±0.003)	0.35 (±0.01)	0.05
. extracted in sea water, 56 days	0.094	0.23	0.002

* DMTDP = di-myristyl-thio-di-proprionate

Table 5. Ovenlife (days) at 150°C of PLZ 453 stabilized block-mattress yarns, before and after extraction at 100°C.

	warp	weft
. unextracted	9	12
. extracted in sea water		
- 7 days	6	10
- 14 days	2	8

Table 6. Ovenlife (days) at 150°C of improved PLZ 453 stabilized block-mattress warp yarns, before and after extraction at 100°C.

	Oven A	Oven B
. unextracted	15	16
. extracted in sea water,		
- 1 week	10	11
- 2 weeks	4	4

* Ionox 330 : 1,3,5-trimethyl-2,4,6-tris-(3,5-ditert. butyl-4-hydroxy-benzyl)-benzene

Although an improvement in oxidative stability was again achieved, the additive system was accompanied by a lower production speed, so that Shell Nederland Chemie decided to develop another low-leach stabilizing system, this time especially intended for extrusion processing of these fibres. This system was called PLZ 458, the components of which were not disclosed. The warp yarns produced were tested as before, giving the following results (Table 7).

It was concluded that the PLZ 458 system could be considered as equivalent to PLZ 453, if not being better. It was recognised furthermore, that within one production stream a certain scattering in ovenlife can occur due to imperfections in dispersion of the antioxidant masterbatch over the bulk polymer. This illustrates the

Table 7. Ovenlife (days) at 150°C of PLZ 458 stabilized block-mattress warp and weft yarns before and after extraction at 100°C.

	Sample I warp	Sample I weft	Sample II warp	Sample II weft	Sample III warp	Sample III weft
. unextracted	15	17	13	20	19	18
. extracted in sea water						
- 1 week	10	13	5	13	10	16
- 2 weeks	10	13	4	12	8	14

Table 8. Ovenlife (days) at 150°C of two warp yarn samples PLZ 453 stabilized, before and after extraction at 100°C.

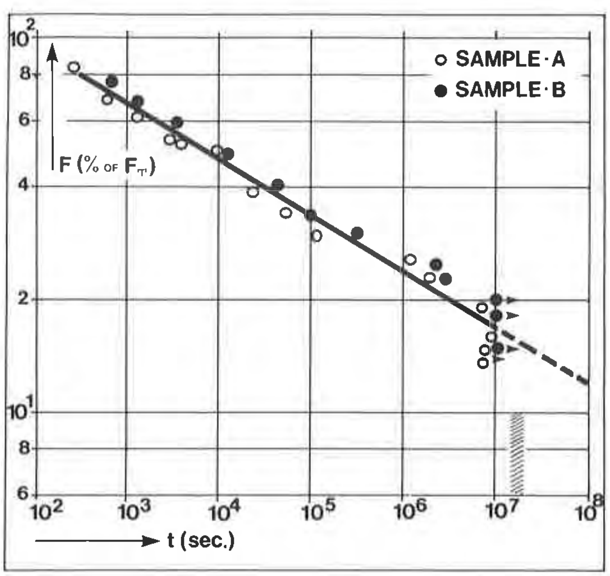
	Sample A	Sample B
. unextracted	11	14
. extracted in sea water		
- 7 days	9	11
- 14 days	8	11

necessity of maintaining an efficient production control.

3.3. Long-term tensile strength of PP-split film yarns

Because of the possibility of sudden soil displacements, leaving parts of a block-mattress in a suspended, load-bearing position, it was necessary to investigate the long-term strength of the PP-yarns. Two warp yarn qualities (PLZ 453 stabilized) coded A and B were investigated. The ovenlife and extraction resistance were also tested (Table 8).
Long-term tensile strength tests were done at 90°C and 110°C, from which the results at 110°C are shown in Figure 1. Experimental details are given in Section 2.3.
There is hardly any difference between sample A and B. The same conclusion can be drawn from the results at

Fig. 1 Time to mechanical failure of two samples polypropylene splitfilm yarns (1900 tex) under constant load at 110°C. The shaded area represents the time interval where the ovenlife of the yarns may be expected.



90°C. An ovenlife at 110°C can be estimated from the ovenlife at 150°C (11 and 14 days), yielding a value ~ 2.10^7 sec. (230 days). This estimate, made on the basis of ovenlife experiments and assumptions given in Section 3.4, has been incorporated in Figure 1 as a shaded area. From construction calculations it is known that under the most unfavourable circumstances the load remaining on the block-mattress after sinking add up to a maximum of 10% of the breaking strength. From Figure 1 it can be seen that for loads up to 0.1 F_T the extrapolated time of failure (>10^8 sec) would become longer than the time needed for (unloaded) thermo-oxidative breakdown. Therefore, at this load-level it is the thermo-oxidative resistance of the polypropylene which constitutes the determining factor in the functioning of the PP-fabric. Tests were terminated at 10^7 sec (116 days). At that time there were no signs of embrittlement.

3.4. Non-woven spunbonded polypropylene sheet

In order to establish an estimate of the durability on the basis of the ovenlife tests the relationship between ovenlife, t_o, and temperature should be known. This relationship is commonly represented by the formula

$$t_o = A.e^{\Delta H/RT}$$

where A = a constant
ΔH = activation energy
R = gas constant
T = absolute temperature

At higher temperatures (>80°C) activation energies for PP are reported in literature, to range from 80-110 kJ/mol. As a consequence, ovenlife tests at lower temperatures than 150°C or 140°C are increasingly time consuming. Experiments on the high-stability low-leach formulations, discussed in Section 3.2, would therefore be hardly feasible. In order to avoid these well-known difficulties as much as possible, the non-woven spunbonded PP-sheet, used in the block-mattress, was chosen for a more elaborate experimental program. The stability level was lower than the block-mattress PP-fabric; moreover, tests on sea water extracted and also on diethylether extracted (= unstabilized) sheet could be done, which presumably would not take a too long time. The mass per unit area of the sheet was 170g/m², the thickness was ca 0.4mm, the diameter of the primary filament being ca 15 µm. The material was analysed for antioxidants, and appeared to contain:
0.004 w/w % Ionol
0.062 w/w % Topanol CA: tris (2-methyl 4-hydroxy 5-tert. butyl-phenyl)butane
0.370 w/w % Cyasorb UV 531: 2-hydroxy 4-n-octyloxy-benzophenone
0.06 w/w % carbon black
Ovenlife tests were done at temperatures ranging from 50°-140°C, each experiment being on 12 test pieces. A logarithmic time-average ovenlife was calculated, together with a logarithmic standard deviation interval for observed values (Table 9).
These results are shown in Figure 2; they clearly show that the relationship is not-linear but turns of slightly towards shorter ovenlife times at lower temperatures. This effect is found generally (6) and is attributed to the varying effectiveness of the stabilizing system, depending on diffusion and solubility in the polymer, being governed by temperature. For this reason ΔH varies with T. For temperatures above 80°C we found a ΔH-value of 93kJ/mol (unstabilized PP), which is in fair agreement with literature. Because of the experimental inaccessibility, literature data on ΔH-values at lower temperatures are scarce. Below 80°C ΔH tends to decrease gradually. For unstabilized natural PP, values of about 50 kJ/mol are reported (7). Our results indicate a value below 70°C of about 65 kJ/mol,

Table 9. Ovenlife (days) of non-woven spunbonded PP-sheet at different temperatures; logarithmic time averages and (s.d. interval).

Temperature (°C)	Unextracted	Extracted in sea water - 7 days, 100°C	Extracted in diethylether
140	13.8 (12-16)	4.1 (3.0-6.0)	< 1 (-)
120	61.6 (53-71)	20.5 (13-32)	1.1 (0.9-1.5)
100	160.7 (151-171)	57.6 (49 - 67)	6 (-)
80	456 (430-480)	237 (225-249)	31.4 (30-33)
70	-	-	83 (-)
60	-	> 300	168 (158-178)
50	-	-	> 300

Table 10. Ovenlife (days) at 120°C of non-woven PP-sheet vs time of extraction (days) in sea water, 50°C; logarithmic averages and (s.d. interval).

time of extraction	0	35	70	175	350
ovenlife	61.6 (53-71)	33.5 (32-35)	29.0 (28-30)	19.0 (15.5-23)	4.4 (2-9)

giving a reasonable agreement, since, in the extracted sheet the low content carbon black may have increased the activation energy. The graph of the extracted sheet appears to be about a factor of 10 lower than the unextracted sheet, whereas, as could be expected, the sea water extracted sheet lies in between. The average temperature of the sea water in the Oosterschelde is about 5°C. If one should calculate for the extracted sheet an induction time for embrittlement at 5°C from the 60°C-value, using as activation energies 50 and 65 kJ/mol (as a low and a high estimate), this would yield 16 and 48 years, in air respectively. Since the oxygen content in the water is about 1/7 of the oxygen content in air, the time to embrittlement in water will be accordingly longer. The relationship between oxygen intake and oxygen pressure (a hyperbolic function) is not known exactly. However, from analogous experiments it may be inferred(8) that a factor 4 is fairly realistic, bringing the expected technical lifetime of the extracted sheet to 65-190 years. As a best guess, we assume that the graphs for the extracted and unextracted sheets will run essentially parallel to each other. Since ovenlife values of the unextracted material are about a factor of 10 higher, this leads to a calculated life of 6.5×10^2 - 1.9×10^3 years. Hence, the non-woven sheet can meet the durability requirements, if losses of stabilizer can be restricted. To get an impression regarding this phenomenon and, at the same time, to evaluate the meaning of the 7 days 100°C-extraction, a long-term extraction at 50°C was carried out. To quantify the effect of extraction, ovenlife values at 120°C were determined (Table 10, Fig. 3).

It can be concluded that extraction at 100°C over 7 days - which produces a reduction in ovenlife to 20.5 (Table 9) - gives a fair impression about the extractability, since the effect is the largest in the initial stage. Apparently a judgement about extractability of stabilizing systems does not necessarily involve long extraction times. Furthermore it can be seen, that after quick initial extraction of stabilizer this process slows down considerably. This is also a feature which was observed at the selected PLZ 458 stabilized PP-yarns (Table 7), where a more or less constant ovenlife level was reached at prolonged extraction. Comparison of the same figures with values in Figure 2 shows, that the corresponding points (which are minimum values and not average values) and graphs of the PP-yarns lie above the not extracted PP-sheet graph. If the above mentioned approximations regarding ΔH also hold true for the PP-yarns, a residual ovenlife level after long-term extraction, say 10 days at 150°C, would imply a 1.5 times higher stability than the unextracted non-woven PP-sheet. This would amount to an expected minimum lifetime of about 10^3 years. The large uncertainty regarding ΔH-values in the low temperature region cannot be denied. On the other hand, this calculation is generally conservative. Therefore it is likely that, within the boundaries of existing knowledge, the durability requirement is met by the PLZ 458 stabilized fabric.

3.5. Polyamide fabric for the gravel-filled bag

When considering the selection of a fabric for the gravel bag (5) a weatherability test was necessary in order to ensure a sufficient UV-resistance of the fabric before immersion in sea water. It was required that the fabric should show a less than 20% reduction in strength in an outdoor exposure period of 1.5 years. Two fabrics were tested as described in Section 2.4, namely

. a fabric, consisting of PP (warp) and linear polyester PETP (weft)
. a fabric, consisting of polyamide (warp and weft).

Fig. 2 Time to mechanical failure of non-woven spunbonded polypropylene sheet, by heat-ageing in an oven, as a function of temperature.

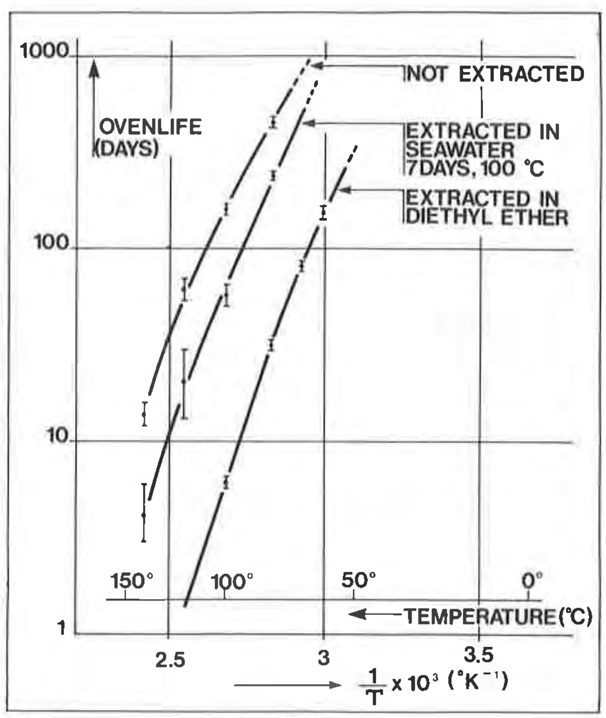


Table 11. Results of the accelerated weathering test.

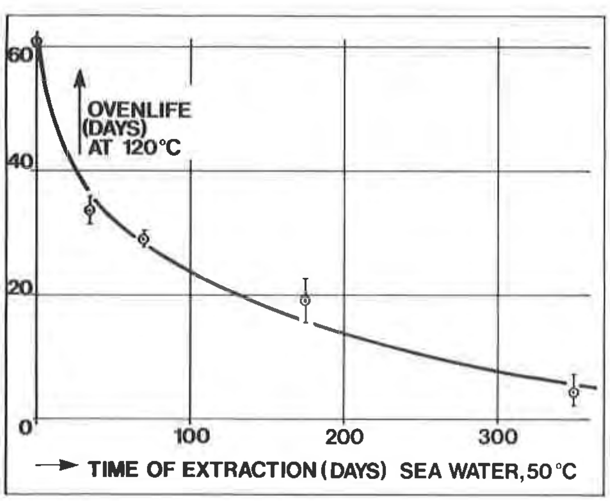
	breaking load (kN)		
ageing time	0 hours	250 hours	500 hours
PP warp	9.7	9.2	8.6
PETP weft	5.9	3.0	2.3
PA fabric	7.9	7.8	7.2

According to our experience, 450 hours accelerated weathering is equivalent to 1.5 years. The reduction in strength regarding PP and PA is acceptable, whereas for PETP the reduction even after only 250 hours was unacceptable. On the basis of this result the PA-fabric was recommended. Suspension loops are sewn on the gravel bag, for tying it up against the pier, during transport and placing of the pier. Therefore, the stitchings of the loops have also been tested for UV-resistance by subjecting a complete sample of gravel bag, with a loop connection in it, to the accelerated UV-test. After this, the test sample was mechanically tested on the Zwick 1484 200 kN tensile tester of the Laboratory for Fibre-Engineering of the Delft University of Technology. The loop was drawn through a 45° angle, increasing the tensile force over a period of 1 minute up to the required initial value of 50 kN. This was done twice, with no effect on the loop. The test was repeated and the applied force was increased up to failure at a maximum force of 56 kN. The ropes were broken whereas the seams remained unimpaired. It was concluded therefore that the gravel bag would be able to resist weathering during the 1.5 years without showing an unacceptable loss in strength.

3.6. Conclusions

The results of the long-term mechanical strength experiments demonstrate, that with regard to the durability of bed protections used in the Oosterschelde project the thermo-oxidative resistance of the PP-woven structure will be the determining factor. It has also been demonstrated that in protecting the PP against thermo-oxidative breakdown in the sea water on a 200 years' term the stabilizing system must have a high stability level combined with a high resistance against leaching by sea water. These two characteristics are the prerequisites for fulfilling the high demands of the Oosterschelde project. It has been shown that this aspect can be effectively controlled by a combination of extraction tests and oven tests. Although there is uncertainty about the appropriate values of ΔH in the lower temperature region, a generally conservative calculation demonstrates that the PLZ 458 stabilised fabric can meet these high durability requirements: the technical life is estimated at 10^3 years.

Fig. 3 Time to mechanical failure of non-woven spunbonded polypropylene sheet by heat-ageing in an oven at 120°C, plotted against the time of preceding extraction in sea water at 50°C.



4. APPLICATIONS OF TESTING METHODS IN CERTIFICATION AND PRODUCTION CONTROL

On the initiative of Rijkswaterstaat Deltadienst, a Working Group "Plastics in Water Works" was formed in 1976, which started to formulate requirements related to PP-yarns for fabrics for block-mattresses as well as fixtone mattresses. These requirements plus test methods were issued as so-called K-sheets (9), and contain requirements regarding linear mass, specific tensile strength and elongation at break, for warp and weft separately. Also a test requirement for the thermo-oxidative resistance of the yarns before and after sea water extraction at 100°C during 7 days was established, being 12 and 6 days minimum ovenlife at 150°C. Experimental conditions were specified in accordance with Section 2.1. In the Netherlands a KOMO-certificate is granted to manufacturers able to deliver yarns meeting these specifications. To achieve this the manufacturer must maintain an internal quality control system which is assessed and controlled by the KIWA Institute, the external executive testing institute of the KOMO Foundation. The internal quality control applies to not only the testing of yarns but also to the complete production process. A detailed list of points to be checked is drawn up by the manufacturer together with the KIWA Institute. Regular checks and samplings are made by KIWA inspectors, who are authorized to enter the factory at any time. It has been our experience, that certification and production control has improved the quality of the components to a considerable extent. Also the evaluation of new stabilizing systems and their incorporation in production has undeniably been stimulated by the certification activities. In the near future the results of these experiments discussed above will be utilized for the formulation of general specifications concerning a limited number of geotextile classes.

5. REFERENCES

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APPENDIX

SECOND INTERNATIONAL CONFERENCE ON GEOTEXTILES
DEUXIÈME CONGRÈS INTERNATIONAL DES GÉOTEXTILES

LIST OF SYMBOLS
LISTE DE SYMBOLES (*)

1. GENERAL SYMBOLS
SYMBOLES GENERAUX

1.1 Dimension symbols
Symboles de dimension

Symbols used for the dimensions are: Les symboles de dimension sont:

L :	length	longueur
M :	mass	masse
T :	time	temps
- :	dimensionless	sans dimension

1.2 Unit Symbols
Symboles d'unités

m	meter	mètre
m^2	square meter	mètre carré
m^3	cubic meter	mètre cube
mm	millimeter	millimètre
µm	micron	micron
g	gram	gramme
mg	milligram	milligramme
kg	kilogram	kilogramme
s	second	seconde
N	newton	newton
kN	kilonewton	kilonewton
Pa	pascal	pascal
kPa	kilopascal	kilopascal
MPa	megapascal	mégapascal
J	joule	joule
tex	tex	tex
L	liter	litre
^{o}	degree	degré
%	percent	pourcent
-	pure number	nombre pur

(*)This list of symbols has been prepared by J. P. Giroud.
Cette liste de symboles a été établie par J. P. Giroud.

The following relationships exist:

Relations entre les unités ci-dessus:

$$1\ \text{Pa} = 1\ \text{N/m}^2$$

$$1\ \text{tex} = 1\ \text{mg/m}$$

$$1\ \text{J} = 1\ \text{mN}$$

1.3 Mathematical symbols
Symboles mathématiques

ln x	natural logarithm of x	logarithme naturel de x
lg x	logarithm of x base 10	logarithme décimal de x

1.4 Subscripts
Indices

Subindex / Indice	Applies to / Se rapporte à	
a	air or active (earth pressure)	air ou actif (poussée)
d	dry state	état sec
f	failure or final	rupture ou final
g	geotextile	géotextile
H	horizontal	horizontal
i	immediate or initial	immédiat ou initial
p	passive (earth pressure)	passif (butée)
r	radial	radial
s	solid particles	particules solides
sat	saturated	saturé
sec	secant	sécant
u	undrained conditions	conditions non drainées
V	vertical	vertical
w	water	eau
x, y	two orthogonal horizontal axes	deux axes orthogonaux horizontaux
z	vertical axis	axe vertical
o	at rest or initial conditions	au repos ou conditions initiales
1,2,3	principal directions	directions principales

1.5 Geometry and kinematics
Géométrie et cinématique

L	L	(m)	length	longueur
B	L	(m)	breadth	largeur
H	L	(m)	height, thickness	hauteur, épaisseur
D	L	(m)	depth	profondeur
z	L	(m)	vertical coordinate	abscisse verticale
d	L	(m)	diameter	diamètre

A	L^2	(m^2)	area	aire
V	L^3	(m^3)	volume	volume
t	T	(s)	time	temps
v	$L\ T^{-1}$	(m/s)	velocity	vitesse
g	$L\ T^{-2}$	(m/s^2)	acceleration due to gravity ($g = 9.81\ m/s^2$)	accélération de la pesanteur ($g = 9.81\ m/s^2$)

1.6 Stress and strain
Contraintes et déformations

u	$ML^{-1}T^{-2}$	(kPa)	pore pressure	pression interstitielle
			pressure of the fluid in the voids of a porous medium (soil, geotextile)	*pression du fluide dans les vides d'un milieu poreux (sol, géotextile)*
σ	$ML^{-1}T^{-2}$	(kPa)	normal stress	contrainte normale
σ'	$ML^{-1}T^{-2}$	(kPa)	effective normal stress	contrainte normale effective
			normal stress transmitted by intergranular contacts ($\sigma' = \sigma - u$ for saturated soils)	*contrainte normale transmise par contacts intergranulaires ($\sigma = \sigma - u$ pour les sols saturés)*
τ	$ML^{-1}T^{-2}$	(kPa)	shear stress	contrainte de cisaillement
$\sigma_1, \sigma_2, \sigma_3$	$ML^{-1}T^{-2}$	(kPa)	principal stresses	contraintes principales
ε	-	(-, %)	strain, élongation	déformation, élongation
			change in length per unit length in a given direction (called elongation if expressed in % <u>and</u> applied to a geotextile)	*variation relative de longueur dans une direction donnée (appelée élongation lorsqu'elle est exprimée en % <u>et</u> appliquée à un géotextile)*
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	-	(-,%)	principal strains	déformations principales

1.7 Hydraulic parameters
Paramètres hydrauliques

h	L	(m)	hydraulic head or potential	charge hydraulique ou potentiel hydraulique
			sum of pressure height (u / γ_w) and geometrical height (z) above a given reference level	*somme de la hauteur piézométrique (u / γ_w) et de la hauteur géométrique (z) au-dessus d'un niveau de référence*
q	L^3T^{-1}	(m^3/s)	rate of discharge	débit
			volume of water seeping through a given area per unit of time	*volume d'eau percolant à travers une section donnée d'un sol, par unité de temps*
v	$L\ T^{-1}$	(m/s)	discharge velocity	vitesse d'écoulement
			rate of discharge per total unit area perpendicular to direction of flow	*débit qui s'écoule à travers une section totale unitaire du milieu, perpendiculaire à la direction de l'écoulement*

i	-	(-)	hydraulic gradient	gradient hydraulique
			loss of hydraulic head per unit length in direction of flow	*perte de charge hydraulique par unité de longueur dans la direction de l'écoulement*
j	$ML^{-2}T^{-2}$	(kN/m^3)	seepage force per unit volume	force de filtration (ou d'écoulement) par unité de volume
			force per unit volume of a porous medium generated by action of the seeping fluid upon the solid elements of the porous medium ($j = i\gamma_w$)	*force volumique exercée sur les éléments solides d'un milieu poreux par un fluide en écoulement:* ($j = i.\gamma_w$)

2. PROPERTIES OF FLUIDS

PROPRIETES DES FLUIDES

ρ_w	ML^{-3}	(kg/m^3)	density of water	masse volumique de l'eau
γ_w	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of water	poids volumique de l'eau
η_w	$ML^{-1}T^{-1}$	(kg/ms)	dynamic viscosity of water	viscosité dynamique de l'eau

NOTE: Instead of w , use any other appropriate subscript of other fluids (e.g.: η_a, dynamic viscosity of air)

A la place de w, on peut utiliser d'autres indices relatifs à d'autres fluides que l'eau (par exemple: η_a, viscosité dynamique de l'air)

3. PROPERTIES OF SOILS

PROPRIETES DES SOLS

3.1 Solid particles and their distribution

Particules solides et leur distribution

ρ_s	ML^{-3}	(kg/m^3)	density of solid particles	masse volumique des particules solides
			ratio between mass and volume of solid particles	*quotient de la masse des particules solides par leur volume*
γ_s	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of solid particles	poids volumique des particules solides
			weight of solid particles per unit volume	*quotient du poids des particules solides par leur volume*
d	L	(mm)	particule diameter	diamètre de particule
			particle size as determined by sieve analysis or wet mechanical analysis	*taille de particule déterminée dans l'analyse granulométrique par tamisage ou sédimentométrie*
d_n	L	(mm)	n percent-diameter	diamètre à n pourcent
			diameter corresponding to n percent by weight of finer particles	*diamètre correspondant a un tamisat de n pourcent sur la courbe granulométrique (n% des particules ont des dimensions inférieures à ce diamètre)*

c_U	-	(-)	uniformity coefficient defined as : d_{60}/d_{10}	coefficient d'uniformité defini par : d_{60}/d_{10}

3.2 Physical properties of soils
Propriétés physiques des sols

ρ	ML^{-3}	(kg/m^3)	density of soil *ratio between total mass and total volume of soil*	masse volumique du sol *quotient de la masse totale du sol par son volume*
γ	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of soil *ratio between total weight and total volume of soil*	poids volumique du sol *quotient du poids total du sol par son volume*
ρ_d	ML^{-3}	(kg/m^3)	density of dry soil *ratio between mass of solid particles and total volume of soil*	masse volumique du sol sec *quotient de la masse des particules solides par le volume total du sol*
γ_d	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of dry soil *ratio between weight of solid particles and volume of soil*	poids volumique du sol sec *quotient du poids des particules solides par le volume total de sol*
ρ_{sat}	ML^{-3}	(kg/m^3)	density of saturated soil *ratio between total mass and total volume of completely saturated soil*	masse volumique du sol saturé *quotient de la masse totale du sol complètement saturé par son volume total*
γ_{sat}	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of saturated soil *ratio between total weight and total volume of completely saturated soil*	poids volumique du sol saturé *quotient du poids total du sol complètement saturé par son volume total*
ρ'	ML^{-3}	(kg/m^3)	density of submerged soil *difference between density of soil and density of water*	masse volumique du sol déjaugé *différence entre la masse volumique du sol et la masse volumique de l'eau*
γ'	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of submerged soil *difference between unit weight of soil and unit weight of water*	poids volumique du sol déjaugé *différence entre le poids volumique du sol et le poids volumique de l'eau*
e	-	(-)	void ratio *ratio between volume of voids and volume of solid particles*	indice des vides *rapport entre le volume des vides et le volume des particules solides*
n	-	(-. %)	porosity *ratio between volume of voids and total volume of soil*	porosité *rapport entre le volume des vides et le volume total du sol*

w	–	(-.%)	water content *ratio between weight of pore water and weight of solid particles (expressed in percentage)*	teneur en eau *rapport entre le poids de l'eau interstitielle et le poids des grains solides (exprimé en pourcents)*
S_r	–	(-. %)	degree of saturation *ratio between volume of pore water and volume of voids*	degré de saturation *rapport entre le volume de l'eau interstitielle et le volume des vides*

3.3 Permeability of soils
Perméabilité des sols

k	LT^{-1}	(m/s)	coefficient of permeability (or hydraulic conductivity) *ratio between discharge velocity and corresponding hydraulic gradient (v / i)*	coefficient de perméabilité (ou conductivité hydraulique) *quotient de la vitesse d'écoulement par le gradient hydraulique correspondant (v / i)*

3.4 Mechanical properties of soils
Propriétés mécaniques des sols

E	$ML^{-1}T^{-2}$	(kPa)	deformation modulus *ratio between a given normal stress change and the strain change in the same direction (all other stresses being constant)*	module de déformation *quotient de la variation d'une contrainte principale par la déformation obtenue dans la même direction, les autres contraintes restant inchangées.*
υ	–	(-)	Poisson's ratio *ratio between strain changes perpendicular to and in the direction of a given uniaxial stress change*	coefficient de Poisson *rapport entre les deux déformations respectivement perpendiculaire et parallèle à la direction d'une sollicitation uniaxiale*
k_s	$ML^{-2}T^{-2}$	(kN/m^3)	modulus of subgrade reaction *ratio between change of vertical stress on a rigid plate and corresponding change of vertical settlement of the plate*	module de réaction *quotient de la variation de la contrainte verticale sur une plaque rigide par la variation correspondante de tassement vertical de la plaque*
E_{oed}	$ML^{-1}T^{-2}$	(kPa)	oedometric modulus	module oedométrique
C_c	–	(-)	compression index *slope of virgin compression curve in a semi-logarithmic plot "effective pressure-void ratio": $Cc = -\Delta e / \Delta lg\sigma'$*	indice de compression *pente de la courbe de compression vierge dans un diagramme semi-logarithmique "contrainte effective-indice des vides": $Cc = -\Delta e / \Delta lg\sigma'$*
c_v	L^2T^{-1}	(m^2/s)	coefficient of consolidation	coefficient de consolidation
T_v	–	(-)	time factor *defined as $T_v = t\, c_v / d^2$, t being the time elapsed since application of a change in total normal stress*	facteur temps *défini par $T_v = t\, c_v / d^2$, t étant le temps écoulé depuis l'application d'une variation de contrainte normale totale*

τ_f	$ML^{-1}T^{-2}$	(kPa)	shear strength	résistance au cisaillement
			shear stress at failure in rupture plane through a given point	*contrainte de cisaillement, lors de la rupture, dans le plan de rupture en un point donné*
c'	$ML^{-1}T^{-2}$	(kPa)	effective cohesion	cohésion effective
ϕ'	-	(°)	effective angle of internal friction	angle de frottement effectif
			shear strength parameter with respect to effective stresses. Defined by the equation: $\tau_f = c' + \sigma' \tan \phi'$	*paramètre de résistance au cisaillement en contraintes effectives, défini par l'équation :* $\tau_f = c' + \sigma' \text{ tg } \phi'$
c_u	$ML^{-1}T^{-2}$	(kPa)	apparent cohesion	cohésion apparente
ϕ_u	-	(°)	apparent angle of internal friction	angle de frottement apparent
			shear strength parameters with respect to total stresses. Defined by the equation: $\tau_f = c_u + \sigma \tan \phi_u$*. In undrained situation, with saturated cohesion soils,* c_u *is also called undrained shear strength*	*paramètres de résistance au cisaillement en contraintes totales, défini par l'équation:* $\tau_f = c_u + \sigma \text{ tg } \phi_u$*. Pour les sols cohérents saturés en sollicitation non drainée,* c_u *est appelé également cohésion non drainée*
CBR	-	(-)	California Bearing Ratio	indice portant californien
q_c	$ML^{-1}T^{-2}$	(kPa)	static point resistance (or cone resistance)	résistance de pointe statique (ou résistance de cône)
			average pressure acting on the conical point in the standard static penetration test	*pression moyenne agissant sur la pointe conique dans l'essai normalisé de pénétration statique*
f_s	$ML^{-1}T^{-2}$	(kPa)	local side friction	frottement latéral unitaire
			average unit side friction acting on the friction sleeve in the standard static cone penetration test	*frottement latéral par unité de surface du manchon de frottement dans l'essai normalisé de pénétration statique au cône*
N	-	(-)	SPT blow count	nombre de coups SPT
			standardized result of the Standard Penetration Test (number of blows for 30 cm)	*résultat normalisé de l'essai SPT (nombre de coups pour 30 cm)*
P_1	$ML^{-1}T^{-2}$	(kPa)	pressuremeter limit pressure	pression limite pressiométrique
			limit pressure defined in the standard Ménard pressuremeter test	*pression limite définie dans l'essai pressiométrique normal Ménard*
E_M	$ML^{-1}T^{-2}$	(kPa)	pressuremeter modulus	module pressiométrique
			conventional modulus defined in the standard Ménard pressuremeter test.	*module conventionnel défini dans l'essai pressiométrique normal Ménard*

3.5 Consistency of soils
Consistance des sols

w_L	–	(-.%)	liquid limit *water content of a remolded soil at transition between liquid and plastic states (determined by a standard laboratory test)*	limite de liquidité *teneur en eau d'un sol remanié au point de transition entre les états liquide et plastique (déterminée par un essai normalisé de laboratoire)*
w_p	–	(-, %)	plastic limit *water content of a remolded soil at transition between plastic and semi-solid states (determined by a standard laboratory test)*	limite de plasticité *teneur en eau d'un sol remanié au point de transition entre les états plastique et solide avec retrait (déterminée par un essai normalisé de laboratoire)*
w_s	–	(-, %)	shrinkage limit *maximum water content at which a reduction of water content will not cause a decrease in volume of the soil mass*	limite de retrait *teneur en eau maximale pour laquelle une réduction de teneur en eau ne cause plus de diminution de volume du sol*
I_p	–	(-,%)	plasticity index *difference between liquid and plastic limits*	indice de plasticité *différence entre les limites de liquidité et de plasticité*
I_L	–	(-,%)	liquidity index *defined as* $(w - w_p) / I_p$	indice de liquidité *défini par* $(w - w_p) / I_p$
I_C	–	(-,%)	consistency index *defined as* $(w_L - w) / I_p$	indice de consistance *défini par* $(w_L - w) / I_p$
e_{max}	–	(-)	void ratio in loosest state *maximum void ratio obtainable by a standard laboratory procedure*	indice des vides dans l'état le plus lâche *maximum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
e_{min}	–	(-)	void ratio in densest state *maximum void ratio obtainable by a standard laboratory procedure*	indice des vides dans l'état le plus dense *minimum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
I_D	–	(-)	density index *defined as* $(e_{max} - e) / (e_{max} - e_{min})$	indice de densité *défini par* $(e_{max} - e) / (e_{max} - e_{min})$

4. PROPERTIES OF GEOTEXTILES
PROPRIETES DES GEOTEXTILES

4.1 Solid elements and their distribution
Eléments solides et leur distribution

ρ_f	ML^{-3}	(kg/m^3)	density of fibers or filaments	masse volumique des fibres ou des filaments
d_f	L	(μm)	diameter of fibers or filaments	diamètre des fibres ou filaments

λ	ML^{-1}	(tex)	linear density of yarns, fibers or filaments *mass per unit length of yarns, fibers or filaments*	masse linéique (titre) des fils, fibres ou filaments *masse par unité de longueur des fils, fibres ou filaments.*
O_n	L	(mm, µm)	n-percent opening size *opening size corresponding to n-percent of finer openings (as measured by a test to be defined, such as sieving glass beads) (e.g. O_{95}, opening size corresponding to 95% of finer openings, sometimes called "Equivalent Opening Size")*	ouverture à n pourcent *dimension d'ouverture telle que n pourcent d'ouvertures soient plus petites (préciser l'essai utilisé, par exemple tamisage de billes de verre) (par exemple: O_{95}, dimension de l'ouverture telle que 95% des ouvertures soient plus petites, quelquefois appelée "Dimension d'Ouverture Equivalente")*

Note: Standard sieve numbers vary from one country to another. Opening sizes expressed by a standard sieve number would not be understood by most of the readers. Consequently, authors are strongly requested to express opening size in millimeters (mm) or microns (µm). Standard sieve numbers can be given in parenthesis after the value in mm or µm.

Note: Les numéros standards de tamis diffèrent d'un pays à l'autre. Les dimensions d'ouverture exprimées par un numéro standard de tamis ne signifieraient rien pour la plupart des lecteurs. En conséquence, il est fortement conseillé aux auteurs d'exprimer les dimensions d'ouverture en millimètres (mm) ou en microns (µm). Le numéro standard de tamis peut être donné entre parenthèses après la valeur en mm ou µm.

4.2 Physical properties of a geotextile
Propriétés physiques d'un géotextile

μ	ML^{-2}	(kg/m^2, g/m^2)	mass per unit area *ratio between mass and area of geotextile*	masse surfacique *masse par unité de surface d'un géotextile*
e	–	(-)	void ratio *ratio between volume of voids and volume of solid elements of a geotextile*	indice des vides *quotient du volume des vides par le volume des éléments solides d'un géotextile*
n	–	(-, %)	porosity *ratio between volume of voids and total volume of geotextiles*	porosité *quotient du volume des vides par le volume total d'un géotextile*
T_g	L	(mm)	thickness of geotextile	épaisseur d'un géotextile

4.3 Hydraulic properties of a geotextile
Propriétés hydrauliques d'un géotextile

k_n	LT^{-1}	(m/s)	coefficient of normal permeability of a geotextile	coefficient de perméabilité normale d'un géotextile
k_p	LT^{-1}	(m/s)	coefficient of permeability in the plane of a geotextile	coefficient de perméabilité dans le plan d'un géotextile
Ψ	T^{-1}	(s^{-1})	permittivity of a geotextile ($\Psi = k_n/H_g$)	permittivité d'un géotextile ($\Psi = k_n/H_g$)
θ	L^2T^{-1}	(m^2/s)	transmissivity of a geotextile ($\theta = k_p/H_g$)	transmissivité d'un géotextile ($\theta = k_p H_g$)

4.4 Mechanical properties of a geotextile
Propriétés mécaniques d'un géotextile

α_ε	MT^{-2}	(kN/m)	force per unit width of the geotextile at a given elongation ε (e.g. α_{30} is the force per unit width of the geotextile at 30% elongation)	force par unité de largeur du géotextile pour une élongation ε donnée (par exemple: α_{30} est la force par unité de largeur du géotextile pour une élongation de 30%)
α_f	MT^{-2}	(kN/m)	force per unit width of the geotextile at failure	force par unité de largeur du géotextile à la rupture
J	MT^{-2}	(kN/m)	"modulus" of the geotextile	"module" du géotextile
		Note:	The "modulus" of a geotextile is defined as a force per unit width while modulus is usually defined as a force per unit area.	Note: Le "module" d'un géotextile est défini comme une force par unité de largeur, alors qu'un module represente généralement une force par unité de surface.
J_i	MT^{-2}	(kN/m)	initial (tangent) "modulus" of the geotextile	"module" (tangent) initial du géotextile
$J_{t\varepsilon}$	MT^{-2}	(kN/m)	tangent "modulus" of the geotextile at elongation ε (e.g. J_{t30} is the tangent "modulus" of the geotextile at 30% elongation)	"module" tangent du géotextile pour une élongation ε donnée (par exemple J_{t30} est le "module" tangent du géotextile pour une élongation de 30%)
$J_{sec\ \varepsilon}$	MT^{-2}	(kN/m)	secant "modulus" of the geotextile between 0 and elongation ε (e.g. $J_{sec\ 30}$ is the secant "modulus" of geotextile between 0 and 30% elongation)	"module" sécant du géotextile entre 0 et l'élongation ε (par exemple, $J_{sec\ 30}$ est le "module" sécant du géotextile entre 0 et 30% d'élongation)
υ_g	-	(-)	Poisson's ratio of a geotextile	coefficient de Poisson d'un géotextile
ζ_t	M^2T^{-2}	(N/tex)	tenacity of a thread *quotient of the tensile force at failure of a thread to its linear density*	tenacité d'un fil *quotient de la force de traction à la rupture d'un fil par sa masse linéique*
ζ_g	M^2T^{-2}	(J/g)	tenacity of a geotextile *quotient of the force per unit width at failure of a geotextile to its mass per unit area*	tenacité d'un géotextile *quotient de la force par unité de largeur d'un géotextile à la rupture par sa masse surfacique*
			(Note: 1 J/g = 1000 N/tex)	
F_G	MLT^{-2}	(N,kN)	breaking force of geotextile as measured in grab test	résistance du géotextile dans l'essai d'arrachement
F_P	MLT^{-2}	(N, kN)	breaking force of geotextile in a puncture test (to be defined)	résistance du géotextile à la perforation (préciser l'essai utilisé)
F_T	MLT^{-2}	(N, kN)	breaking force of geotextile in a tear test (to be defined)	résistance du géotextile à la déchirure (préciser l'essai utilisé)
P_B	$ML^{-1}T^{-2}$	(kPa, MPa)	bursting pressure of a geotextile.	résistance du géotextile à l'éclatement

5. PRACTICAL PROBLEMS
PROBLEMES PRATIQUES

5.1 General
Généralités

FS	–	(-)	Factor of safety	Coefficient de sécurité

5.2 Earth pressure
Pression des terres

δ	–	(^{o})	angle of wall friction	angle de frottement sol-mur
			angle of friction between wall and adjacent soil	*angle de frottement entre le mur et le sol adjacent*
a	$ML^{-1}T^{-2}$	(kPa)	wall adhesion	adhésion sol-mur
			adhesion between wall and adjacent soil	*adhésion entre le mur et le sol adjacent*
K_a, K_p	–	(-)	active and passive earth pressure coefficients	coefficients de poussée et de butée des terres
			dimensionless coefficients used in expressions for active and passive earth pressure	*coefficients sans dimension intervenant dans les expressions de poussée et de butée*
K_o	–	(-)	coefficient of earth pressure at rest	coefficient de pression des terres au repos
			ratio of lateral to vertical effective principal stress in the case of no lateral strain and a horizontal ground surface	*rapport entre les contraintes effectives horizontale et verticale à déformation horizontale nulle et lorsque la surface libre du sol est horizontale*

5.3 Foundations and embankments
Fondations et remblais

B	L	(m)	breadth of foundation or embankment	largeur de la fondation ou du remblai
L	L	(m)	length of foundation or embankment	longueur de la fondation ou du remblai
D	L	(m)	depth of foundation or base of embankment beneath ground	profondeur de la fondation ou de la base du remblai au-dessous du niveau du terrain
s	L	(m)	settlement	tassement
U	-	(-,%)	degree of consolidation	degré de consolidation
			ratio of settlement at a given time to final settlement	*quotient du tassement à un temps donné par le tassement final*

5.4 Slopes
Pentes

H	L	(m)	vertical height of slope	hauteur verticale du talus
D	L	(m)	depth below toe of slope to hard stratum	profondeur du substratum rigide sous le pied du talus
β	–	(^{o})	angle of slope to horizontal	angle d'inclinaison du talus avec l'horizontale

Conversion factors
Facteurs de conversion

Conversion factors between the English system and SI are as follows:

Length, area, volume

1 mil	= 25.4 microns (µm)
1 inch	= 25.4 millimeters (mm)
1 foot	= 0.305 meter (m)
1 yard	= 0.914 meter (m)
1 square foot	= 9.29×10^{-2} square meters (m^2)
1 acre	= 4047 square meters (m^2)
1 cubic foot	= 2.83×10^{-2} cubic meters (m^3)
1 gallon	= 3.79×10^{-3} cubic meters (m^3)
1 gallon	= 3.79 liters (L)

Mass

1 ounce	= 28.3 grams (g)
1 pound	= 0.454 kilogram (kg)
1 ton (US)	= 907 kilograms (kg)
1 long ton (GB)	= 1016 kilograms (kg)

Linear Density

1 denier	= 0.111 milligram per meter (mg/m)
1 denier	= 0.111 tex (tex)

Mass per unit area

1 ounce per square yard	= 33.9 gram per square meter (g/m^2)
1 ounce per square yard	= 3.39×10^{-2} kilogram per square meter (kg/m^2)

Density

1 pound (mass) per cubic foot	= 16.0 kilograms per cubic meter (kg/m^3)

Force

1 pound	= 4.45 newtons (N)
1 kip	= 4.45 kilonewtons (kN)

Force per unit width or per unit length

1 pound per inch	= 175 newtons per meter (N/m)
1 pound per inch	= 0.175 kilonewton per meter (kN/m)

Stress, pressure

1 pound per square inch	= 6895 pascals (Pa)
1 pound per square inch	= 6.89 kilopascals (kPa)
1 pound per square foot	= 47.9 pascals (Pa)
1 kip per square foot	= 47.9 kilopascals (kPa)
1 ton (US) per square foot	= 95.8 kilopascals (kPa)

Unit weight or reaction modulus

1 pound (force) per cubic foot	= 157 newtons per cubic meter (N/m^3)
1 pound (force) per cubic foot	= 0.157 kilonewton per cubic meter (kN/m^3)

Velocity, Coefficient of Permeability

1 centimeter per second	= 0.010 meter per second (m/s)

Flow rate

1 gallon per minute	= 6.31×10^{-2} liter per second (L/s)
1 gallon per minute	= 6.31×10^{-5} cubic meter per second (m^3/s)
1 cubic foot per second	= 2.83×10^{-2} cubic meter per second (m^3/s)
1 cubic foot per minute	= 0.472 liter per second (L/s)
1 cubic foot per minute	= 4.72×10^{-4} cubic meter per second (m^3/s)
1 gallon per day	= 4.38×10^{-5} liter per second (L/s)

Dynamic viscosity

1 poise	= 0.100 kilogram per meter-second (kg/sm)
1 pound (force) second per square foot	= 0.478 kilogram per meter-second (kg/sm) (Note: 1 kg/sm = 1 Pa.s)

Gravity

g = 32.2 feet per square second	= 9.81 meters per square second (9.81 m/s^2)

U. S. Sieve Designation	Opening Size mm	µm
200	0.075	75
170	0.090	90
140	0.106	106
120	0.125	125
100	0.150	150
80	0.180	180
70	0.212	212
60	0.250	250
50	0.300	300
45	0.355	355
40	0.425	425
35	0.500	500
30	0.600	600
25	0.710	710
20	0.850	850
18	1.000	
16	1.18	
14	1.40	
12	1.70	
10	2.00	
8	2.36	
7	2.80	
6	3.35	
5	4.00	
4	4.75	

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Volume IV

Volume IV (to be published January 1983) includes the following: the proceedings of the opening and closing sessions, the transcripts of formal questions and answers, an index of authors and paper topics, and errata.

Le volume IV (à paraître en janvier 1983) comprend: les comptes rendus des sessions d'ouverture et de clôture, les transcriptions des questions et réponses, les errata et un index des auteurs et titres de communications.

PROCEEDINGS
COMPTES-RENDUS

Second International Conference on Geotextiles

Deuxième Congrès International des Géotextiles

August 1–6, 1982
1–6 Août, 1982
MGM Grand Hotel
Las Vegas, Nevada, U.S.A.

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The Selection of Testing Procedures for the Specification of Geotextiles

La seléction de procédures d'essai pour les specifications des géotextiles

The considerable growth in the application of geotextile for civil engineering works has led to a demand for appropriate specifications. To assist in producing these specifications a joint programme of research is being carried out by the Transport and Road Research Laboratory and the University of Strathclyde to select suitable testing methods for both design and quality control purposes. In this paper the basis for the approach adopted in carrying out the research is discussed and the present position outlined with regard to recommended testing methods and those procedures which are as yet to be defined.

La forte croissance de l'application de géotectiles aux travaux publics a entraîné une demande pour des spécifications appropriées. Pour faciliter la production de telles spécifications, le Laboratoire de Recherches sur les Transports et les Chaussees, conjointement avec l'Université de Strathclyde, est en train d'effectuer un programme commun de recherches avec le but de sélectionner des méthodes d'essai convenables en ce qui concerne non seulement la conception, mais aussi le contrôle de qualité. Dans cet article on examine la base de la méthode de recherches suivie, et on fait un bref compte rendu de la situation actuelle en ce qui concerne les méthodes d'essai recommandées, et d'autres procédures qui ne sont pas encore définies.

INTRODUCTION

During the past decade, considerable growth has occurred in the application of geotextiles in civil engineering works, particularly in the United Kingdom. Initially these applications were related to works of a temporary nature such as, contractor's access roads having a life of only a few years at most. More recently, however consideration has been given to their use for permanent works such as retaining structures and embankment stabilisation, where the required life can exceed one hundred years. For works of a permanent nature, those engineering structures which utilise geotextiles for their integrity must compete on equal terms with traditional type structures constructed from material which have well established techniques for their design and specification. To ensure that an alternative mode of construction which involves geotextiles is offering an equivalent structure to the traditional versions, in terms of both life and functional performance, it is essential that the required characteristics of geotextiles be properly specified. Moreover, appropriate testing techniques must be available to provide the required design data and to ensure that the quality of the product is being maintained.

In this paper the requirements for geotextile specifications are considered together with the approach adopted in a joint research porgramme being undertaken by the Transport and Road Research Laboratory and the University of Strathclyde to establish suitable testing techniques and develop design procedures on which appropriate specifications may be based.

REQUIREMENTS OF SPECIFICATIONS FOR GEOTEXTILES

Although there are a considerable number of applications for geotextiles in civil engineering works (Fig. 1), their primary functions can always be identified as including one or more of the following categories (1,2):

(i) Separation

(ii) Filtration

(iii) Drainage-in-the-plane

(iv) Reinforcement

The importance of the engineering and physical properties of geotextiles in relation to the basic functions is detailed in Table 1. It is apparent from this that the information needed to specify a geotextile for each of the functions will be rather different. A further complication in respect of the specifications is that product quality can be significantly affected both at the manufacturing stage and subsequently, hence appropriate testing procedures are needed to ensure that the minimum acceptable quality is maintained. Thus the basic requirements of specifications for geotextiles should include the following items:

(a) Identification of design procedures to be followed for specific applications.

(b) Limiting values of geotextile properties, measured according to standard test procedures, which may be adopted in a design.

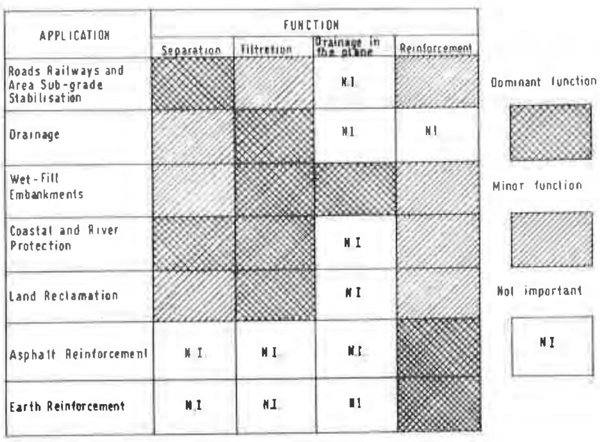

Fig. 1. THE BASIC FUNCTIONS OPERATING IN VARIOUS APPLICATIONS.

(c) Procedures for transporting, storing and handling geotextiles.

(d) Construction or installation procedures for civil engineering works utilising geotextiles for their integrity.

(e) Limiting values of geotextile properties, measured according to standard test procedures, for the purposes of quality control.

Clearly, some of the above items will be common to a range of geotextile applications, thus reducing the number of test methods needed generally and also simplifying the procedure for developing specifications for particular applications.

PROCEDURES AND ENVIRONMENTAL CONDITIONS FOR SPECIFICATION TESTING

For major works a contractural specification will usually involve a comprehensive programme of testing as outlined in Fig. 2. Different environmental conditions and prior treatments may be involved for the quality control and the design data testing of geotextiles, thus it may not be appropriate for the information obtained from any particular test to be used for both purposes. In any case the tests required to provide information for a design will often be more complex than those used for quality control purposes and the design data tests may need to be carried out in specialised soil or textile laboratories where close environmental control can be maintained to simulate, as far as possible, the field conditions.

TABLE 1. PROPERTIES IMPORTANT IN BASIC FUNCTIONS.

GROUP	PROPERTIES	FUNCTIONS			REMARKS
		SEPARATION	FILTRATION AND DRAINAGE	REINFORCEMENT	
1. CONSTRUCTIONAL AND BASIC PHYSICAL PROPERTIES	MATERIAL CONSTITUENTS	NOT IMPORTANT	NOT IMPORTANT	MINOR + DOMINANT	CONSTITUENT MATERIALS CAN BE THE DOMINANT FACTOR CONTROLLING STRESS-STRAIN BEHAVIOUR AND SURFACE FRICTION
	METHOD OF MANUFACTURE	MINOR + DOMINANT	DOMINANT	DOMINANT	THIS DETERMINES THE STRUCTURE WHICH CONTROLS MANY PROPERTIES HENCE IT MUST BE CAREFULLY CONTROLLED
	MASS PER UNIT AREA	DOMINANT	"	"	A GOOD MEASURE OF THE CONSISTENCY OF THE PRODUCT QUALITY
	PORE SIZES	"	"	NOT IMPORTANT	OF PARTICULAR IMPORTANCE FOR SEPARATION AND FILTRATION AND DETERMINES THE CAPABILITY OF GETEXTILES TO RETAIN SOIL INTERNALLY OR AT THE SOIL-GEOTEXTILE INTERFACE
	OPEN AREA	"	"	" "	
	THICKNESS	"	"	" "	COMPRESSION IN SOIL AND REORIENTATION OF FIBRES DURING STRAINING CHANGES THICKNESS AND STRUCTURE WHICH MODIFIES MANY OF THE GEOTEXTILES PROPERTIES
	RIGIDITY OF STRUCTURE	"	"	DOMINANT	
2. MECHANICAL AND HYDRAULIC PROPERTIES	OVERALL STRESS-STRAIN BEHAVIOUR	MINOR	MINOR	DOMINANT	REINFORCEMENT FUNCTION IS DEPENDENT ON ALL THESE PROPERTIES. ALSO THE ENTIRE STRESS-STRAIN-TIME RELATIONSHIP AND NOT JUST PEAK LOAD AT A STANDARD STRAIN RATE IS SIGNIFICANT
	OVERALL EXTENSION TO RUPTURE	MINOR	MINOR	DOMINANT	
	CREEP AND STRESS RELAXATION	NOT IMPORTANT	NOT IMPORTANT	DOMINANT	
	LOCAL BURST STRENGTH	DOMINANT	DOMINANT	MINOR + DOMINANT	THE ABILITY TO RESIST AND REDISTRIBUTE LOCAL LOADS ARE TWO OF THE PRINCIPAL BENEFITS OF GEOTEXTILES AND MUST BE ASSOCIATED WITH BURST AND TEAR RESISTANCE
	TEAR PROPAGATION	DOMINANT	DOMINANT	DOMINANT	
	SURFACE FRICTION/ADHESION	NOT IMPORTANT	NOT IMPORTANT	DOMINANT	TO MOBILISE TENSILE STRENGTH THERE MUST BE SUFFICIENT SURFACE FRICTION/ADHESION
	FLUID FLOW CAPACITY PER UNIT AREA OR UNIT THICKNESS	NOT IMPORTANT	DOMINANT	NOT IMPORTANT	THE CAPABILITY OF GEOTEXTILES TO ALLOW WATER TO FLOW THROUGH THEM IS FUNDAMENTAL TO ALL FILTRATION AND DRAINAGE FUNCTIONS
3. ENVIRONMENTAL PROPERTIES	DURABILITY	DOMINANT	DOMINANT	DOMINANT	GEOTEXTILES MUST BE CAPABLE OF MAINTAINING THEIR FUNCTIONAL PERFORMANCES WITH TIME AND THEREFORE MUST ACCOMMODATE THE ENVIRONMENT CONDITIONS IN WHICH THEY ARE REQUIRED TO PERFORM
	TEMPERATURE STABILITY	"	"	"	
	CHEMICAL STABILITY	"	"	"	
	U.V. LIGHT STABILITY	"	"	"	

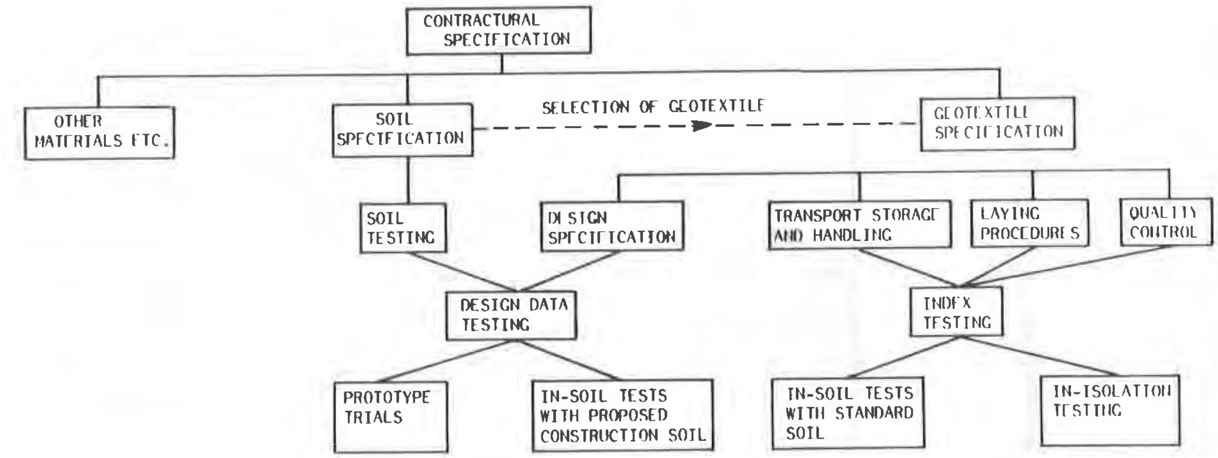

Fig. 2. THE COMPONENTS OF CONTRACTURAL SPECIFICATIONS FOR SOIL-GEOTEXTILE SYSTEMS.

On these bases four main areas of testing may be identified:

(a) Soil testing: The engineering properties of the soil to be used should be determined to identify the required geotextile properties. This should include chemical tests to ensure that materials liable to attack geotextiles are not present in significant amounts.

(b) In-isolation tests: The geotextile sample for testing may be unconfined or confined as indicated in Fig. 3. The laboratory environmental conditions for such tests should be standardised and in the United Kingdom be in accordance with BS 1051:1972 (3), which specifies a standard temperature of 20 °C ± 2 °C and relative humidity of 65 ± 5 per cent. At present it is considered that most in-isolation tests have very little relevance for the purposes of design although it may be possible with time and extensive usage to relate certain test results to the performance of structures utilising geotextiles and so produce empirical design data.

(c) In-soil tests: These tests are carried out with the geotextile confined in-soil as shown in Fig. 3. For design purposes the soil employed should be that proposed for the construction works although the use of a standard soil may be of value for obtaining relative performance data as well as for quality control purposes.

In carrying out these tests for design applications the temperature conditions should ideally correspond to those anticipated on site, Fig. 4. In the United Kingdom the average annual temperature for foundation soils is approximately 10 °C. Relative humidity can vary greatly on site, therefore it is judged best to carry out design data tests on geotextiles in a fully saturated condition.

Those in-soil tests which involve the use of standard soils and all in-isolation tests on geotextiles are collectively referred to as Index Tests. They should generally be carried out in the United Kingdom at the standard environmental conditions detailed in BS 1051: 1972 (3).

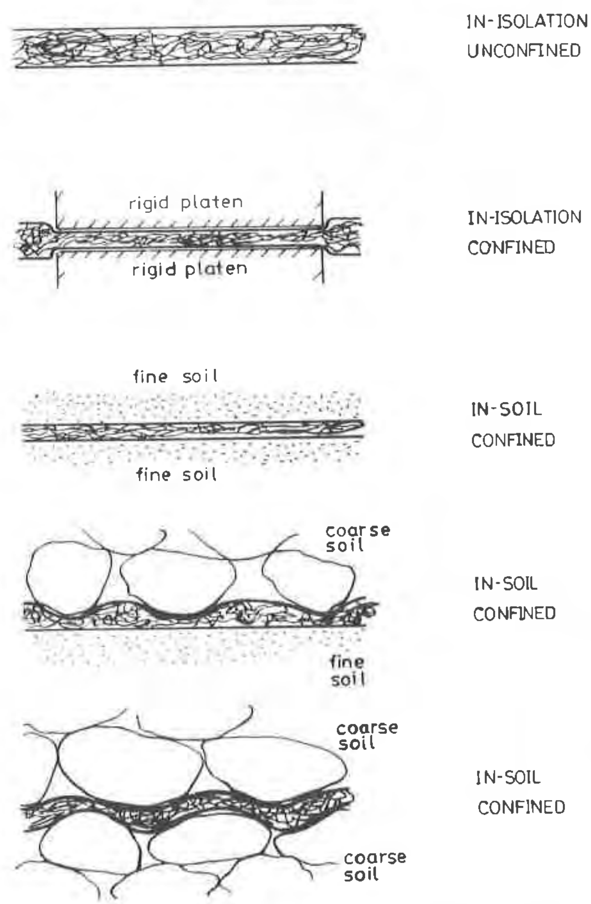

Fig. 3. THE VARIOUS UNCONFINED AND CONFINED CONDITIONS FOR GEOTEXTILES.

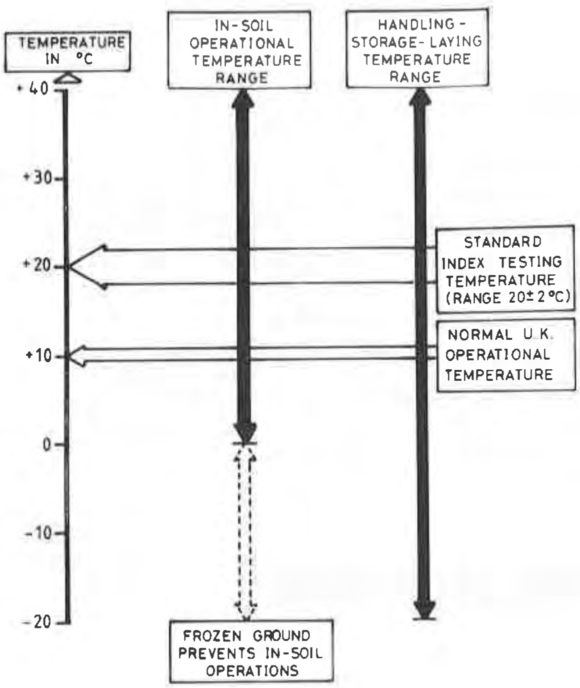

Fig. 4. TEMPERATURE CONDITIONING FOR TESTING.

(d) Prototype trials: Large scale laboratory tests or field trials may be used to measure the overall performance of soil-geotextile systems and the resulting data used directly in design providing that the environmental conditions of the trial conform to those on site. Alternatively, it may be possible to asses the performance of the soil and geotextile components of the system separately for design purposes.

TEST METHODS TO OBTAIN DESIGN DATA

The range of tests required is listed in Table 2 and it can be seen from this that all the necessary tests are not yet identified although the number of additional tests is expected to be fairly small. However, an important requirement will be to ensure that all of the test methods finally adopted permit the soil-geotextile behaviour to be assessed at the stress levels and environmental conditions anticipated on site.

Because of the difficulty of site conditions and the complexity of the soil-geotextile-structure interaction problems, it may be essential on some occasions to obtain design data from field trials. A further need for such trials may arise because of construction effects, which are generally too difficult to simulate in laboratory tests, will sometimes dominate soil-geotextile system behaviour. Further aspects of the tests required to obtain design data are considered to be as follows:

(a) Particle size retention: No suitable procedure has as yet been developed but what is required is a method which permits the soil retention properties of geotextiles to be measured during steady state, transient and reversing flow conditions, both across and along the geotextile. However, it may prove possible to interpret the data from fluid flow tests to obtain this information.

TABLE 2. IN-SOIL TESTS TO OBTAIN DESIGN DATA

PROPERTIES	REFERENCE TO TESTS METHODS PROPOSED	FUNCTIONS IN WHICH TEST DATA USED			
		SEPARATION	FILTRATION	DRAINAGE	REINFORCEMENT
SOIL PARTICLE RETENTION	NOT YET IDENTIFIED	*	*	*	-
OVERALL LOAD-EXTENSION DURING FIRST TIME LOADING	THE BASIC METHOD IS THAT DESCRIBED BY McGOWN, ANDRAWES, WILSON-FAHMY AND BRADY(1981) AND MODIFIED FOR CYCLIC LOADING AND CREEP/STRESS RELAXATION AS DESCRIBED BY McGOWN, ANDRAWES AND KABIR (1982)	-	-	-	*
CYCLIC LOADING		-	-	-	*
CREEP AND STRESS RELAXATION		-	-	-	*
SURFACE FRICTION/ ADHESION	SHEAR BOX TEST SIMILAR TO THAT OF THE R.I.L.E.M. RECOMMENDATION. SM-G-13 AND ASTM PROPOSED "SLIDING RESISTANCE"	-	-	-	*
FLUID FLOW CAPACITY ACROSS THE GEOTEXTILE	THE METHOD DESCRIBED BY McGOWN, MURRAY AND KABIR (1982) IS PRESENTLY USED BUT A MODIFIED VERSION OF THE APPARATUS SIMILAR TO THAT DESCRIBED BY FIERZ, MARTIN AND DURST (1980) IS LIKELY TO REPLACE THE APPARATUS NOW IN USE	-	*	-	-
FLUID FLOW CAPACITY ALONG THE GEOTEXTILE		-	-	*	-

(b) Load-extension tests: Apparatus has been developed, as part of the current study (4, 5), for obtaining the load-extension properties of geotextiles in soil. The maximum width of geotextile specimen which can be tested is 500 mm and the soil particles applying the confining stress may be up to 25 mm diameter. At present the upper limit of confining pressure which can be applied is 250 kN/m^2. Modified versions of the apparatus permit tests to be carried out involving cyclic loading, stress relaxation and creep.

(c) Surface friction/adherence properties: The use of geotextiles for reinforcement purposes requires that their tensile strength is mobilised through friction at the interface between the soil and geotextile. the friction/adhesion parameters are most commonly obtained using the direct shear apparatus. One approach involves confining the geotextile between the two halves of the shear box containing the soil proposed for use in the construction works.

(d) Fluid flow capacity: Apparatus for obtaining this property is described in a companion paper to this conference (6). At present circular test specimens are employed but as these can introduce edge leakage problems it is proposed to develop a further version utilising square specimens. The apparatus would then be similar to that described by Fierz et al (7).

QUALITY CONTROL TESTS

The object of these tests is to ensure that minimum standards defined according to a manufacturer's specification are maintained both during manufacture and subsequently after transportation, storage and handling. The tests would normally be simple to perform, of low cost and able to be carried out either in standard geotechnical and textile laboratories or on site. The tests appropriate to a particular function for which a geotextile is employed are listed in Table 3 and, although only a selection of those tests would require to be carried out in normal circumstances, such tests would need to be done on a regular and frequent basis.

Attempts are sometimes made to use data from quality control tests for design purposes. Generally such data areinappropriate as the test procedures rarely simulate the field conditions. More importantly, the influence of environmental conditions, soil confining stresses, porewater pressures and other aspects of soil behaviour can produce significant effects which will often necessitate in-soil testing.

DISCUSSION

During the past four years since the study was initiated many of the procedures for testing geotextiles have been established and the problems associated with most of those remaining have been identified.

TABLE 3. INDEX TESTS FOR QUALITY CONTROL PURPOSES

PROPERTY	TEST METHODS LIKELY TO BE ADOPTED	FUNCTIONS IN WHICH TEST DATA USED				REMARKS
		SEPARATION	FILTRATION	DRAINAGE	REINFORCEMENT	
MASS PER UNIT	R.I.L.E.M. RECOMMENDATION SM-G-6 "MASS PER UNIT AREA"	*	*	*	*	–
PORE SIZES	R.I.L.E.M. RECOMMENDATION SM-G-8 "PORE SIZE MEASUREMENT" (BOTH DRY AND WET SIEVING TECHNIQUES EMPLOYED)	*	*	*	*	PROBLEMS OF TESTING THICK NON-WOVENS EXIST WITH DRY SIEVING TECHNIQUES
OPEN AREA	DIRECT OBSERVATION, CALHOUN (1972) (8)	*	*	*	*	ONLY WOVENS AND VERY THIN NON-WOVENS CAN BE TESTED
NOMINAL THICKNESS	R.I.L.E.M. RECOMMENDATION SM-G-7 "NOMINAL THICKNESS"	*	*	*	*	–
COMPRESSIBILITY	COMPRESSION BETWEEN TWO RIGID PLATES McGOWN, MURRAY and KABIR (1982)	*	*	*	*	–
OVERALL LOAD-EXTENSION IN FIRST TIME LOADING IN-ISOLATION	UNIAXIAL TENSILE TEST ON SPECIMENS OF 200 MM MINIMUM WIDTH AND ASPECT RATIO OF 2:1 (WIDTH: HEIGHT)	–	–	–	*	SIMILAR TO ASTM PROPOSAL "WIDE WIDTH TEST"
LOCAL BURST STRENGTH	MULLEN BURST TEST, B.S.4768(1972) (9)	*	*	*	*	DIFFICULTIES EXIST WHEN TESTING VERY STRONG GEOTEXTILES IN THIS APPARATUS
TEAR PROPAGATION	R.I.L.E.M. RECOMMENDATION SM-G-12. "TEAR RESISTANCE" OR ASTM PROPOSAL"	*	*	*	*	FURTHER TESTING IS REQUIRED BEFORE A DECISION ON WHICH TEST TO ADOPT IS MADE
SURFACE FRICTION	SHEAR BOX TESTS CARRIED OUT IN THE MANNER OF R.I.L.E.M. RECOMMENDATION SM-G-13 AND ASTM PROPOSAL "SLIDING RESISTANCE"	–	–	–	*	THE R.I.L.E.M. AND ASTM TESTS ARE VERY SIMILAR IN THIS CASE
PERMITTIVITY	R.I.L.E.M. RECOMMENDATION SM-G- 9. "HYDRAULIC PERMITTIVITY"	–	–	*	–	THIS TEST IS NOT CONSIDERED SUITABLE FOR DESIGN DATA PURPOSES
TRANSMISSIVITY	R.I.L.E.M. RECOMMENDATION SM-G-10. "HYDRAULIC TRANSMISSIVITY"	–	–	*	–	(AS ABOVE)
DURABILITY	NOT YET IDENTIFIED	*	*	*	*	–
TEMPERATURE STABILITY	NOT YET IDENTIFIED	–	–	–	*	–
CHEMICAL STABILITY	NOT YET IDENTIFIED	*	*	*	*	–
U.V. LIGHT STABILITY	UNLIKELY TEST PROCEDURES WILL BE INCLUDED AS EXTENDED EXPOSURE OF GEOTEXTILES TO U.V. LIGHT WILL NOT BE PERMITTED IN THE SPECIFICATIONS TRANSPORTING, STORAGE AND HANDLING OR LAYING PROCEDURES					

It has been demonstrated during the course of the study that in order to obtain reliable design data, it will often be necessary to combine the geotextile with the proposed soil in specially developed apparatus to simulate, as far as possible, the anticipated behaviour of a structure. One such apparatus which has been developed for assessing load-extension properties has shown that the soil confining stress significantly affects the behaviour of many geotextiles.

A selection of index,(quality control) tests have been proposed which are intended to ensure that minimum standards based on a manufacturer's specification are being mainatined, both during and after manufacture. These tests are selected largely for their simplicity, low cost and facility for laboratory and field application. At the present time the results of index tests are considered to be of little value for design purposes, but as experience is gained in using these tests and from measured soil-geotextile systems performance, it may prove possible to produce empirical design data on the basis of data from Index tests. As yet no specific tests unique to the storage, handling and laying of geotextiles have been identified with the exception of strength testing of overlaps and joints which will be tested in the in-soil load-extension apparatus. However, it may be necessary to check that operations do not adversely affect geotextile properties and quality control tests will be appropriate for this purpose.

It is hoped that the approach adopted of making a clear distinction between tests needed for design purposes and those for application only to quality control purposes will assist in simplifying specifications for the use of geotextiles in the United Kingdom and elsewhere. Because index tests are to a large extent independent of the geotextile application, the preparation of an appropriate specification for a scheme may then concentrate mainly on appropriate testing procedures for obtaining design data.

ACKNOWLEDGEMENTS

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Recommendations on Applications, Testing and Classification of Geotextiles in Road Construction in Germany

Recommandations pour les applications, essais, et classifications des géotextiles pour la construction routière en Allemagne

The working party "Geotextiles" of the German Road and Transportation Research Association is engaged in the preparation of guidelines for the use of geotextiles in road construction.

The following points will be covered:

- definition and explanation of terms, and of the influence of material specific·parameters on product properties.
- description of applications of geotextiles with rules for their use
- details on the product description required of the producer
- the setting up of a programme of testing
- rules for selection (design, classification)

Le comité 'Géotextiles' de l'Association des Recherches sur les Routes et sur les Transports s'occupe de la préparation d'une notice pour l'emploi des géotextiles en construction routière.

Les points suivants seront inclus:

- définition et explication de la terminologie, et des propriétés du géotextile relatifs au materiaux utilisés.
- description des applications des géotextiles avec des règles pour leur utilisation.
- détails de la déscription du produit requise du producteur.
- établissement d'un programme d'essais
- règles pour la sélection (calcul, classification)

0. ORGANIZATION OF RESPONSIBILITIES IN THE GERMAN FEDERAL REPUBLIC

Two bodies are currently engaged in work on geotextiles in Germany. One, a working party of the German National Society of Soil Mechanics and Foundation Engineering, is concerned with the use of geotextiles in the broad field of soil mechanics and hydraulic engineering.

The second group, under the auspices of the German Road and Transportation Research Association, is a working party named "Geotextiles", and it is this group whose work is reported on here. The group is made up of representatives from fabric manufacturers, consulting engineers, academic institutions, the construction industry and the highway administration.

The brief of this working party is to draw up guidelines which:

- describe possible applications of geotextiles
- indicate how they are to be used, or set up rules on their installation
- give details on testing
- set up a basis for selection
- provide ruling on contractual matters concerning geotextiles.

Certain sections are already complete in draft form, while others are still under discussion at the time of writing (March 1982). These latter will be presented in more detail at Las Vegas.

The aim of these recommendations is not only to regulate constuction methods already practised in Germany but also to indicate new "geotextile" solutions to construction problems and to clarify conditions and constructions methods for these. For example, the use of geotextiles as separators under base layers or embankments is already commonplace, whereas German highway engineers still make little use of geotextiles' potential in increasing factors of safety of embankments or of the technical and practical improvements they have to offer in the construction of drainage systems.

1. CLARIFICATION OF GEOTEXTILES AS CONSTRUCTION MATERIALS

The Guidelines begin with three chapters which set out their scope, define terminology for those unfamiliar with textiles and indicate the influence of raw material and construction type on the geotextiles' properties. Non-wovens, wovens, and composite materials are covered - geomembranes are not.

Only textiles made from synthetic fibres are used. The mechanical and hydraulic properties of woven fabrics are determined principally by the fabric construction and fixing of the fibres or yarns. Non-wovens are used principally as separators and filters; because of the random orientation of their fibres their mechanical properties are more or less isotropic. When non-wovens are subjected to tension, some of the fibres are stressed immediately, others orientate themselves along the direction of stress. This results in a high extensibility in comparison to wovens, and the less the fibres are fixed in place, the more extensible the non-woven. Depending on their extensibility, non-wovens conform to an uneven sub-grade or to an unevenly developing interface between a soft sub-grade and a granular layer whose settlement is non-uniform. In the case of local damage, for example if a stone punches through, fibres bunch round the damaged area. The continuity of the surrounding fabric is not destroyed.

The filtration properties are characterized by the effective opening size and the load-dependent permeability. Extension or lateral compression of a non-woven have a negligible effect on its effective opening size.

The reduction in permeability under load and due to partial clogging by fine soil particles must be taken into consideration.

Non-wovens can also be used to transport water in the plane of the fabric.

Woven geotextiles should be used primarily where a reinforcing effect is required. The mechanical properties of wovens are characterized by the arrangement of yarns in warp and weft (fill) directions and are thus directionally dependent. If one or more yarns are broken the fabric loses a part of its strength in that direction.

The filtration properties of wovens are characterized by the effective opening size and the load-dependent permeability. Lateral compression of the fabric has little effect on its filtration properties; extension can cause the pores of a woven to open up. A possible reduction in permeability when in contact with soil must be taken into consideration in filter design.

Composite materials are used where a combination of the properties of non-wovens and wovens is required.

2. END-USE AREAS

Three principal end-use areas for geotextiles are considered:

- separation under soil layers
- stabilization of artificial slopes
- filtration in drainage systems

For each of these areas, typical examples are illustrated and selection criteria and practical construction advice are given.

3. REQUIREMENTS

The requirements placed on geotextiles vary according to their function and to the nature and duration of the loads etc. imposed during installation, the consolidation phase, and thereafter.

As a rule the greatest demands are placed on the geotextile during installation e.g. through transport, tipping of angular aggregate and loads imposed by construction traffic on the first sub-base layer ("kneading" or "rolling" stresses). UV-radiation may lead to a reduction of strength when geotextiles are left uncovered.

When used under granular layers on soft sub-grades, a geotextile may on occasion be locally very differentially stressed during the consolidation phase. Its function must not be impaired as a result (extensibility). When consolidation is complete the geotextile ceases to function as a separator except in the case of an embankment constructed with coarse, poorly graded material which may be contaminated by fines from below.

When geotextiles are used as reinforcing elements, shear and tensile loads must be accomodated at extensions which the structure can tolerate. Creep must also be considered.

Highway drainage requires geotextiles with good filtration properties for the soil to be drained. These properties must also remain adequate after long contact with soil (fines washed into the fabric), and under lateral pressure. If the water contains iron or calcium salts then a reduction in the hydraulic conductivity of a drainage system due to iron or calcium deposits must be anticipated.

4. TESTING

Testing of road construction materials takes place on three levels in Germany: the properties of a geotextile which are important for its use in road construction are determined in suitability testing. Product variability is monitored by producer quality control testing. Check testing is carried out by the client for a particular project to ensure compliance with specification.

Scope of suitability testing

- weight per square metre including coefficient of variation
- thickness (DIN 53855, loaded area 20 cm², pressure 2 kN/m²)
- tensile strength and load/extension behaviour in the strip tensile test (with limited sample necking) (non-wovens)
- max. load and extension at max. load in machine and cross directions (DIN 53857) with load/extension curve wet and dry (wovens)
- plunger push-through force (DIN 54307 "CBR Test")
- dynamic punture test
- relative coefficient of friction (*)
- resistance to sunlight (*)
- creep (*)
- determination of effective opening size
- determination of water permeability under loads of 2 - 200 kN/m²

Tests marked with an asterisk (*) are only to be carried out when their results are decisive for the application in question.

Scope of producer quality control testing

- weight per square metre
- thickness
- max. load and extension at max. load (DIN 53857)

Scope of check testing

- weight per square metre
- thickness
- CBR test

5. DESCRIPTION OF PRODUCT

The producer description of all geotextiles should cover at least the points listed below, and is complemented by the suitability testing. In addition it is required that the producer's name, fabric type and date of production be printed recurringly along the edge of the fabric.

Product details:

- name of producer
- product name
- where produced

- nominal weight per square metre (DIN 53854) with production tolerance
- thickness (DIN 53855, area 20 cm^2, pressure 2 kN/m^2)

  a) fibre raw material and melting point
  b) for fibre mixtures: ratio by weight % and details as in a)
  c) for bi-component fibres: type (e.g. core/sheath, side by side, etc.); ratio by weight % and details as in a).

- type of binder (including melting point if applicable e.g. fibre sheath or bonding fibre)
- fibre length
- fibre diameter
- filament or yarn
- fibre orientation (orientated, cross-laid or randomly-orientated non-woven)
- type of bonding
- combination with nets, films, threadlines etc.
  a) net etc. type and raw material
  b) details of direction of net etc.

6. CLASSIFICATION FOR DESIGN

For certain end use areas it is possible to quantify the demands placed on a geotextile and thus to dimension it. These areas include the use of geotextiles as filters and as reinforcing elements. The working party has drawn up filter criteria; the user is referred to technical literature on reinforcement.

However the largest fields of application - separators under granular layers, slope stability, etc. cannot be designed quantitatively, as the size and nature of the stresses is still too little understood. A division into classes based on the ease of damage by aggregate is currently being undertaken. Examples of such classifications are to be found in Norway, Finland, and the German State of Hesse.

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Basic Principles Underlying the Swiss Guidelines for the Use of Geotextiles

Principes fondamentaux des recommandations suisses pour l'application des geotextiles

A Standards Commission Working Group of the Swiss Association of Highway Engineers (VSS) is in the process of preparing guidelines regarding a data sheet, testing the mechanical and hydraulic properties as well as the classification of geotextiles. This includes also minimum requirements of geotextiles for specific application in ground and highway engineering. On the basis of the results of standardized tests to determine the mechanical and hydraulic properties it is possible to classify geotextiles. This classification enables the design engineer to select the most suitable geotextile with the aid of a catalogue of standard applications. In this way the choice of product for normal applications is substantially simplified. Special uses, such as reinforcing functions for important earth structures, which requires further special investigation, do not fall within the scope of the proposed guidelines. Representatives of the technical university, government testing stations, users and manufacturers of geotextiles are involved in this work. The guidelines are based both on the experience of engineers from other countries and on the results of specific Swiss studies.

Un comité des normes de l'Union Suisse des Professionnels de la Route (VSS) est en train de préparer des recommandations pour la détermination du comportement mécanique et hydraulique et pour la classification des géotextiles. Ceci inclut aussi les exigences minimales pour les applications spécifiques en génie civil. En se basant sur les résultats d'essais standardisés pour la détermination des propriétés mécaniques et hydrauliques, il est possible de classifier les géotextiles. Cette classification permet à l'ingénieur de choisir le produit le plus approprié à l'aide d'une tabelle des applications standard. De cette façon la sélection du produit pour une application normale est grandement simplifiée. Les applications spéciales telles armement de terre pour projets importants, qui nécessitent des études plus approfondies, ne sont pas traitées par ces recommendations. Des représentants des écoles polytechniques, des laboratoires d'essais, des utilisateurs et des producteurs de géotextiles prennent part à ce travail. Les recommendations sont fondées à la fois sur l'expérience d'ingénieurs d'autres pays et sur les études spécifiques réalisées en Suisse.

1. INTRODUCTION

The Swiss Standards Association (SNV) pertains to the codes in Switzerland. Within the SNV the following organisations are responsible for building codes:
- Swiss Society of Engineers and Architects (SIA), which develops codes for civil engineering and building construction.
- Swiss Association of Highway Engineers (VSS), which develops codes for highway construction.

Codes for textiles are treated in the so-called interdisciplinary sector. Each SNV code can be a recommendation, a guideline or a regulation. It has to be specified accurately what are regulations and what are recommendations in the introduction of a code. The aim of the codes is to promote high quality construction; they are primarily intended for normal cases. They should only provide a regulation where a regulation is needed and where it offers a support in the application. The codes have to be guided by the practical application and should allow sufficient flexibility for the creativity of designers and contractors. They should be drawn up in such a way that they enable a requirement- and performance-oriented bidding procedure.

Codes are based on well-proven experience in the construction practice. Theoretical knowledge and good workmanship are prerequisite in every application. The Swiss codes are intended for use primarily by experts in design and execution. The codes as such have no force in law. In a legal proceeding concerning the case of construction damage they are considered as the state of the art. They are obligatory as soon as they are included in the contract of the owner. This is quite often the case in highway construction. Hence the legal effect particularly of the VSS-codes is quite important. A deviation from the code is always possible, but requires justification.

The VSS appointed a committee for the development of codes in the field of geotextiles. The committee consists of eight members. They represent planning and design, contractors, manufacturers, government as well as technical universities and government testing stations
As the VSS uses no notification procedure of codes before their coming into force, special considerations have been given to an equal representation in the code-committee of all interested parties.

The committee has developed the following provisional structuring of possible VSS textile codes:

- "Definitions and Product Description"
- "Testing Regulations"
- "Classifications of Geotextiles"
- "Minimum Requirements"

The first two code proposals are nearly completed. Work on the code "Classification of Geotextiles" is about to be started by the committee and for the code section on the "Minimum Requirements" application for research projects to investigate the necessary fundamentals have been submitted. In the following the individual codes as far as they have been developed till the beginning of 82 shall be presented. None of the proposals have yet been officially approved.

2. GEOTEXTILES "DEFINITIONS AND PRODUCT DESCRIPTION"

This code turns to the actual user of geotextiles. It defines the most important terms, which are necessary for the description of geotextiles and prescribes the range and content of the product description in data sheets. It is divided into 6 chapters:
- General (scope, application)
- Definitions
- Data sheet
- Structural data
- Mechanical properties and durability
- Hydraulic properties

In the following the individual chapters are commented upon:

2.1 Definitions

In this chapter the most important technical terms are defined. Special emphasis has been put on the use of a terminology which is readily understandable for a practical man. Intensive discussions have proved that the incorporation of these definitions is very important for the user, as the original definitions have to be collected in different codes and publications in textile technology and they are written in a language which is not easy to understand even for a civil engineer.

In addition to the basic terms like geotextile, continuous filament, staple filament, titter, monofilament, multifilament, etc. terms describing the structure of the geotextile are included. Together with a table showing the structure of geotextiles the user gets an overview of the large variety of geotextile products and is in a position to classify his product.

2.2 Data Sheet

A data sheet with prescribed minimum content represents the central part in the description of geotextiles. Table 1 shows the proposed data sheet. In this data sheet a comparatively large number of test results are included. It reflects the extraordinary broad field of application of geotextiles. The committee is of the opinion that at present it is not possible to characterise a geotextile well enough for the various applications on the basis of only one or two tests. All specifications in the data sheet rely on standard tests on the geotextile only, in spite of the fact that a geotextile is never used as a pure construction material but always in conjunction with soil. With the proposed tests the physical properties of the geotextile should be described in a manner adequate for practical purposes. The basic idea of this concept is as follows: If the properties of the soil around the geotextile are well known and if a model of the soil-geotextile interaction does exist it is always possible to investigate the behaviour of the soil-geotextile-system analytically at a later stage. Thus an analogous concept like that used for composite structural material e.g. reinforced concrete is applied. However, in the field of geotextiles analytical models are still at an elementary stage. Nonetheless it is the committee's belief that this is the only physically correct procedure from an engineering point of view. The user of geotextiles has a very broad range of application and must therefore be aware that the geotextile which is most suitable for a specific application need not have optimum values in all tested properties. In some applications, like drainage or filter for instance, the creep behaviour is only of minor concern.

2.3 Structural Data

The structural data comprise the description of the fiber material, its shape, if appropriate additives in the geotextile and the structure of the geotextile. These data will be furnished by the manufacturer. A control test by a testing station will only be carried out in special cases.

The raw material has to be described by its type and its percentage of the whole. In addition, the manufacturer has to declare the maximum permissible temperature for short and long duration exposure and the corresponding duration data. Unfortunately no test procedure for this temperature duration effect can be proposed. The shape of the textile fibers has to be specified by their length, shape of their cross-section as well as by their dimensions related to length.

As a third item the structure has to be described in detail. Both descriptions follow in their form the generally adopted form in the textile industry.

The declaration of the mass in grams per m^2 allows for a simple control of the delivered material by a comparison of the weight per unit area and the specified weight. It is pointed out in the code, however, that the mass per unit area should not be used as the only quality criterion, as is done quite often today. The scatter in the mass per unit area, determined on small samples of 100 cm^2 area, should give an indication of the homogeneity of the product.

2.4 Mechanical Properties

One of the most important sections in the data sheet is the declaration of the mechanical properties. The mechanical properties are largely

Table 1 Data Sheet

General data	- Name of product - Supplier, manufacturer - Minimum mass per unit area
Structural data	- Description of raw materials - Form of textile fibers - Structure of geotextiles
Mechanical properties (according to standard tests)	- Thickness and compressibility - Breaking strength, elongation at rupture and load-deformation relationship - Tearing strength - Penetration and push-through resistance - Creep
Durability (according to standard tests)	- Bacterial stability - Chemical (alkali/acid) stability - Ultra-violet light stability
Hydraulic properties (according to standard tests)	- In-plane permeability - Transverse permeability - Effective pore size
Special properties (optional)	- e.g. Soil-fabric friction

responsible for the range of application of geotextiles. They form together with the hydraulic properties the basis for the suitability characteristic of the individual geotextiles. This suitability characteristic shall be developed in the code sections 3 and 4 based on the test results. A considerable scatter in mechanical properties has to be anticipated for geotextiles due to their specific structure. Therefore, the properties have always to be evaluated as an average of a minimum of 10 samples and have to be specified together with the coefficient of variation.

Depending on the type and manufacturing process, geotextiles can have a variable compressibility normal to the plane of fabric. This influences the mechanical and hydraulic properties. In order to obtain an idea about the compressibility of the geotextile, the thickness has to be determined with three different applied pressures. The smallest pressure has been adopted such that the small projecting fibers are just pressed down. The pressure is 0.2 kPa. The two other pressures are 20 kPa and 200 kPa. With these data the thickness of the geotextile can be determined by interpolation for all practical purposes. The results for the average thickness with the corresponding applied pressure and the corresponding coefficient of variation are included in the data sheet.

Among the mechanical properties the breaking strength is of predominant importance, particularly if the geotextile has to assume some reinforcing function. Equally important is the breaking elongation. In addition to the peak values, the load-deformation-behaviour over the entire range plays a major role. For the testing at the breaking strength, the breaking elongation and the load-deformation behaviour, different testing methods are applied depending on the actual geotextile (woven or unwoven fabric). The basic test is the tensile test after textile code SN 198 461. This test is intended primarily to determine the reduction in breaking strength after different durability tests and secondly to obtain the load-deformation characteristic.

The results have to be presented as follows:
- For woven fabric the average breaking strength in kN per meter width and the average breaking elongation in percent obtained from 10 samples are given together with the coefficient of variation. In addition the envelope of the 10 load-deformation-diagrams has to be included.
- For all other geotextiles the results of the strip tensile test will not appear on the data sheet itself. They serve rather as index parameters for the evaluation of the durability tests.

The testing of the load-deformation behaviour on non-woven fabric presents some difficulties, as the fabric contracts extensively in a tension test. This can be avoided by one of two ways: Rods fitted with pins to resist contraction during the test can be used or tests with larger specimens are carried out. Both testing methods have advantages and disadvantages in the experimental realization. The test with rods to resist contraction presents difficulties with some of the materials but has the advantage that the conventional testing machines for textiles can be used. The test with larger specimens - up to 50 cm width - offer the advantage that no additional equipment has to be used. However, difficulties with the clamping system and with the measuring cell (off-center forces) may arise. Often the conventional testing machines for textiles cannot be used due to the larger tensile forces. Comparative tests in Switzerland have shown that samples with 50 cm width and samples with rods to prevent contraction produce practically identical results. After extensive discussion the committee decided to prescribe for non-woven fabric a tension test with rods to prevent contraction. The results are presented in the same manner as the strip tensile test in the textile code.

The tearing strength is a parameter for the ability of the geotextile to prevent spreading of local damage. In a specific tear-propagation tension test (trapezoid test) the resistance in both directions i.e. weft and warp is determined. Average values and coefficient of variation are included in the data sheet. The sample size is limited again by the dimensions of the ordinary testing machines for textiles. For some products, particularly for coarser material, testing of larger samples would be an advantage. But then the tests could not be carried out in the conventional test equipment. For this reason the committee has decided to maintain the standard sample size for textiles.

The penetration resistance is an index for the vulnerability of the geotextile, which is particularly important if the geotextile is going to have large angular cobbles dumped onto it. In this test a falling cone drops on an unsupported sample hold by a circular clamp. The mass of the standardized falling cone is 1 kg, the drop height 50 cm. As test result the average diameter of the created holes with the corresponding coefficient of variation is included in the data sheet.

In cases where geotextiles have to assume a reinforcing function the creep characteristics is a very important design parameter. In spite of the controversial opinions on the importance of the creep behaviour in practical applications, it has been decided to include a creep test. It is a comparatively simple test. The applied load corresponds to 25% of the average breaking strength. Tests have shown that loads of over 25% produce extensive creep deformation in most geotextiles. To avoid large creep deformations the stress has to be kept below 20 to 25% of the breaking strength. Strips of 5 cm width are tested. In tests on non-woven fabric rods to prevent contraction are used. For common applications this test characterizes the product well enough. But for more stringent requirements on the reinforcing function, additional specific tests have to be carried out. The results are given as average strain in percent after 1, 24 and 500 hours, separately for warp and weft.

The conventional testing procedures have the disadvantage that they require tensile test machines as used in the textile industry. This equipment is, however, hardly available on a building site. In soil mechanics laboratories mainly testing machines for the compression test are found. For the proposed push-through test a slightly modified CBR-testing device can be used. A smooth metal cylinder is forced at a speed of 1 mm per second through a sheet of geotextile held taut within a circular clamp of 15.2 cm diameter. As result the average maximum recorded force in Newtons is listed together with its coefficient of variation. For the purchaser of large quantities this test serves as a simple check of the strength of the delivered material.

In the soil the geotextiles are exposed to a large variety of environmental influences. As they have to maintain their function in most cases during a long span of time they have to be highly resistant to environmental influence. It is, however, impossible to test a product for all possible influences. Furthermore, it is uncertain how well these long-term effects can be investigated by a short time test procedure. Thus a considerable simplification in the test procedure seems to be appropriate. The codes provide tests for 5 environmental influences:

- physical influence (UV, modified SNV 195 809)
- biological influence (bacteria, 150 DIS 846.2; 1977)
- chemical influence (0.1 n lactic acid, pH 2.4; 0.1 n sodium-carbonate, pH 11.6; 10 g per liter $Ca(CH)_2$ pH 12.5)

The results of these tests give an indication whether the textile or part of it is going to be affected by bacteria, ultra-violet light or by chemicals which occur in the natural environment. The method of testing allows only a comparison between different products. Only limited conclusions can be drawn regarding the long term behaviour. The result is given as average breaking strength degradation in percent after bacteriologic, ultra-violet or chemical treatment. For all types of geotextiles the test is carried out as strip tensile test according to the code for textiles.

2.5 Hydraulic Properties

If the geotextile has to assume a filtering or draining function, the hydraulic properties are of predominant importance. In most applications the transverse permeability is of primary interest but if used as drains the in-plane permeability is important. Besides the permeability the filtering ability plays a major role.

The extent to which a geotextile can work as a filter or can build up a filter in conjunction with the surrounding soil material, determines the long-term efficiency of the geotextile. The hydraulic properties are determined with standard tests. Also, in these tests the geotextile is tested as such i.e. not together with the soil. These tests characterize the product well enough for common applications. If, however, specially high requirements are placed on the hydraulic properties and their maintenance then additional investigations have to be carried out.

The two permeabilities, normal to and in the plane of the geotextile are determined in a special permeameter using demineralized, deaired water. The permeability test is conducted with two different normal pressures, 20 kPa, 200 kPA, which also correspond to the pressures applied in determining the thickness. With these overburden pressures one is on the safe side regarding the permeability. The results are presented in units which are still somewhat unfamiliar. The permeability normal to the plane of the geotextile is designated as permittivity, which is a measure for the flow of water through the textile. For the permeability within the plane of the geotextile the term transmissivity is used, which is a measure for water transport capacity.

The effective pore size is determined for a geotextile in an unloaded state. The geotextile can be considered to be a sieve fed with different fractions of sand grains or glass beads. In the test the average grain diameter is determined at which 85, 90 and 95% of the fraction remains in or on the geotextile. Knowing the effective pore size the effectiveness as a filter can be estimated. The tests on unloaded geotextiles yield values which are on the safe side with regard to the filter criteria.

3. GEOTEXTILES "TESTING REGULATIONS"

The purpose of this code is to establish regulations for a standardized testing procedure of the mechanical and hydraulic properties of geotextiles. It supplements the code "Definitions and Product Description" and is mainly intended for testing stations.

In the code "Testing Regulations" the individual tests have to be described in such a way that they can be accurately reproduced. Minor details in the regulations are not yet fixed as they are not generally accepted. The standard method of testing for geotextiles is not uniform in different countries despite intensive coordination efforts and is still being developed. Often the methods of testing in the various countries are based on different concepts. For this reason many interested groups consider the publication of testing standards to be premature at the present time. On the other hand, the variety of tests available today (which, besides, are often poorly defined in the data sheets)do not permit in every case objective comparisons between the properties of different groups of geotextiles. Therefore, users as well as manufacturers are interested in achieving as quickly as possible a uniformity of testing methods.

After lengthy discussions the commissioned working group proposes to make the testing regulations an official publication of the VSS and not as an actual standard. This publication is mentioned in the code "Definitions and Product Description" and forms a part of the code. In the form of a separate publication this would allow an easier adaption to any new development in materials testing. Thus, as in the case of a provisional regulation experience can be gathered from practical application. The work of the committee for the texts of the two mentioned standards is more or less finished and the modification and acceptance procedure has been initiated.

4. "CLASSIFICATION" OF GEOTEXTILES

The basic studies carried out by the research group will be completed early in the Summer of 1982 and the results will be passed on to the committee. As a result, the committee will be able to prepare the proposal for the code section "Classification of Geotextiles" before the Spring of 1983.

The envisaged classification will be a short description of the results of the suitability tests. Conceptually it is based on the classification system of the French committee for geotextiles, whereby, however, a possible simplification will be aimed at. This classification still does not give any direct indication of possible applications for a specific geotextile. Since this regulation is closely related to the recommendations for minimum requirements it is intended to publish it as a code.

5. GEOTEXTILES "MINIMUM REQUIREMENTS"

For the user of geotextiles the main part of the code is the section on minimum requirements. It is planned to bring out recommendations for the various primary functions of geotextiles like separating two layers, drainage, filter, reinforcement, etc. Since, however, various functions can be coupled and requirements may vary with the importance of structures, for each function not only one but several recommendations will be necessary. The minimum requirements will be directed to applications in highways and related topics. In this way about 80 to 90% of practical applications in Switzerland will be covered.

The recommendations should be so drawn up that after the choice of the primary and secondary functions the determination of the recommended minimum properties may be carried out without difficulties. A non-specialist should be in the position to choose the appropriate geotextile at the site itself. Only minimum requirements have to be specified for so-called normal cases. In the case of structures for which geotextiles are of special importance, guidelines for a selection method should be given. Such uses, however, still mostly require special theoretical and material-technological investigations and it is not possible to standardize in these cases.

6. OUTLOOK

Should it be possible to begin the research work in early Summer 1982 then these minimum requirements could appear in the Spring of 1984. Until then the user has to depend on the recommendation of the manufacturers with the aid of foreign guidelines e.g. the catalogue produced by the French committee on geotextiles. The significance of the recommendations or minimum requirement within the framework of SNV-Standard demands a careful development of the basic data and an examination of their practical applicability. Therefore, a quick procedure is only possible to a certain extent although this would be desirable in practice.

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Experiences with "VTT-GEO" Classified Non-Woven Geotextiles for Finnish Road Constructions

Experiences sur les géotextiles classifiés "VTT-GEO" pour la construction de routes en Finlande

The Geotechnical Laboratory of the Technical Research Centre of Finland has developed a new, the "VTT-GEO" use classification for non-woven geotextiles to be used in road constructions. The principles of this classification system, the six necessary testing procedures for it and the valuation of the ten different criteria are dealt with in detail. Since in autumn 1980 the Finnish Road Board has adopted the new "VTT-GEO" classification for its district organisations' works, non-woven geotextiles are now generally marketed according to the new system. As a condition type-approval tests have to be performed once a year. With a simple field test procedure, based on two of the type-approval tests, the quality of fabrics delivered to the end user can easily be checked. The "VTT-GEO" classification has proved to work well and is of essential aid both for the end user and the producers of geotextiles.

Le laboratoire géotechnique du Centre national de recherches techniques de Finlande a mis au point une nouvelle classification "VTT-GEO" pour des géotextiles non-tissés pour la construction de routes. Les principes de ce système, les six procedés d'essai nécessaires et l'évaluation de dix critères differentes sont traités en détail. La Direction générale de routes en Finlande ayant adopté la nouvelle classification "VTT-GEO" en automne 1980 dans les travaux de ses organisations régionales, les non-tissés sont maintenant généralement vendus selon ce système. De nouveaux essais pour l'approbation du type doivent être effectués une fois par an. Par une simple méthode d'essai sur le terrain, à partir de deux essais pour l'approbation du type, la qualité des matériaux livrés à l'utilisateur peut être contrôlée facilement. La classification "VTT-GEO" a paru fonctionner bien et offre une aide considérable aussi bien à l'utilisateur qu'aux producteurs de géotextiles.

INTRODUCTION

During the past decade non-woven geotextiles, later in this article also the term fabrics will be used in that sense, have found their use in many applications in civil engineering works in Finland. In the pioneer stage of fabric applications both the users and the representatives of the producers concentrated their main efforts on finding technically and economically advantageous solutions. The nowadays in Finland reached sales level of about 3,5 million m^2 (1981) allow to draw the conclusion, that in many cases fabrics have replaced the traditionally of soil materials built up layers, but additionally to these many new applications must have been realized due to the use of fabrics. The most important fabric applications form still permanent and access roads, where the fabric is replacing the traditionally used sand filter layer and is overtaking a filtration and separation function.

Following to the pioneer stage the civil engineering market was flooded with fabrics. Mainly weight and strength specifications were promoted. It was still left to the customer to choose the most suitable product for his specific application. Two aspects, the costs and the announced strength properties, had the main influence on the purchase decisions. Scarcely other aspects, as permeability or filtration properties, were even considered. For different products the given strength values were seldom comparable, due to manysided possibilities for their determination. Some of these test methods obviously were also advantageous for certain types of fabrics.

On behalf of the Nordic Road Federation and under the responsibility of the Norwegian Road Research Laboratory a first classification of fabrics was published in 1977 (1). Based on a fall cone, a CBR-penetration test and on some yet unknown rules of the thumb different products were classified into 4 groups. The table with the so classified products was published in four languages, but until now the basic principles of the group requirements can only be guessed.

In Finland, Sweden and Norway this classification was immediately adopted, as it undoubtedly had the advantage of making real comparisons between different products possible. The earlier on weight limits based requirements were replaced by the fabric groups I...IV. For a short time both producers and their customers could concentrate all their efforts on price negotiations. Clouds appeared soon on the horizon, as new products had to be classified. Lack of information on the classification principles and on the rules of thumb adjusted to these made it impossible to assign the proper group to new products elsewhere but Norway.

GEOTEXTILE RESEARCH PROJECT AT THE GEOTECHNICAL LABORATORY OF THE TECHNICAL RESEARCH CENTRE OF FINLAND

In 1978 a research project with the working name Quality Control Project was started at the Geotechnical Laboratory of the Technical Research Centre of Finland. Nine producers and the dominating user groups were represented in the project supervision committee. In the first instant it was intended to clear up from a geotechnical and hydraulical point of view, which requirements have to be set to geotextiles at their different applications. Those questions were more thorougly dealt with in the articles (3) (4) and finally in (6) in respect of the long term stability of geotextiles.

In the next stage a minimized set of testing procedures, which in the best way describe the requirements set by practical applications, was invented and recommended for future use. For that purpose comparative tests were performed on a large scale, nowadays used testing methods were analysed and, where necessary, new methods were developed. It could clearly be shown, that depending only on the testing procedure products of different bonding type could suffer unequal treatment. A typical example of such a testing arrangement is the CBR-penetration test, if the test results were interpreted according to the calculation method used at the Norwegian Road Research Laboratory (2). In this case the interpretation of the test results is obviously of advantage for thermally bonded products.

With the new test methods, developed by the Geotechnical Laboratory of the Technical Research Centre of Finland, it was attempted to avoid any possible protectionism.

ON THE LONG TERM STABILITY OF NON-WOVEN GEOTEXTILES IN ROAD CONSTRUCTIONS

The long term properties of non-woven geotextiles, which had been installed into road constructions, were studied in the Nordic countries as a part of a research project of the working group 31 within the Nordic Road Federation. In Finland at 22 different road sites, covering the whole country, geotextiles were recovered by the Finnish Road Board's district organisations and the City of Vaasa. The quality of eight product types from 5 producers could be controlled several years after their installation. Even though the strength properties have undergone serious changes in several cases, it can be stated, that the fabrics have functioned satisfactorily as filter and separator during the controlled life time. These results are described more in detail in the articles (5) and (6).

PRINCIPAL CONSTRUCTION OF THE VTT-GEO FABRIC CLASSIFICATION

The recovering of the different non-woven fabrics allowed the essential conclusion, that, for fabric applications in permanent structures no remarkable design criteria concerning the stress-strain behaviour of non-woven fabrics can be drawn from the stress state prevailing in the earth structure. For that reason all the strength requirements have to be related to the installation procedure and the technics used at its different stages.

In addition to strength criteria other facts influencing the installation process, like roll width (a product may be delivered folded) and roll weight (rolls of 500 kg weight cannot and due to health regulations for workmen must not be handled by man power only).

Since the majority of the non-woven fabrics for civil engineering purposes available on the market are functioning above all as separator and filter within the earth construction, it is obvious, that also these functions have to be valued in a classification.

In the VTT-GEO classification the fabric properties were therefore valued according to the following key principles. The valuation of different strength properties was assessed to approximately 50 %. The pore structure and its stability, which characterize the filtration properties and the permeability, is judged with a weight of 35 to 40 %. 10- 15 % of the valuation is reserved for production-dependent values, like the uniformity of the product and for such terms of delivering like the roll dimensions and its weight, which finally influence the progress of installation.

The new VTT-GEO classification principles are presented in Table 1. Outside the classification remained still a group of fabric properties, which essentially contribute to the long term stability of the products. Those are UV-radiation resistance, chemical and biological resistance. As a basic requirement the fabrics have to be resistant to all in road constructions usually applied chemicals (as stabilizing agents like lime, fly ash, cement, oil,...), and further they have to resist the attack by bacteria and fungi present in the fabric environment. As these properties depend both on the raw material and on possibly used fixing agents, it seems to be proper to require from the producer side a guarantee concerning the long term stability of the fabric for the specific application in question.

THE CLASSIFICATION CRITERIA AND THEIR INTERPRETATION

In the VTT-GEO classification the product properties are judged in 10 criteria by aid of a point system. For 8 of these the points are calculated from test results evaluated from 6 classification tests. Two of the criteria are based on optimum modes of delivery, facts which normally are given by the producers.

The THICKNESS of the fabric is measured at a surcharge of σ=20 kPa on an area of F= 25 cm^2 at 3 points over the sample.

The GRAB TENSILE TEST is performed following in principle the procedure of ASTM D 1682 but modified with respect to the width of the fabric specimen and the deformation rate. The width of the sample is 200 mm and the rate of deformation 100 mm/min.

The PLUNGER PULL OUT TEST is performed following the testing procedure developed at the Geotechnical Laboratory of the Technical Research Centre of Finland. In the test a fabric specimen of a diameter ϕ 185 mm with a precut, centric hole of 10 mm diameter is clamped horizontally between the specially designed clamping rings fitting to the CBR-cylinder (see Fig. 1).

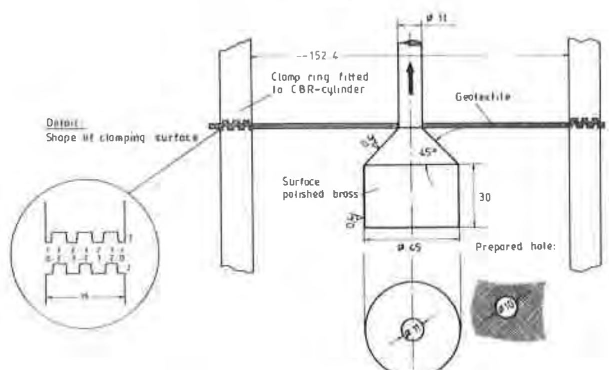


Fig.1 Plunger pull out test. Shape and dimensions of the plunger and the clamping rings.

The to the conical part of the pull-out plunger fixed rod of 11 mm diameter is pulled through the precut hole of the fabric sample and lifted until the fabric surface is touching the plane of intersection between rod and cone. In the test the plunger is pulled through the hole at a constant rate of deformation of 10.8 mm/min. During the test the vertical force acting on the plunger and the corresponding deformation are registered. As test results are listed the maximum force (P_{max}) and the corresponding pull length (h_{max}), the force measured at a pull length of 20 mm (P_{20}) and the vertical component of the

CRIT. NR.	CLASSIFICATION CRITERIA	UNIT	NUMBER OF TESTS	SAMPLE SIZE mm	WEIGHING FACTOR	REMARKS
	UNIFORMITY TESTS:					
1	WEIGHT: $\bar{W}_o$	g/m^2	10	$400\ cm^2$	$\frac{\bar{W}_o - 100}{20}$	WEIGHT INCREASE-BENEFIT
2	WEIGHT VARIATION: s	%			$((5/s)-0{,}5)\times 5$	LESS THAN 5 % DESIRED
	STRENGTH VARIABILITY: GRAB TENSILE TEST IN L AND C DIRECTION					
	TENSILE STRENGTH: $\bar{P}_{max} - s$	N	10+10	200/200	$\frac{(\bar{P}_{max}-s)_{strong}}{(\bar{P}_{max}-s)_{weak}} = A$	A > 1 HOMOGENEITY DESIRED, FACTOR A USED WITH CONE PULL OUT FORCE
	THICKNESS: $\bar{t}_o$ (at 20 kPa)	mm	10	$25\ cm^2$		$\bar{t}_o$ USED WITH POROSITY CRITERIA
	STRENGTH TESTS: CONE PULL OUT TEST					
3	STRENGTH: $\bar{P}_{max}-s$	N	10	200/200	$\frac{(\bar{P}_{max}-s)}{20 \times A}$	AVERAGE STRENGTH REPRESENTED
4	$\overline{P_{max} - P_{20}}$	N	10		$\sqrt{(\overline{P_{max}-P_{20}})-s} - 5$	AVAILABLE STRESS AFTER INITIAL STRAIN (h = 20 mm)
5	ELONGATION: $\bar{h}_{max}-s$	mm	10		$\left(\frac{\bar{h}_{max}-s}{45}\right)^2 \times 20$	BENEFIT FOR h_{max} >45 mm
6	FRICTION: $\bar{P}_{clamp}$	N	10		$\frac{(\bar{P}_{clamp}-s)}{2}$	TEAR RESISTANCE REFLECTED
	PUNCHING AND PERMEABILITY: CONE PENETRATION TEST, WATER SUPPORTED GEOTEXTILE.					
7	HOLE SIZE: $\bar{\varnothing} + s$	mm	10	200/200	$(\frac{50}{\bar{\varnothing} + s} - 1)\times 5$	SHOCK TEAR TEST + PERMEABILITY: MAX. 45 POINTS
	POROSITY TEST: AIR FLOW RATE AT 0.1 kPa OVERPRESSURE					
8	AIR FLOW: $\bar{Q}_{air}$	$m^3/m^2,s$	10	$10\ cm^2$	$(2{,}5\times\sqrt{\bar{Q}_{air}} - \frac{\bar{Q}_{air}}{\sqrt{n \cdot \bar{t}_o}})\times 20$	LOW AIR FLOW WITH THICK PRODUCTS COMPENSATED MAX. 50 POINTS
	WORKABILITY:					
9	ROLL WIDTH: (max) b	m			$(b\times 2) - 10$	
10	ROLL WEIGHT: W_r (NORMAL SIZE, IF NOT SPECIFIED BY THE CUSTOMER).	kg			$10\times\sin(\frac{W_r - 15}{210} \times \pi)$	AVAILABILITY OF ROLL SIZE CONVENIENT TO HANDLE BY MAN-POWER

GEOTEXTILE CLASSIFICATION: GROUP REQUIREMENTS

CLASSIFICATION GROUP (FOR USE IN ROAD CONSTRUCTION)		POINTS NECESSARY
1	UNCLASSIFIED, NO SPECIAL REQUIREMENTS	$\leq$ 99,9
2	SEPARATOR TO NATURAL SOILS	100,0...140,0
3	SEPARATOR TO MACADAM, SORTED BLASTED ROCK	140,1...220,0
4	SEPARATOR TO UNSORTED BLASTED ROCK	$\geq$ 220,1

Table 1. The principles of the VTT-GEO use classification for non-woven geotextiles to be used as separator and filter in earth constructions.

friction force acting on the cylindrical part of the pull out cone (P_{clamp}) (see Fig. 2).

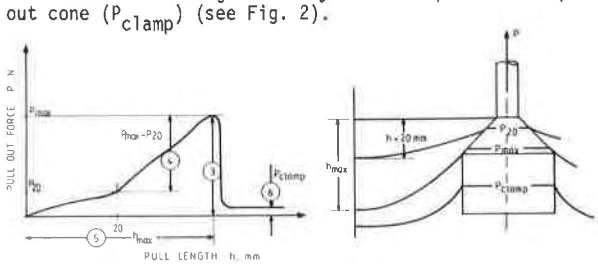


Fig. 2 Plunger pull out test. Interpretation of test results according to the criteria nos. 3..6.

The CONE DROP TEST with water support is in principle performed according to the test procedure described by Alfheim and Sorlie, Paris 1977 (2), but modified with respect to a water support of the fabric specimen clamped into the CBR-cylinder. The CBR-cylinder of a height of 142.5 mm and a volume of 2.6 litres below the fabric is filled up with water until the water level is touching the underside of the fabric sample. In this test the falling cone has to displace an amount of water through the fabric tested, which is equal to that partial volume of the cone which penetrated the sample.

The WEIGHT PER UNIT AREA of the fabric type is determined using samples of size 200 x 200 mm.

The AIR FLOW RATE TEST is performed according to DIN 53 887 using a circular test area of 10 cm^2 and a pressure gradient of 100 Pa.

In the following the criteria are briefly interpreted. Also the valuation of each criterion is explained in the sequence of Table 1. The correlations between criteria elements and the evaluation of points are presented in the Figures 4a...d. The relative weights of the main groups of criteria are given in Table 2 for different fabric classes. The calculations were based on the results of tests performed since 1978.

The weight per unit area can hardly be seen as a product property, but its presence as a classification criterion is justified for two reasons. A product with a specific weight greater than that of water can easily be installed on flooded sites compared to floating ones. Obviously this criterion could be of slight advantage to needle punched non-wovens, which normally need more raw material to meet strength requirements equally to termically bonded types, if in the classification only strength properties would be judged.

The WEIGHT VARIATION reflects the homogeneity of the product. As a matter of the production process the materials in question simply cannot be produced without any mass variation, and trimming to minimum variation is probably uneconomic. For the user the inhomogeneity of a product becomes increasingly disadvantageous with increasing mass variation. For these reasons the criterion 2 is given a construction, which is of benefit for homogeneous products, especially for those, which have a weight variation less than 5 %.

The 25 mm GRAB TENSILE TEST is used only to judge the uniformity of the product in respect of its maximum tensile strength. Products may have essential variations in strength properties depending on the direction of testing. For the majority of civil engineering applications obviously only the strength values in the weakest direction will be of advantage for the user. The fabric homogeneity in respect of tensile strength is judged by the homogeneity parameter A ($A \geq 1$), a ratio of the average tensile strength values in two directions, from which the standard deviation has to be deducted. The homogeneity parameter is used with the proper strength criterion no. 3.

The PLUNGER PULL OUT maximum force reflects the force necessary to extend an 11 mm wide hole to a diameter of $\phi = 45$ mm. The criterion judges the reserve of the geotextile structure to conform with the unevenness of the underground, without successive tear resp. extension of a hole, which forsome reason might have been created during the installation procedure.

The STRENGTH REMAINDER of the plunger pull out test is derived from the pull-out force readings at an initial plunger travel of 20 mm and at maximum. It is a measure for the available strength after an initial strain, which might have been consumed by the installation process.

The ELONGATION criterion is derived from the plunger travel at maximum pull-out force. Fabrics with average "elongation" values exceeding the maximum plunger diameter of 45 mm will gain a slight benefit of the criterion's construction. This built-in advantage is justified by the most positive experiences gained with extensible fabrics in numerous applications. The friction criterion is derived from the vertical component of the clamp force developed during the final stage of the plunger pull out test, where the plunger has practically extended the hole in the fabric sample to a size of $\phi = 45$ mm and the pulling of the plunger is continued. By aid of this criterion it is possible to judge, to which extent the fabric structure will tolerate the penetration of sharp stones, stumps etc. The clamp force is both depending on the amount and strength of intact fibres around the hole and on the bond strength between the fibres, so the criterion can be looked at as belonging to the group of strength criteria. Nevertheless, the friction criterion is mainly judging the stability of the fabric structure, which will ensure satisfactory performance of tne separator and filtration function. For that reason this criterion is grouped together with the following criteria nos. 7 and 8.

The PUNCHING AND PERMEABILITY criterion No. 7 is derived from the hole size in the cone drop test. This test is a modification with a reduced energy input of the one invented by Alfheim and Sorlie (1) (2). At the Norwegian Road Research Laboratory some fabrics had to be tested using a reduced fall height of the cone of 250 mm. Otherwise the test results would have consisted of mainly 50 mm hole sizes. The too high energy input could have been reduced also by changing the weight of the fall cone, the cone angle or diameter of the clamping rings.

Supporting the fabric by water has, besides the energy input reduction, an additional advantage. The falling cone will, immediately after its tip has reached the surface of the fabric, displace a certain amount of water through the pores of the fabric, depending on the penetration depth of the cone. As the cylinder is closed, the water can be discharged only through the pores of the fabric. Thus the test results will be depending on the water permeability of the fabric. The better the discharge capacity of the fabric, the more effectively it can restrain the energy input of the fall cone. A fabric of poor permeability will be lifted upwards by the shock wave of the water, so less restraining the falling cone. This effect will also influence the size of the hole opened by the falling cone. The construction of the criterion is advantageous to small hole sizes, which corresponds to the original Norwegian interpretation of the test results. The criterion judges mainly the separating function of the fabric, but to some extent also its filter function.

The POROSITY of the fabric is judged in the criterion no. 8. It was the ambition not to relate the pore structures of fabric and the soils in contact with the fabric to each other. Fabric filters have generally to be more permeable than the soil material to be filtered, but in principle a 20 times higher water permeability coefficient will be sufficient. The water permeability coefficients of non woven fabrics fulfill this requirement in respect of all those soil materials, to which the application of fabrics might be of benefit. We must not expect miracles from the filter properties of fabrics, and it seems needless to try to dimension the fabric filter according to filter criteria. In cases, where filters built up from soil material, have to be composed of 2 or more layers, it cannot be expected, that a single-layered fabric with more or less uniform pore structure will fulfill similar filter requirements at various stress and strain conditions. In praxis it could be observed, that, but only with static flow conditions, a natural filter cake will be built up in the contact zone between soil and fabric. Together with the fabric structure this filter cake probably will function like a multilayered filter. Dynamic loading conditions will prevent the building up of such a stable natural filter cake behind the fabric. A cake of fines might then appear on top of the fabric, which is a clear sign for misfunctioning of the fabric as filter.

The correlation of test results from water and air permeability tests is generally considered to be poor, but new light was thrown on the matter from comparative tests on compressed fabrics. Ground-installed the fabrics will always be in compressed condition. The water permeability tests, if performed carefully and with de-aired water, are both time consuming and extremely laborious, for which reason the porosity criterion was developed on the basis of the air permeability test. The criterion is composed of two elements, which valuate both the air discharge quantity and the basic parameters of the pore structure of the compressed fabric. By this construction the criterion will ensure, that permeability and porosity of a fabric are kept within reasonable limits.

FABRIC PROPERTIES LEFT OUTSIDE THE CLASSIFICATION SYSTEM AND PRACTICE FOR THEIR DETERMINATION

A good deal of work can be done, if all the significant fabric properties were tried to determine, and as the branch is developing fast, new research topics will arise all the time. Some of the production parameters may easily be changed, like the composition of fibres, the fibre raw material or the type of bonding. The same production lines, which expel civil engineering fabrics, produce also similar looking products for customers in many other branches. Intensive research work on fabric properties could gain long term advantage, supposed the fabric properties could be linked to the most essential production parameters. Such attempts, even of an independent research institution, were generally refused referring to production secrets.

For that reason the user has to insist on fresh testing certificates given by independent research bodies, on all these properties, which might be important for his application, but remained outside the classification or a type approval system. All facts concerning long term behaviour fall within this group of properties. UV-stability is one of these, though fabrics used for road constructions will seldom be exposed to sunlight over longer periods. Further the producer has to guarantee stability against chemicals generally used in civil engineering works, e.g. stabilizing agents, like lime, cement, oil, fly ash, slag etc.. Fabrics have also to resist biological attacks by bacteria and fungi present in the top soil layers. To clear up these questions suitable test methods have recently been developed. So the user can insist on getting sufficient information on the long term behaviour of the nowadays marked fabrics.

CHECKING THE FABRIC QUALITY IN THE FIELD

The quality of VTT-GEO classified fabrics can be controlled in the field by two simple testing procedures. These are the plunger pull out test and the fall cone test with water supported fabric samples. Three parallel tests of each type have to be performed. The average values of the two test series form the field acceptance criteria.

I. FALL CONE TEST. The three tests are performed according to the specifications described above. As test result is determined the average hole size opened by the cone.

II. PLUNGER PULL OUT TEST. The plunger is pulled or alternatively pushed through the fabric sample clamped into the CBR-rings. The pull force or pushing force may be applied statically by adding weights, with a constant rate of loading device (e.g. hydraulic jack), but also with a CRS testing machine. The loading principles are shown in Fig. 3.

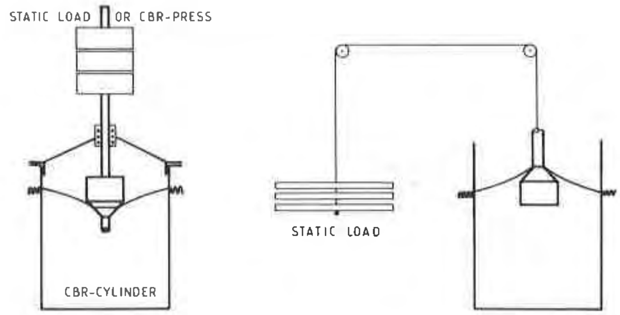


Fig. 3 Plunger pull out test. Alternative loading arrangements for field use.

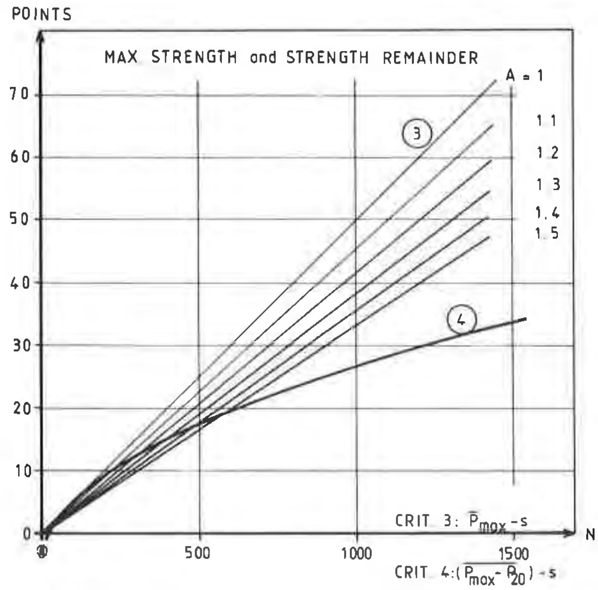
In all cases the required maximum load has to be supplied within 5 minutes. The force requirements for test acceptance have to be determined from the average hole size evaluated from the fall cone tests. In the one case the average maximum force will be reported as the test result,in the alternative case the test gives acceptable results, if the fabric could resist the plunger penetration at the required force level in all the 3 parallel tests. The acceptance criteria for the field tests are given in Tab. 3.

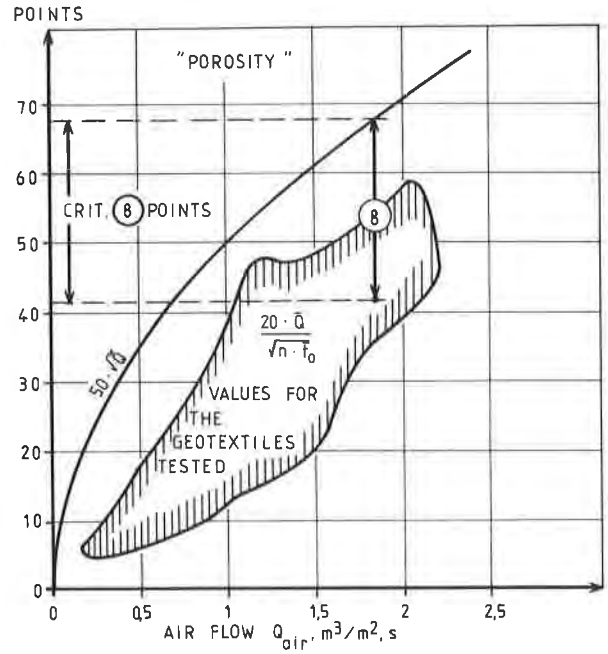
Table 3. Acceptance criteria for the field test series

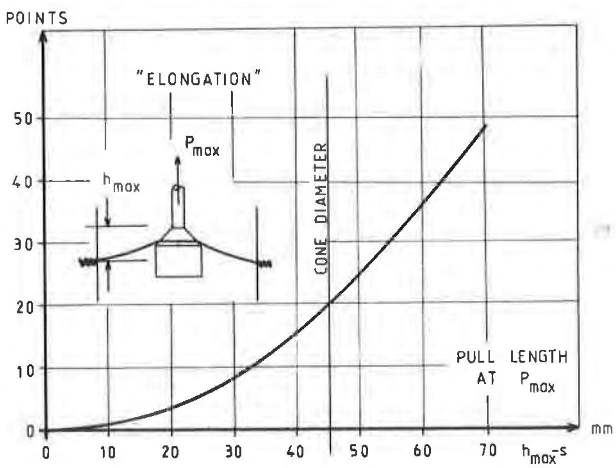
USE CLASS	ALTERNATIVE PAIRS OF ACCEPTANCE CRITERIA			
	I. FALL CONE TEST AVERAGE HOLE SIZE mm		II. PLUNGER PULL OUT TEST AVERAGE PULL OUT RESISTANCE $\bar{P}_{max}$ kN	
II	a)	≤ 20	and	≥ 35
	b)	≤ 25	and	≥ 45
	c)	≤ 30	and	≥ 55
	d)	≤ 35	and	≥ 65
III	a)	≤ 20	and	≥ 70
	b)	≤ 25	and	≥ 80
	c)	≤ 30	and	≥ 90
IV	a)	≤ 20	and	≥130

APPLICATION OF THE VTT-GEO CLASSIFICATION IN FINLAND

By the end of the year 1979 the VTT-GEO geotextile classification has been adopted for general use in road constructions. The National Board of Public Roads and Waterways, several purchase departments of cities, companies and organizations require nowadays new testing certificates when inviting tenders for geotextiles on an annual basis. The testing work has generally been performed by the Technical Research Centre of Finland and the following conditions were linked to the application of the VTT-GEO classification. The fabric samples have to be taken from stock of a representative quantity and by an independent person. The testing certificate is valid over a period of one calendar year. If the field tests bring up that the type does not fulfill the class requirements, the complete set of tests has to be carried out for a new type approval.







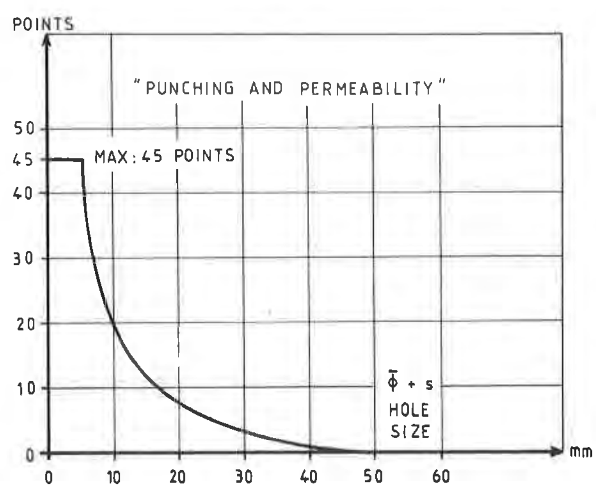


Fig. 4a...d. Correlation between criteria elements and the accumulation of points.

The VTT-GEO classification system has proved to work well during the time it has now been used. Only few reclamations concerning the quality of products delivered to customers were made in cases, where the construction had simply failed. In all cases the geotextiles fulfilled the class requirements, when their quality was controlled with field tests. It could be shown that design faults or unsuitable working methods had caused the failures.

Careful steps were taken to apply the here described classification system also to water constructions. The basic principles seem to work well, though heavier classes and higher point limits are obviously needed. Combined with additional testing procedures, like the BAW (Bundesanstalt für Wasserbau, Karlsruhe, Germany) turbulence test for filtration stability of certain soil types and clearly defined requirements on long term stability, the VTT-GEO classification will serve the demands set to geotextiles for water constructions.

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Specifications and Recommendations of French Geotextiles Committee

Spécifications et recommandations du Comité Français des Géotextiles

Having determined and standardized a number of tests allowing to characterize the performance of geotextiles, the french committee has carried instalments of recommendations out. They allow a rational choice of geotextiles used into different constructions. This paper explains and justifies the method used to build these recommendations and shows an example. The instalments relative to the use of geotextiles into the roads for construction traffic, the low trafficked roads, the subgrades, the stocking areas and the parking places have already been edited, as well as an instalment about their setting and control. Other instalments will be likely to publish in 1982. Thanks to the collaboration of the interested parties inside of the committee (administrations, university, manufacturers, design firms, contractors), the use of these instalments is quite accepted in France, which allows homogeneous and profitable prescriptions.

Après avoir défini et normalisé un certain nombre d'essais permettant de caractériser le comportement d'un géotextile, le Comité Français a établi des fascicules de recommandations rendant possible un choix rationnel des géotextiles utilisés dans différents ouvrages. Cette communication expose en la justifiant la méthode adoptée pour l'élaboration de ces recommandations et présente un exemple d'application. Les fascicules concernant l'utilisation des géotextiles dans les pistes, les voies à faible trafic, les couches de forme, les aires de stockage et de stationnement ont déjà été publiés ainsi qu'un fascicule concernant leur mise en oeuvre et leur contrôle. D'autres fascicules devraient être publiés en 1982. Grâce à la collaboration, dans le cadre du Comité, des différents intéressés (administrations, universités, producteurs, bureaux d'études, entrepreneurs), l'utilisation de ces fascicules fait l'objet en France d'un large consensus ce qui permettra une homogénéïté des prescriptions profitable à tous.

1 - INTRODUCTION

1.1 NECESSITE D'UNE NORMALISATION

Dans le monde entier, et spécialement dans les pays industrialisés comme la France, le marché des géotextiles en génie civil est en forte et régulière progression.

Toutefois, en France, jusqu'en 1980, l'absence de spécifications a laissé souvent les prescripteurs désarmés devant des produits encore nouveaux pour beaucoup d'utilisateurs possibles.

La diversité, ou au contraire l'absence, d'essais appropriés pour mesurer une même caractéristique augmentait leur embarras en rendant difficile ou impossible la comparaison de deux produits différents.

Ces graves lacunes pouvaient fausser la concurrence et conduire parfois à des contre-performances lorsque le critère de prix était le seul retenu.

Le développement des géotextiles a donc nécessité une meilleure définition des produits exigés pour chaque application.

En France, chaque organisme public ou semi-public est responsable de ses spécifications. Il fallait donc éviter la multiplication de spécifications différentes ou contradictoires pour un même emploi, ainsi que celle d'essais exigés par des organismes différents pour apprécier une même propriété.

Cette multiplicité de spécifications et d'essais placerait les "prescripteurs isolés", par exemple les bureaux d'études et les producteurs de textiles devant une forêt inextricable.

Dès 1978, une association s'est formée en France, sous le nom de "Comité Français des Géotextiles" avec pour but, parmi d'autres, de proposer des bases communes de spécifications et d'essais.

Ce Comité réunit :

- les principaux services publics concernés,
- des établissements publics et privés d'enseignement et de recherche,
- des bureaux d'études,
- des producteurs et des distributeurs de géotextiles,
- des entreprises publiques et privées de génie civil.

La présente communication expose les premiers résultats de ce travail commun d'harmonisation qui, sans se substituer aux responsabilités de chaque organisme, fournit aux prescripteurs des guides pour la mise au point d'un projet ou la rédaction d'un cahier des charges.

1.2 METHODE SUIVIE

Le Comité s'est d'abord attaché à définir les principales caractéristiques nécessaires et suffisantes des géotextiles pour caractériser ces matériaux.

Ce travail débouche sur une échelle de classification des caractéristiques retenues. Il est ensuite établi un fascicule de recommandations d'emploi pour chaque type d'utilisation où les caractéristiques du géotextile à employer sont déterminées en fonction de son rôle et des sollicitations qu'il supporte. Cette détermination est faite le plus souvent à l'aide de tableaux se référant aux échelles de classifications.

Les valeurs recommandées correspondent à des cas moyens et peuvent être modulées en fonction des données propres du projet. Le prescripteur ou l'ingénieur d'étude est ainsi guidé par les recommandations sans en être absolument prisonnier.

2 - CLASSIFICATION DES GEOTEXTILES

2.1 PARAMETRES PRIS EN COMPTE

2.1.1. Généralités

Les fonctions que remplit un géotextile dans un ouvrage et les sollicitations qu'il y subit sont multiples et varient d'une utilisation à l'autre. Il n'est donc pas possible de classer valablement les produits par référence à une seule propriété ou à un seul essai.

Il faut toutefois pour respecter l'objectif, qui est l'étude des cas courants :
- limiter le nombre de propriétés prises en compte, pour éviter d'avoir des tableaux trop lourds à manipuler,
- pour la même raison, prévoir pour chaque propriété un nombre de divisions de l'échelle pas trop faible, mais non pas aussi trop élevé,
- utiliser des grandeurs familières aux projeteurs de génie civil, afin d'être immédiatement compréhensibles.

Le choix du Comité s'est finalement arrêté sur les propriétés suivantes :

2.1.2. Propriétés mécaniques (mesurées dans le sens production et le sens travers)

- résistance à la traction (en KN/m)
- allongement à l'effort maximal (en %)
- résistance à la déchirure (en KN)

Par souci de simplicité, il n'a pas été fait appel, pour caractériser le comportement mécanique,aux notions de module et d'énergie de rupture. Il est toutefois sous entendu que les produits doivent présenter une énergie de rupture suffisante (aire située sous la courbe effort-déformation dans un diagramme effort-déformation) et proche ou supérieure de celle correspondant à un diagramme effort-déformation sensiblement linéaire. Les géotextiles actuels répondent bien à cette exigence.

2.1.3. Propriétés hydrauliques

- écoulement normal au textile : permittivité $\frac{Kn}{e}$ (en s^{-1}),
- écoulement dans le plan du textile : transmissivité Kt.e (en m2/s) mesurée dans le sens de l'écoulement

  Kn : coefficient de perméabilité de Darcy mesuré dans le sens perpendiculaire au géotextile

  Kt : coefficient de perméabilité de Darcy mesuré suivant le plan du géotextile

  e : épaisseur du géotextile.

2.1.4. Propriétés filtrantes

La porométrie est représentée par le seul diamètre O_{95} qui est par définition le diamètre tel que 95 % des pores aient un diamètre inférieur, et 5 % un diamètre supérieur. Il correspond donc sensiblement au plus gros diamètre des éléments du sol pouvant traverser le textile, appelé diamètre de filtration.

2.2 MODES D'ESSAI A RESPECTER POUR L'USAGE DU TABLEAU

2.2.1. Devant la grande diversité d'essais et leurs résultats divergents, il est nécessaire d'examiner les différentes méthodes pour déterminer celles qui répondent aux objectifs suivants :
- permettre la comparaison entre les produits différents,
- être relativement simples,
- être physiquement interprétables et donner des chiffres suffisamment précis pour pouvoir être éventuellement utilisés dans des calculs de dimensionnement.

Les essais textiles traditionnels (traction sur bande de 5 cm,déchirure ASTM ou EDANA), permettent de contrôler la régularité d'une production, mais ne répondent pas aux critères ci-dessus.

En particulier, les éprouvettes doivent avoir des dimensions suffisantes, par rapport à la structure des produits, permettant effectivement aux liaisons entre fibres de jouer.

Pour les essais mécaniques, une dimension minimale de 100 mm a été retenue.

2.2.2. Résistance à la traction et allongement à l'effort maximal

2.2.2.1. règles générales

- les conditions de l'essai doivent être telles que l'éprouvette se déforme peu dans la direction perpendiculaire à la traction,
- dimension minimale de l'éprouvette : 100 mm
- vitesse de déformation : entre 10 et 100 % par minute.

2.2.2.2. essai retenu

L'essai retenu pour l'établissement et l'emploi du tableau est une traction sur éprouvette de 100 mm entre pinces et 500 mm parallèlement aux pinces.

L'allongement à la rupture utilisé dans le tableau est une valeur calculée conventionnelle R

$$\varepsilon_R = \varepsilon_1 + \varepsilon_2 + \varepsilon_1 \cdot \varepsilon_2$$

ε_1 déformation moyenne à la rupture

ε_2 déformation à la rupture, dans la direction perpendiculaire, mesurée au milieu de l'éprouvette

(déformation positive pour un allongement et négative pour un raccourcissement).

2.2.2.3. autres essais

D'autres essais répondant aux règles générales peuvent être utilisés. Par contre, les essais sur bande de 200 x 50 mm, l'essai d'arrachement (grab test), l'éclatomètre circulaire, ne doivent pas être utilisés comme données d'entrée dans le tableau.

2.2.3. Résistance à la déchirure

Elle est mesurée sur éprouvette trapézoïdale de grande dimension (bases 670 et 225 mm, hauteur 445 mm).

déchirure amorcée de 50 mm au milieu de la petite base.

vitesse de déplacement des pinces : 50 mm/mn.

La valeur retenue comme force nécessaire pour propager la déchirure est la moyenne des cinq valeurs maximales notées pendant l'essai.

2.2.4. Caractéristiques hydrauliques

Les mesures de perméabilité doivent être faites avec de l'eau désaérée et sous un gradient assez faible pour se trouver dans les conditions de validité de la loi de Darcy.

2.3.5. Porométrie

Elle est déterminée en faisant passer à travers le textile un sol à granulométrie continue et appropriée en suspension dans l'eau. Le processus de passage doit être tel qu'il n'y ait pas accumulation de particules sans mouvement à la surface du géotextile pendant l'essai.

On admet que la porométrie (O_{95}) est égale au D_{95} de la courbe granulométrique du matériau ayant traversé le géotextile.

2.3.6. Bien d'autres essais peuvent être pris en compte lors d'une étude pour un ouvrage particulier : essais de traction en croix, résistance au poinçonnement, fluage, sensibilité à la température, résistance dynamique, frottement sol-textile, ou textile-textile.

Nous ne les avons pas pris en compte, soit que leur utilisation ne se fasse que dans le cadre d'une étude spécifique par un bureau d'étude spécialisé, soit que les essais existants ne soient pas encore suffisamment adaptés.

2.3 ECHELLES DE CLASSIFICATION

Les principales propriétés des géotextiles, mesurées par les essais appropriés, sont repérées par rapport à des échelles de classification. Une échelle de douze classes est établie pour chaque propriété. Ces échelles sont données dans le tableau ci-dessous (fig.1)

		CLASSES											
		1	2	3	4	5	6	7	8	9	10	11	12
RESISTANCE A LA TRACTION kN/m	1 2	4	8	12	16	20	25	30	40	50	75	100	
ALLONGEMENT A L'EFFORT MAXIMAL ε_R %	3 4	8	11	15	20	25	30	40	50	60	80	100	
RESISTANCE A LA DECHIRURE kN	5 6	0.1	0.2	0.3	0.5	0.8	1.2	1.7	2.3	3	4	6	
PERMITTIVITE K_n/e S^{-1}	7	10^{-2}	2.10^{-2}	5.10^{-2}	0.1	0.2	0.5	1	2	5	10	50	
TRANSMISSIVITE $K_t.e$ m^2/s	8	10^{-8}	2.10^{-8}	5.10^{-8}	10^{-7}	2.10^{-7}	5.10^{-7}	10^{-6}	2.10^{-6}	5.10^{-6}	10^{-5}	5.10^{-5}	
POROMETRIE O_{95} µm	9	600	400	200	150	125	100	80	60	40	20	10	

Nota : Les lignes 1,3 et 5 correspondent au sens production.
Les lignes 2, 4 et 6 correspondent au sens travers.

Fig. 1 - Echelles de classification

3 - PRESENTATION DES FASCICULES DE RECOMMANDATIONS

3.1 GENERALITES

Un fascicule est établi pour chaque type d'utilisation : pistes, routes à faible trafic, voies ferrées, aires de stockage et de stationnement, travaux hydrauliques, ouvrages de drainage, etc... En outre, un fascicule particulier traite des règles générales de mise en oeuvre, de l'agrément et du contrôle des géotextiles.

Pour chaque utilisation, la démarche consiste à :
- définir le rôle du produit dans l'ouvrage,
- analyser les sollicitations qu'il supporte et ses intéractions avec le sol. Cette analyse débouche sur la prise en compte de paramètres propres au type d'ouvrage,
- déterminer les caractéristiques du géotextile à employer en fonction des paramètres définis précédemment.

3.2 PARAMETRES A PRENDRE EN COMPTE

Ces caractéristiques sont en particulier fonction du rôle que le géotextile doit assurer dans l'ouvrage suivant l'analyse résumée ci-après.

3.2.1. Séparation - anticontamination

Le géotextile sépare deux couches de matériaux et l'ensemble n'est soumis à la percolation d'eau que de façon occasionnelle et peu importante.

Un minimum de perméabilité est cependant nécessaire, en général, pour éviter la stagnation de l'eau ou le développement de sous-pressions.

La continuité du géotextile à l'interface est essentielle. Cette continuité nécessite, non seulement une certaine résistance à la rupture, mais surtout une déformabilité compatible avec celle du sol.

De plus la résistance à la déchirure doit être suffisante pour éviter, lorsque le textile est sous tension, la propagation d'une déchirure à partir d'une coupure localisée (provoquée par un caillou par exemple).

Cette continuité suppose également que le matériau résiste aux sollicitations auxquelles il est soumis lors de la mise en oeuvre : efforts de traction, déchirures statiques ou dynamiques, poinçonnement etc....

3.2.2. Filtration

Dans ce cas, le textile joue le rôle de séparateur dans une zone soumise à une importante percolation d'eau, perpendiculairement à la nappe. Son rôle est de laisser passer l'eau tout en maintenant les particules de sol.

Ce rôle se rencontre dans les géotextiles autour des tranchées et tapis drainants, sur les berges de rivière et parements de barrage, entre le sol et le parement en enrochement, dalles de béton etc....

Comme précédemment, la fonction continuité est évidemment primordiale dans ce rôle, mais en plus le produit doit être impérativement plus perméable que les sols environnants et avoir une porométrie adaptée pour retenir les particules de sol entrainées par l'action de l'eau.

3.2.3. Drainage

La continuité doit être assurée, au moins dans le sens de l'écoulement.

La perméabilité et l'épaisseur du géotextile (mesurées sous une contrainte égale à celle existant dans l'ouvrage) doivent être suffisantes pour évacuer le débit voulu sous une charge suffisamment faible.

Le drain ne doit pas se colmater. S'il est monocouche, sa porométrie doit être appropriée. Il est possible aussi de séparer les fonctions drainage et filtration en utilisant une partie drainante grossière, protégée par des parties filtrantes plus fines.

3.2.4. Renforcement mécanique

Les paramètres principaux à prendre en compte sont la résistance et la déformation du géotextile soumis à la traction, le coefficient de frottement sol-textile, et éventuellement les caractéristiques de fluages.

Toutefois, ces deux derniers paramètres ne sont pas pris en compte dans les tableaux de dimensionnement, parce que trop spéciaux.

3.3 SOLLICITATIONS

Les sollicitations du géotextile sont fonction de son rôle et des caractéristiques propres de l'ouvrage. Ces dernières sont analysées en tant que paramètres particuliers au type d'emploi :
- désignation précise de l'ouvrage (par exemple : tranchée drainante, piste, etc...),
- structure de l'ouvrage et position du géotextile (par exemple : structure de piste comportant un ou deux textiles, géotextiles sous une couche drainante ou non drainante).
- caractéristiques des sols de part et d'autre du

géotextile (sol support, matériaux d'apport...),
- sollicitations externes sur l'ouvrage : trafic, charges etc....

3.4 DIMENSIONNEMENT

On obtient, en croisant tous ces paramètres, un très grand nombre de cas. On ne retient que les cas vraisemblables où l'emploi d'un géotextile est justifié. Ces cas sont portés dans un répertoire ou un tableau qui renseigne immédiatement l'utilisateur et donne la référence du dimensionnement.

Celui-ci est présenté le plus souvent sous forme de tableaux utilisant les échelles de classification indiquant les valeurs minimales ou maximales recommandées pour chaque cas.

Ces valeurs correspondent à des cas moyens et peuvent être modulées, pour chaque cas particulier, en tenant compte de l'analyse du rôle du géotextile et de ses sollicitations.

Pour certaines applications où l'attribution de classes a priori n'est pas possible, des méthodes de détermination sont proposées (écoulement hydraulique, filtration, renforcement).

L'exemple qui suit illustre cette démarche.

4 - EXEMPLE D'UN FASCICULE : LE FASCICULE "RECOMMANDATIONS POUR L'EMPLOI DES GEOTEXTILES DANS LES VOIES DE CIRCULATION PROVISOIRE, LES VOIES A FAIBLE TRAFIC ET LES COUCHES DE FORME"

4.1 ROLES DU GEOTEXTILE

Le rôle essentiel gu géotextile est un rôle de séparation entre le sol support et le matériau d'apport, permettant ainsi à ce dernier de conserver ses caractéristiques.

Le rôle de renfort mécanique se limite généralement à l'effacement, sur les sols de portance très variable à l'échelle de quelques dizaines de centimètres, des points de portance les plus faibles où se serait amorcé l'orniérage. Certains géotextiles particulièrement résistants ou placés en plusieurs couches peuvent néanmoins participer au fonctionnement mécanique de l'ouvrage. Il faut alors veillez au mode d'assemblage des bandes ainsi qu'à un ancrage latéral du géotextile.

Lorsque le sol support est mou et que le matériau d'apport est peu perméable (grave polluée) il peut être intéressant que le géotextile joue un rôle de drain, permettant ainsi une certaine consolidation du sol support, dans sa partie superficielle.

Enfin, dans le cas où des écoulements chargés peuvent se produire entre le sol et le matériau d'apport, le géotextile doit jouer un rôle de filtre.

4.2 PARAMETRES A PRENDRE EN COMPTE

4.2.1. L'importance de ces différents rôles et les caractéristiques nécessaires au géotextile pour qu'il puisse les remplir sont liées essentiellement à la qualité du sol support, à la nature et à l'épaisseur du matériau d'apport ainsi qu'au trafic que doit supporter la voie.

4.2.2. Chacun de ces paramètres est précisé et divisé en plusieurs classes :

a) le trafic est analysé en fonction du type de véhicule, du nombre de véhicules ou du tonnage total devant passer sur la voie. On distingue ainsi six classes de trafic (4 s'appliquent pour les pistes, 3 pour les voies à faible trafic et 2 pour les couches de forme),
b) les sols supports sont répartis en trois classes en fonction de la portance la plus faible qu'ils peuvent avoir pendant la période d'utilisation de la voie
   - les sols SU1 d'indice CBR > 5
   - les sols SU2 d'indice CBR compris entre 2 et 5
   - les sols SU3 d'indice CBR < 2

   Ces classes sont également définies par rapport à d'autres essais courants de géotechnique.
c) les matériaux d'apport sont classés en trois types :
   - les matériaux G1 : matériaux concassés
   - les matériaux G2 : graves propres
   - les matériaux G3 : graves polluées.
d) les épaisseurs de matériau d'apport sont réparties en
   - couches minces (20 à 25 cm)
   - couches moyennes (30 à 45 cm)
   - couches épaisses (50 à 80 cm)

   Pour les couches moyennes et épaisses on distingue en plus les couches renforcées, comportant non seulement un géotextile à leur base mais également un autre dans leur milieu. On obtient ainsi un total de cinq structures différentes.
e) dans le cas de pistes, le niveau de service, défini en fonction de la profondeur d'ornière tolérable (P.O.T) n'est pas déterminé a priori mais peut être choisi. On distingue ainsi un niveau de service moyen (15 cm)et un niveau excellent (5 cm).

4.3 CAS TRAITES

La combinaison de ces différents paramètres pour les trois types de voies considérées conduit à un total de 585 cas. Cependant un grand nombre de cas ne sont pas réalistes. Par exemple sur un sol support de très faible portance (CBR < 2) on n'envisage pas de faire passer un trafic lourd sur une couche mince. D'autre part, certains cas possibles bien que peu vraisemblables (couche épaisse sur sol support SU1 avec trafic léger par exemple) ne justifient pas l'emploi d'un géotextile et ont été éliminés.

Ces considérations ont permis de restreindre le nombre de cas traités à 93, correspondant à 88 tableaux. Un répertoire permet à l'utilisateur de trouver rapidement le cas qui le concerne et le tableau correspondant.

Rappelons enfin que le document est destiné à dimensionner le géotextile et non l'épaisseur de la voie de circulation pour laquelle certains paramètres supplémentaires doivent être pris en compte (entretien de la voie, risque admis).

4.4 EXEMPLE D'UTILISATION

Prenons le cas d'une couche de forme. Nous rappelons que la terminologie française désigne par ce terme la couche de transition (mise en oeuvre lors des terrassements) entre le sol support et la chaussée proprement dite lorsque les propriétés mécaniques du sol ne sont pas suffisantes(1).

Supposons que le projeteur prévoit qu'il n'y aura pas de circulation d'engin particulièrement lourd hors code (on a donc un trafic PLb), que le matériau d'apport disponible est une grave propre G2et que la couche de forme traverse une zone d'indice CBR 3 (SU2) et une zone d'indice CBR 6 (SU1).

La liste des cas envisagés est reproduite à la figure 2.

Afin d'économiser le matériau d'apport, le projeteur élimine la possibilité de couche épaisse.

Pour le sol support SU2, le répertoire montre que deux cas sont prévus : couche moyenne (grille 78 cf. fig. 3) et couche moyenne renforcée (grille 81 cf. fig. 4). Pour le sol SU1, un seul cas est prévu (couche moyenne - grille 71 cf. fig. 5) la qualité du sol support ne justifiant pas une couche renforcée.

(1) Pour plus de précision concernant les couches de forme, leur dimensionnement et leur mise en oeuvre on pourra consulter la Recommandation pour les Terrassements Routiers éditée par le Service d'Etudes Techniques des Routes et Autoroutes et le Laboratoire Central des Ponts et Chaussées - Ministère des Transports.

OUVRAGE	TRAFIC	PROFONDEUR ORNIERE TOLERABLE	SOL SUPPORT	COUCHE D'APPORT	MATERIAU G	CARACTERISTIQUES DU GÉOTEXTILE GRILLE	PAGE
COUCHE DE FORME	PLb	2 A 3 CM	1	MOYENNE	1	70	36
					2	71	36
					3	72	36
				ÉPAISSE	2	73	36
					3	74	36
				MOYENNE RENFORCÉE	2	75	36
					3	76	36
			2	MOYENNE	1	77	37
					2	78	37
				ÉPAISSE	2	79	37
					3	80	37
				MOYENNE RENFORCÉE	2	81	38
					3	82	38

Fig. 2 Liste des cas envisagés

Valeurs déconseillées (pour un cas moyen)

1 : Structure avec un seul géotextile.

2 : Structure avec deux géotextiles.
A. Premier géotextile en contact avec le sol support.
B. Deuxième géotextile au sein du matériau d'apport.

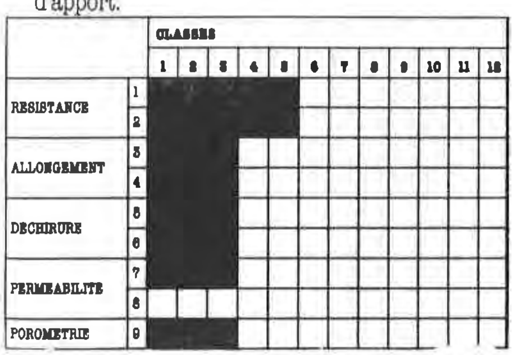

Fig. 3 - Grille n° 78 (Sol SU2 - couche moyenne)

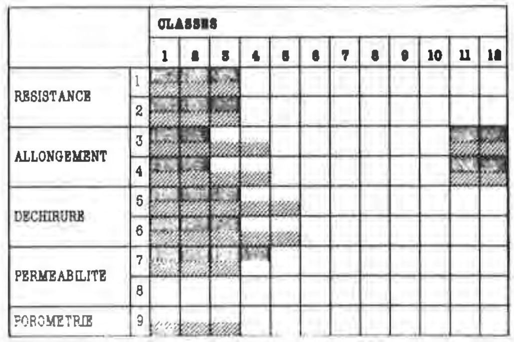

Fig. 4 - Grille n° 81 (Sol SU2-couche moyenne renforcée)

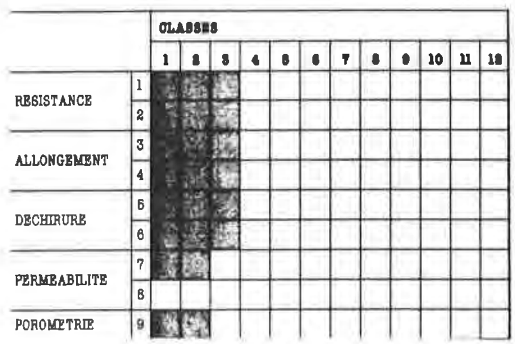

Fig. 5 - Grille n° 71 (Sol SU1 - couche moyenne)

Les valeurs fournies par les grilles correspondent au dimensionnement indiqué au tableau de la Fig. 6.

Ce tableau appelle les commentaires suivants :

- résistance à la traction : le géotextile n° 2 devra être plus résistant car le sol est peu portant et la couche de forme non renforcée par un second géotextile,
- allongement à l'effort maximal : pour les géotextiles n° 3 et n° 4, l'allongement est limité supérieurement pour empêcher l'utilisation de géotextiles de trop faible module, les géotextiles participant au fonctionnement de l'ouvrage. Par ailleurs, le géotextile n° 3 doit avoir une capacité d'allongement suffisante pour tenir compte du fait qu'il sera très sollicité lors de la mise en oeuvre (1ère couche d'apport réduite).
- résistance à la déchirure: Pour cette même raison le géotextile n°3 doit être très résistant à la déchirure,
- permittivité : elle doit être supérieure pour les géotextiles sur sol mou (SU2) pour faciliter le processus de consolidation de la frange supérieure du sol. Pour le géotextile n°4, elle doit être très importante pour ne pas diminuer la perméabilité verticale de la grave.
- transmissivité : aucune exigence n'est formulée ; la grave propre étant perméable et le géotextile ayant une permittivité suffisante, le géotextile n'a pas à jouer le rôle de drain,
- porométrie : les risques de pollution de la grave étant plus importants sur les sols de faible portance,

on demande une porométrie plus fine aux géotextiles n°2 et n°3 qu'au géotextile n°1. En ce qui concerne le n°4, sa position au sein de la couche d'apport n'exige aucune caractéristique porométrique.

	Sol SU_1	Sol SU_2		
		Couche non renforcée	Couche renforcée géotextile	
			à la base	au milieu
	N°1	N°2	N°3	N°4
Traction kN/m	≥ 12	≥ 20	≥ 12	≥ 12
Allongement %	≥ 15	≥ 15	$20 \leq \quad \leq 80$	$11 \leq \quad \leq 80$
Déchirure kN	$\geq 0,3$	$\geq 0,3$	$\geq 0,8$	$\geq 0,3$
Permittivité s^{-1}	$\geq 2.10^{-2}$	$\geq 5.10^{-2}$	$\geq 5.10^{-2}$	$\geq 0,1$
Transmissivité m^2/s				
Porométrie µm	≤ 400	≤ 200	≤ 200	

Fig. n°6 - Dimensionnement des Géotextiles

5 - AUTRES FASCICULES

Nous présentons ici succinctement les autres fascicules déjà publiés ou qui le seront très bientôt.

5.1 SYSTEMES DE DRAINAGE ET DE FILTRATION

Ce fascicule ne traite que du drainage en génie civil. Pour la filtration des règles simples mais donnant une certaine sécurité ont été dégagées. Des formules permettent de dimensionner les drainages horizontaux et verticaux, avec ou sans collecteurs.

5.2 AIRES DE STOCKAGE ET DE STATIONNEMENT

Quatre cas d'utilisation (stockage de courte durée, stationnement etc...) et six structures géotextile - matériau d'apport sont envisagés conduisant à 25 grilles de dimensionnement.

5.3 GEOTEXTILES SOUS REMBLAIS

Le dimensionnement, sous forme de grilles, est ici fonction des propriétés du sol support et de celles de la 1ère couche du remblai.

5.4 ESPACES VERTS, TERRAINS DE SPORT ET DE LOISIRS

Ce fascicule traite des jardins, aires de jeux, terrains de sport, tennis, plantations, etc.... Sept structures sont envisagées conduisant à 17 grilles de dimensionnement.

6 - FASCICULE MISE EN OEUVRE ET CONTROLE

Ce fascicule concerne l'agrément et le contrôle du géotextile, son stockage, la préparation du support et les règles générales de pose.

On trouvera en annexe les modes opératoires des essais définis par le Comité Français.

Nous ne développerons ici que l'agrément et le contrôle du géotextile.

L'agrément consiste à s'assurer que le géotextile proposé est conforme aux spécifications. Il s'effectue à partir des fiches indiquant l'identification des produits et leurs caractéristiques techniques déterminées selon les méthodes du Comité Français.

Le contrôle comporte deux phases :
- lors de la livraison du produit il convient de vérifier qu'il s'agit bien du produit agréé, grâce à l'observation du géotextile et de son étiquetage et si possible en effectuant un essai d'identification (masse surfacique notamment),
- lorsque le risque encouru en cas de déficience du géotextile est jugé suffisamment important on peut, suffisamment en avance sur les travaux, vérifier directement que les caractéristiques techniques imposées sont atteintes.

7 - CONCLUSIONS

Le Comité Français des Geotextiles a déjà publié trois fascicules de recommandations traitant respectivement de :
- la mise en oeuvre et du contrôle des géotextiles,
- l'emploi des géotextiles dans les aires de stockage et de stationnement,
- l'emploi des géotextiles dans les voies de circulation provisoire, les voies à faible trafic et les couches de forme.

D'autres fascicules sont en fin de rédaction et doivent être publiés dans le courant del'année 1982. Ils ont pour objet :
- l'utilisation des géotextiles dans les espaces verts, les terrains de sport et de loisirs,
- l'utilisation des géotextiles sous les remblais,
- l'utilisation des géotextiles dans les ouvrages de drainage et de filtration.

Ces recommandations sont conçues pour être utilisées aussi bien par les chefs de chantier que par les ingénieurs de bureaux d'études.

Bien qu'ils n'aient pas de caractère obligatoire, ces fascicules sont distribués comme documents techniques et recommandés pour l'établissement des clauses contractuelles par plusieurs administrations dans leurs services (Ministère de l'Agriculture, Ministère de l'Environnement, Ministère des Transports).Leur influence sera donc rapidement sensible.

La collaboration et la coordination au sein du Comité en matière de recherche ont permis de déboucher rapidement sur la mise au point d'essais nouveaux, permettant une comparaison valable des produits et donnant des valeurs plus fiables pour les projeteurs.

A partir de ces données, permettant la confrontation des expériences et des études, il a été possible de bâtir des recommandations d'emploi, qui sont l'objet d'un large consensus au sein de l'ensemble des administrations et services publics français et qui assureront une homogénéïté des prescriptions profitable aussi bien pour le Génie Civil que pour l'Industrie Textile.

L'action de concertation entreprise et continuée au sein du Comité Français des Géotextiles, entre Administrations, Bureaux d'Etudes privés, Entreprises de Génie Civil et Producteurs de Textiles, apparaît donc très fructueuse.

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Quality Standards for Vertical Drains

Demandes de qualité pour drains verticaux

An ever increasing number of various types of prefabricated drains are being marketed to replace the traditional sand drain.
On the basis of the primary function of a vertical drain quality standards for drains should state:
a. those qualities of drains which should be determined;
b. the way in which drain properties should be established.
Quality standards and testing methods should be related to the various situations in building practice and to installation procedures. Quality standards should state that a drain can only be accepted for a job if a test report is available from a recognized laboratory in combination with a classification on the basis of requirements caused by subsoil, design and construction.

L'offre des divers types des drains préfabriqué; pour remplacer les drains sables, augmente de plus en plus.
En émanent des functions primaires du drain vertical, il faudra être fixé:
a. quelles qualités d'un drain doivent être déterminés;
b. comment les propriétés du drain doivent être établites.
Les demandes et méthodes d'inspecter les drains, doivent être en relation aux situations pratiques, qui changent constamment et doivent être en relation aux procédures d'installation des drains, lesquelles on doive suivre.
En vertu des demandes dequlité, on peut seulement admettre des drains préfabriqués à l'usage d'un projet de drainage vertical, s'ils sont guidés par un rapport d'inspection d'un laboratoire reconnu.
L'inspection catégorique doive être arriver en vertu des demandes par rapport à la souterraine, la construction et l'exécution du projet.

1. INTRODUCTION

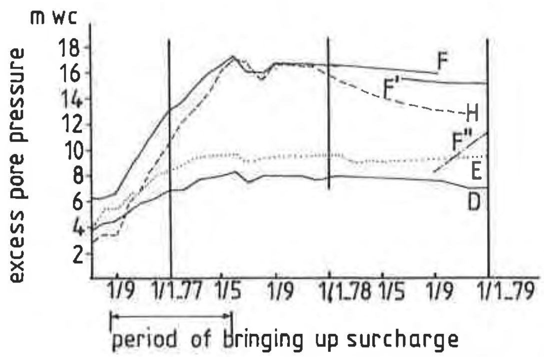
1.1. To shorten the consolidation process initiated by earthworks, vertical drains are used in many cases. In the past mostly sand drains or "sand piles" have been employed. In the seventies a large number of band-shaped prefabricated drains have been marketed, in continuation of the development of the cardboard drain by Kjellmann. Claims are made that they function as well as or even better than sand drains. Quality control of the latter has to be concentrated primarily on the way of installation and the precaution on the job site.
For prefab drains, however, it is necessary as well as effective, to put quality requirements on the drain itself. The necessity for standards is becoming more important since there is a tendency to decrease the available time of consolidation in which through the application of a dense pattern of prefab drains good and interesting possibilities are offered.
In the formulation of quality standards for drains, consideration has to be given to the application in building practice, and the function drains have to perform in the subsoil.
The objective of vertical drainage is: "to bring about a more rapid progress in the settlement process and a more rapid increase in shear resistance in highly impermeable subsoil."
The primary function of the drains is: "to absorb the expelled groundwater from the surrounding soil, with a relatively low entry resistance and to discharge it vertically."

1.2. Until 1974 sand drains were used almost exclusively for this purpose in the Netherlands. Many years of experience in application, supported by theoretical considerations, have resulted in a number of empirical rules with respect to the installation of such drains and the design of the drainage scheme for a project.
The introduction of the prefabricated drains has stimulated the rethinking of the theoretical and practical aspects of vertical drainage considerably.
The users are confronted with a strip, small in size compared to sand drains, which is composed of one or more elements. These elements, which are manufactured out of materials that are foreign to soil machanics engineers, are used as a filter medium and a discharge medium. It is understandable that doubts concerning the working and the durability of these types of drains may arise.
The most currently applied prefab band-shaped drains can be divided into the following types:
- one piece non-woven fabric drains;
- composite drains, having a profiled core with surrounding filter sleeve.

The most current sizes are 100 mm x 3 to 4 mm.

1.3. The usual practice in the Netherlands up to now is to accept a specific brand and type of drain on a job only, after the functioning of that drain has been demonstrated by the producer or supplier by means of prolonged measurements from a field test site. Although this condition is understandable, it does not guarantee the proper functioning of the drain under all conditions. For prefab drains, as with sand drains, not only careful installation is important, but the material quality has to be consistent on any job site.
The circumstances in which the drains are to function can be of great influence, such as the nature of subsoil,

the existence of a horizontally layered soil structure, the type of earthwork, the size of the embankment resulting in high or low earth pressures, small or large vertical and horizontal deformations, etcetera.
It has become evident in several cases that in practice large differences with respect to design expectations can occur. An example of such a case has been reported (1). Fig. 1, taken from that report shows the high level of the excess pore pressures continuing for more than one year, although the same type prefab drain was very succesfully applied to a power plant construction project 400 m from the site.



gauge D: N.A.L. - 15,4 m F: N.A.L. - 21,6 m
E: N.A.L. - 16,8 m H: N.A.L. - 11,1 m

Fig. 1 Excess pore pressures under a surcharge with drains.

Measurements on several types of drains tested under comparable conditions, such as those which have been carried out south-east of Amsterdam, and which are reported in this conference (2) can be of greater value. However, a disadvantage of all in situ measurements is that the outcome is not explicitly related to the performance of the different components of the drain with respect to the surrounding soil. This is one of the major reasons why it is desirable to perform laboratory tests and to relate these results as fully as possible to in situ measurements. The conduction of laboratory tests is a much more suitable approach because the tests can be performed uniformly, they are reproducible and they are much less time and money consuming. As a result the development of new drains and the optimization of the existing types may be stimulated.

1.4. The lack of official quality standards together with the fact that some Dutch principals have good experience with one or two drain brands resulted in prescribing a specific drain brand in the specifications of a job.
However, in such cases also an equivalent product has to be accepted.
Without quality standards the determination of equivalence can be difficult and is usually relatively subjective in practice. This can be a potential cause of conflicts between principals and contractors, the potential being greater if the subsoil conditions and the nature of the earthworks do not allow the determination of the proper performance of the drainage system in a simple and objective way.
From the foregoing it may be concluded, that even though the development of prefabricated drains is an important step in accelerating consolidation, quality standards are necessary to prevent disappointments and conflicts.

2. THE NEED FOR DRAINSPECIFICATIONS

In spite of the fact that many aspects of vertical drainage are not well understood, it must be possible to set up quality standards that drains have to be subjected to, in accordance with our present knowledge, insight and experience.
The objectives for quality standards are as follows.

a. To ensure the effectiveness of vertical drainage in view of the nature of the earthwork and embankment stability during the construction period.
b. To prevent unsuitable drains -with respect to soil conditions and construction- from being applied.

By means of such standards it is hoped that the following will be achieved.

1. An objective laboratory testing procedure.
2. A documentation of in situ measurements demonstrating the effectiveness and reliability of drains classified in relation to subsoil, method of installation and type of earthwork.
3. A stimulation of appropriate further developments based on laboratory and in situ experience.
4. A classification system for drainage works.

The operation of vertical drains as influenced by the surrounding soil, the installation procedures and the interaction between earthwork and subsoil is very complicated. It is therefore almost impossible to set up a unique series of quality standards so that tested drains will function properly and reliably in all the various types of works. In many cases this would lead to an unnecessary rise in costs and would, moreover, reduce the possibilities of new developments to a minimum. It goes without saying that such a series of quality standards would take many years of research at high costs.
Therefore quality standards should be subdivided in relation to subsoil characteristics and types of earthworks. This will lead to standards related to different types of the latter.
The effectiveness and reliability of drains can be determined by measurements on the job site. By introducing a system of in situ measurements in connection with the specified levels of quality standards, properly documented practical experience with drains will become available.

3. ELEMENTS OF QUALITY CRITERIA FOR DRAINS

3.1. Apart from influences which are difficult to quantify, such as the interaction between the drain and the soil, the installation procedure of the drain and the care taken during the installation, the effectiveness of the drain depends on the properties of the materials forming the drain and the construction of the drain. Determination of the material properties lends itself to laboratory investigation. Influences that are dominated more or less by the care and manner of application in the field can hardly be determined in another way than by in situ measurements.

3.2. Firstly, the type of requirements that the drains will have to meet has to be determined and secondly the corresponding laboratory tests that will have to be performed, must be defined.
These requirements and testing procedures should be related to the variation in field circumstances.
Prefabricated drains will have to meet requirements for the following properties.

a. Mechanical properties
   - tensile strength in connection with the installation procedures;
   - elasticity characteristics in view of horizontal deformations in the subsoil;
   - buckling strength in connection with folding during the compression of the subsoil.

b. Permeability and stoppage of clay and silt
   - minimum permeability in relation to soil permeability;
   - maximum permeability to prevent fine particles from clogging the drain.

c. Discharge capacity
   - minimum discharge capacity, at various total pressures;
   - acceptable reduction in discharge capacity, in case of folding or buckling.

d. Durability and dependability
   - sensitivity to chemical, biological and physical deterioration;
   - sensitivity to the clogging of the vertical discharge passages and blocking of the filter.

As a result of the discussions in the committee on "Vertical Drainage" of the Study Centre for Road Construction, the following recommendations are made.

3.3. The determination of drain properties should be performed with prescribed test methods, according to an outline of quality standards for vertical drains with minimum requirements as described in "Drain Standards", see par. 4.

3.4. Minimum acceptance requirements.
Prefabricated drains for vertical drainage will be accepted for a job only, if a test report in which the properties listed in "Drain Standards" is available at least one month before the tender is brought out.
The test report should be drawn up by an authorized laboratory which has tested the drains with the prescribed test methods.

3.5. Low embankments (< 2,5 m).
All types of drains may be accepted, with which the satisfactory operation can be demonstrated by the tests in "Drain Standards" or by in situ measurements in embankments.

3.6. High embankments (> 2,5 m).
Only those types of drain may be accepted, with which proper functioning can be demonstrated by in situ measurements in embankments under construction, with comparable subsoil conditions and total pressures and with a consolidation period comparable to the required period.

3.7. Earthworks, stability cases.
Earthworks, in which the probability of instability is to be reduced by the application of vertical drains, also need extra requirements for the drains.
Only those types of drain may be accepted, with which proper functioning can be demonstrated by in situ measurements in earthworks under construction having comparable subsoil conditions and which have functioned suitably during the required period of operation.

4. PROPOSAL FOR MINIMUM REQUIREMENTS "DRAIN STANDARDS"

4.1. Tensile Strength

4.1.1. In view of the possibility of uncontrolled penetration of the mandrel during installation, the tensile strength of the drain must be more than 500 N per 100 mm of width, at a maximum strain of 10 %, and at a controlled rate of strain of 200 mm/min.

4.1.2. After the drain has been kept under water for 24 hours, the ultimate tensile strength of the filter sleeve must exceed 120 N per 100 mm at un ultimate strain between 2 % and 10 %; the controlled rate of strain is 2,5 mm/min.

4.2. Filter Quality

4.2.1. The percolation resistance c of the filter sleeve must be less than $5d \times 10^8$ s after a percolation test of at least 40 hours, (d = thickness of the filter sleeve in m).

4.2.2. The filter effectiveness, that is, the ability of the filter sleeve to stop fine particles under pressure, must be such that no particles > 10 µm can pass.

4.3. Discharge Capacity

4.3.1. The minimum vertical discharge capacity must be 300 cm^3/min, with a potential difference of 0,25 m across the drain sample having a length of 0,40 m; the cell pressure is equal to 10 N/cm^2.

4.3.2. The minimum vertical discharge capacity, of a folded drain sample (a flattened s-configuration) having a resulting length of 0,40 m, must be 30 % of the discharge capacity determined in accordance with 4.3.1.

4.4. Durability and Dependability

The requirements and testing procedures are under development.

4.5. Vertical Sand Drains

4.5.1. Sand for drainage purposes must be mineral material. The percentage of mineral particles passing through the 63 µm sieve, must be less than 5 % of the fraction passing the 2 mm sieve. The fraction on the 250 µm sieve, must be greater than 50 %. The loss on ignition of the fraction < 2 mm must be less than 3 %.

4.5.2. The minimum coefficient of permeability is 12 m/24 h ($1,4 \times 10^{-4}$ m/s).

4.6. Testing Procedures

All laboratory tests and in situ measurements must be executed according to prescribed test methods.

5. CONSIDERATIONS FOR THE DETERMINATION OF QUALITY STANDARDS

5.1. Introduction

The aforementioned tests provide a starting point for the determination of standards together with the existing experience obtained from investigations on the initial drain types, while, however, the development in this field has continued.
The given values are based on a relatively small number of laboratory tests performed on the first prefabricated drains available in the Netherlands in the early 1970's and with which also field tests were carried out.
The field tests demonstrated that the drains work, but it is not known to which degree the determined characteristics influence their working.
The relation between laboratory tests and field tests requires more study and research, part of which is being carried out at the present time by the Dutch Study Centre for Road Construction (S.C.W. commission F9).
At any rate, significant differences in the working of the filter medium as well as in the discharge capacity have been demonstrated in the laboratory investigation (Fig. 2).
In general the laboratory tests on drains (3) must be carried out without the surrounding soil in order to obtain reproducible test results.
Only then can the similarity or the difference in performance between drain types be determined.
The filter efficiency, of course, cannot be determined without the presence of soil so that a test procedure has been devised in which a reproducible soil mixture is brought to surround the drain (4).

The intention is to make the quality standards officially known when all the laboratory tests have been formulated; hopefully this will occur at the end of 1982 or the beginning of 1983.

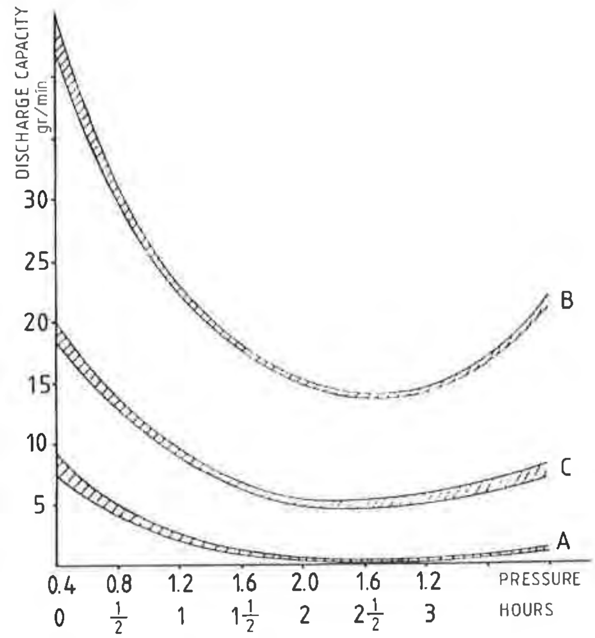


Fig. 2 Discharge capacity at different horizontal pressures.

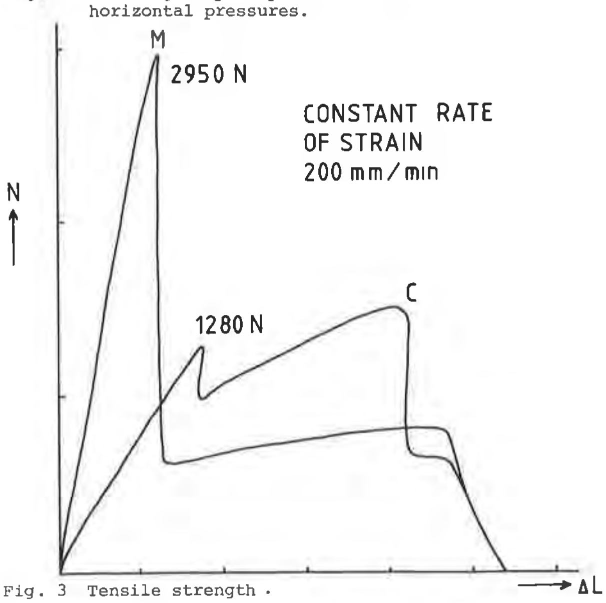


Fig. 3 Tensile strength.

5.2. Description of "Drain Standards"

5.2.1. Tensile Stresses and Strains

The 500 N criterion is based on calculations carried out to estimate the order of magnitude of the tensile forces that can occur during the placing of a drain. A quick tension test in the laboratory is a type of simulation for the field condition. Fig. 3 illustrates the results of the first tension tests performed on prefabricated drain samples.

It is evident that an important consideration concerns the minimum strain value required to prevent failure in tension of any particular component of the drain without total failure. It is clear that the chances of non-functioning drains due to damage during the installation procedure should be minimized.

Consequently, a maximum strain of 10 % has been set as a criterion in order to limit deformations such as reduction in width or in thickness due to installation.

The criteria stated in 4.1.2. is assigned to prevent fundamental alterations in the characteristics of the installated drain such as for example the closing off of the discharge passages due to high horizontal soil pressures.

The determination of a limiting strain value for the filter sleeve presents a problem because the conditions for preventing the choking of the discharge passages in the core conflict with the condition of a high permissible strain value in case of horizontal subsoil deformations.

The above criterion of 120 N strength at 2 % strain has been important in selecting suitable paper filters since in some cases even though their appearance was similar, the behaviour was such that several did not meet the norm.

5.2.2. Filter quality

Conflicting criteria also present a problem for filter specifications. The greater the permeability of the filter, the greater the risk of clogging the discharge passages with silt and clay particles will be.

The upper limit of the filter resistance given in 4.2.1. is initially determined from the resistance of the filter paper of drains (3) that apparently have functioned satisfactorily in the field (Fig. 4).

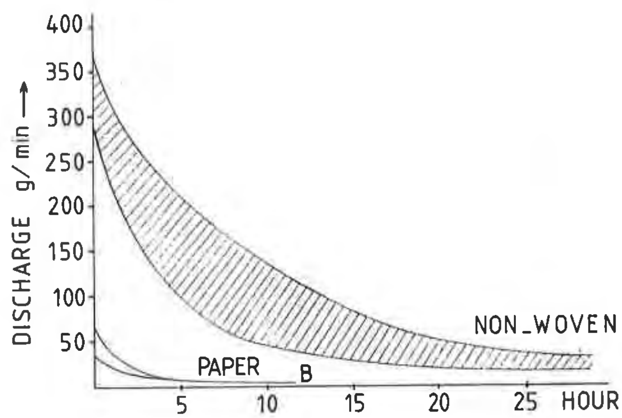


Fig. 4 Filter permeability in wet condition.

Such an upper limit is acceptable for a set of minimum criteria such as those presented in "Drain Standards". The determination of the optimum filter resistance with respect to the soil being drained is beyond the scope of this paper.

The laboratory testing device, apart from some small modifications, is similar to the pressure cell shown in Fig. 5. Very few laboratory tests have been performed for the determination of the required filter efficiency (4.2.2.) and especially with respect to the possibility of blocking the filter.

A research program to study this aspect is being carried out in the Netherlands at the present time.

Useful results are given by Den Hoedt (4) for drains that have been succesfully applied in various projects.

The article also gives an interesting theory concerning the development of a natural filter in the subsoil surrounding the drain. However, the actual occurence of such a phenomenon is difficult to verify.

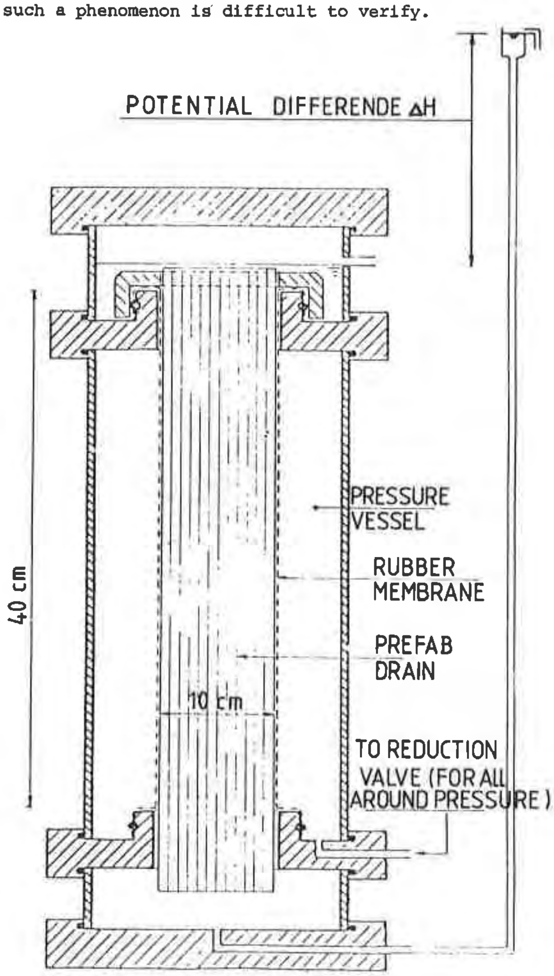


Fig. 5 Test cell.

The ideal manner to confirm this, would be to measure pressures on both sides of the filter sleeve with tiny pore pressure probes.

5.2.3. Discharge Capacity

The discharge capacity of the available drains as measured in the laboratory (3) has in most cases been more than sufficient. At the same time, however, it appears that the vertical discharge capacity can be reduced greatly because of high horizontal pressures (Fig. 5, laboratory testing configuration; Fig. 2, test results).

Furthermore, drains which have been placed in soft soil layers where large vertical strains will develop, may be subjected to strong distortions so that the discharge capacity can be restricted. In the most extreme circumstances, depending on the elastic properties of the drain, it may be folded completely double or even ruptured (Fig. 6).

The lower limit of the vertical discharge capacity is based on those of vertical sand drains. The proposed discharge capacity values of a drain folded double (4.3.2.) has not yet been checked by measurements; this check will most certainly have to be carried out in the future.

5.2.4. Durability and Dependability

To promote the durability and dependability of vertical sand drains, requirements are particularly placed upon the installation procedure, such as a thorough washing of the borehole with clean water, a check on the size of the borehole and a check on certain grain fractions of the filter sand. The filter sand is not subject to any form of deterioration. On the other hand, prefabricated drains may very well experience deterioration, which could be of a chemical, biological or physical nature. However, at the present time little insight is available as far as the deterioration hazards are concerned which various subsoil environments may bring about. As a result, different methods have been examined in which recovery of the drain is possible after an extended period in the soil. A method in which a large diameter casing which was meant to surround the column of soil in which the drain was placed (in order to be able to retrieve the soil plus the drain) did not succeed because of the wedge forming of the soil in the casing.

Fig. 6 Drain folded by vertical strain.

An innovative solution has been found in which the drains are attached to a timber pile in an indentation in the wood. By making a small adjustment to the cross-section of the pile (Fig. 7) the drains will be protected from damage when the pile is extracted.

The disadvantage that the drain is exposed to and drains the soil on one side only is acknowledged.

A set of 6 test piles with 5 different types of drains has been installed at the Diemen test site (2).

The drains will be retrieved at regular 6 months intervals. The tests are being carried out under the auspices of the S.C.W. commission F9 from 1982 to 1984 after which the results will be published.

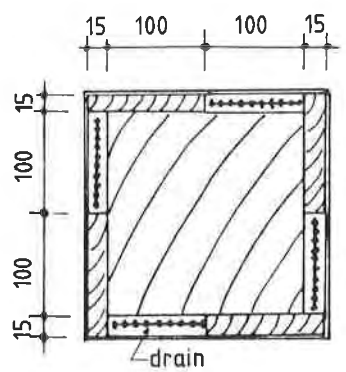


Fig. 7 Cross section of pile for durability test.

5.2.5. Vertical Sand Drains

Up to now, only a few requirements have been placed on the composition of the granular filter material.
A requirement involving the gradation of the sand which can exert an important influence on the working of the drain is not taken into consideration, even though generally moderate permeability test results have been obtained on approved filter sand.
Furthermore, the utilization of a coarse filter sand is important to facilitate the settlement process during the drain installation. The use of fine sands may result in an arch action in the borehole and clay and silt inclusions may also occur.
Along with the material description, the determination of a suitable field procedure was seen as sufficient for the installation of the drains especially in a time of abundant finances and a wide choice of drain materials. However, the current poor economic situation together with the new availability of prefabricated drains have resulted in the need for a more defined formulation of requirements with respect to the material composition, the characteristics and the installation of the drains.

6. ADDITIONAL QUALITY STANDARDS

Drains which are used to facilitate settlements under low embankments are permitted on the basis of an inspection test report according to "Drain Standards" as described in 3.5.
In cases where more stringent criteria are to be applied due to the nature of the works (e.g. a high surface load, marginal slope stability, a limited time for consolidation) or the nature of the subsoil (e.g. low permeability of the soil, very thick soft deposits, high compressibility, low shear resistance) the drains are subject to additional quality standards. This may be achieved by specifying higher limiting values for the tests described in "Drain standards" or by requiring is situ measurements during and after construction periods or during field tests.
To attain this goal, a general procedural method is planned to be set up in which different types of ground works are described with different requirement levels. In view of the fact that in practice many parameters are involved which influence the working of the drain and are dependent on the soil-structure interaction, it is reasonable to set as an initial acceptance criterion the demonstrated succesful operation of the drain in a certain project for soil works in which more demanding conditions are to be expected.
In addition to developing quality standards which are adapted to the in situ application as well as the specific task of the drain in a certain work, a more deliberate method of choosing the drain and determining the drainage design is afforded the engineer as well as the contractor and the supplier.
Also for these additional quality standards the laboratory tests as well as the field measurements must be clearly described.

7. CONCLUSION

To ensure the succesful operation of vertical drains quality specifications have to be determined.
It has been shown that with the aid of laboratory tests a set of minimum criteria may be applied to the drain.
More in situ measurements, preferably of a simple nature, need to be carried out to establish a better relation between the laboratory test criteria and the field results. At the same time, these measurements will stimulate the optimalization of drain design.
It will be greatly appreciated if comments or additions are given on the proposals described in this paper.

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RILEM SM-47 Committee (Synthetic Membranes). Geotextiles Activity Report

Commission SM-47 de la RILEM (Membranes de synthèse). Rapport de l'activité dans le domaine geotextiles.

This report is summarizing the progress in the coordination work dealing with the selection or improvement of standards in the field of geotextiles.
Three types of these materials have been examined belonging to three main groups nonwovens, fabrics and composites. The examined standards are connected with :
a) definitions of geotextile materials;
b) testing methods easiest to apply involved in geotextiles definitions;
c) testing methods of geotextile characteristics immediately needed in designing with geotextiles in civil engineering to solve geotechnical problems.

Le rapport présente l'avancement des travaux du Comité se rapportant à la sélection, voire l'amélioration, des recommandations dans le domaine des géotextiles.
Trois de ces types de matériaux sont examinés, appartenant aux groupes principaux :
non-tissés, tissés et composites.
Les recommandations examinées se rapportent à :
a) la définition des géotextiles;
b) les méthodes d'essais les plus simples à utiliser dans le domaine de la définition des géotextiles;
c) les méthodes d'essais permettant de définir les caractéristiques des géotextiles, d'un intérêt immédiat dans l'étude de leurs dimensions, et destinés à apporter une solution dans les problèmes des constructions géotechniques.

The Committee was set up by the RILEM's Général Council during the annual meeting held (1978) in Athenes (Greece).

The purpose of the Committee is to select and improve some essential testing methods leading to help in choice and in dimensioning of membranes applied in civil engineering and structures.
Three types of synthetic membranes are taken in account:

- stressed membranes, used in air supported structures and roofs;
- geomembranes, used in waterproofing applications;

and

- geotextiles, used in civil engineering to solve geotechnical problems.

All this three types of materials are synthetic ones and work as membranes from mechanical point of view.

Membrane is a structural element having a strength in its plane but its flexural rigidity is zero.
The principal performances of stressed membranes are : hight strength, low deformability, low creep, water and air proof, weathering stability.
The principal performances of geomembranes are : strength, suppleness waterproof and weathering stability in contact with soil, water, pollution bacterias and so one.

As for geotextiles, their principal performances are of mechanical and hydraulic character but in extremely various aspects.
Therefore, the Committee decided to work in three subgroups and in view to gain a profit from discussions in each of them, to hold periodic general sessions.
It must be underlined that the Committee devoted much more effort in the field of geotextiles than with stressed membranes and geomembranes.
The principal reason is that the applications of geotextiles in civil engineering to solve geotechnical problems have increased very rapidly in last years.
By the way many national geotextile Associations have been founded for preparing national requirements in this field. So it becomes urgent to have an international coordination in this matter to avoid the differences between requirements and standards and to make easier the exchange of various products on the international level. Nevertheless this work is very large and difficult and must be made by steps.
Thus the Committee decided that the first step in this field should be to select or improve standards which would be :

a) well recommanded to define the geotextile materials divided in three large groups :
   - nonwovens;
   - fabrics; and
   - composite ones;
b) the easiest to apply for testing some characteristics involved in material definitions;
c) useful to define any material characteristic immediately needed in designing with geotextiles in civil engineering to solve geotechnic problems;

d) to help in coordination with international definitions and symbols used in geotextile testing and designing.

Fixed these purposes, the Committee is composed with people specialized in testing of geotextile materials and soil, thàn with these ones specialized in designing with geotextiles, with people involved in the works of national geotextiles societies and finally of people representative of geotextiles manufacturers.
84 countries being members of R.I.L.E.M. (The International Union of Testing and Research Laboratories for Materials and Structures), the SM-47 Committee is composed of people from many countries but especially from these having a National Geotextile Society.
As yet, SM-47 Committee has held 6 Meetings (June 79 and November 79 in Liège, May 80 in Zürich, May 81 in Glasgow, October 81 in Gdansk, November 81 in Liège).

The following documents are preparing :
the IDENTIFICATION CARD for GEOTEXTILES and RELATED MATERIALS (composites, sandwiches a.s.o.) with the following six headings :
a) Commercial name;
b) Identity of Manufacturer;
c) Method of Manufacture and Constituent Materials;
d) Mass per unit area;
e) Nominal thickness;
f) Form and Packaging.

The principal aim as concerned the first three points is to develop a clear definition of terms used in description to avoid any ambiguity.
As for Mass per Unit area, the principal problem is to find clear description of anisotropic composed materials and establish an easy method of sampling. Of course for some sandwich containing laminated parts, the mass is not uniformly distributed; the size of sample taken in account in this case must be wittingly adapted.
We know that the thickness of nonwoven and some sandwich is connected with the pressure as well as with permeability. Thus the thickness of some materials are defined by a curve thickness/pressure or by some data related thickness to the pressure. For the identification purpose it is enough to have easy to get one point figure; the Committee adopted and recommands for Nominal thickness test the pressure of 2 kPa.
This chapter IDENTIFICATION CARD is coordinating with ASTM Subcommittee (D. 1361 on Geotextiles).

The second important chapter of SM-47 activity is devoted to the test methods needed to define some material characteristics immediately used in designing. They are essentially of mechanical and hydraulic character both extremely important for designing. Therefore a large number of testing methods was developped to measure the materials performances in these two aspects. Without underestimate the scientific value of any test elaborated in the field of strength and elongation by break, the SM-47 adopted and recommends a traction test with width to length ratio as 5 to 1 (0,5 m width to 0,1 m length, in applied force direction).
If for any material the proof is given that the strength and break alongation in a given direction are independent of t/ℓ ratio, any other t/ℓ ratio may be adopted to test such a material.
As for hydraulic test, this in plane and this transverse, permeability are under scruting. In this field, also a large variety of tests was developped and are developping.
If for nonwoven macro-isotropic material the choice of in plane and transverse permeability tests seems easier to do, it is not the same with fabrics and composite anisotropic ones. As for permeability-pressure test the problem can be solved step by step in experimental way or using an analytic function.
In this aspect, the fabrics are less sensitive than nonwoven and composite materials.
But it's easy to understand that in contact with soil some soil particles may reduce the permeability coefficient measured without soil. Both nonwoven and fabrics may be affected less or more depending of the soil nature and its saving curve.
So it will be useful to adopt the permeability coefficient and the permeability pressure curve as a Nominal characteristic and then to recommend the permeability test, the material being in contact with an actual soil or with a standard soil. The results of some long term permeability tests of geotextiles in contact with actual soil show the interest of this tests for designing.

For the filter function, the porometry of geotextile structure is of importance. Here also many dry or wet methods were experimented and are developping using a standard sieve curve granulats of glass or in sand; other prefer a soil which will be actually in contact with a given geotextile. To satisfy these two points of view maybe it will be wise to foresee two tests : one for describe a standard porometry or Nominal porometry and a second one as a long time simulation test. As yet, any decision has been taken in this field as with the tests for evaluating the tearing strength, the sliding resistance and the durability of geotextiles.

The SM-47 SM Committee is hoping that many Manufacturers, Consultants, Civil Engineering Contractors, National Geotechnical Societies, should be interested in the work undertaken.
Any comments and suggestions will be welcome to improve the quality of prepared recommandations, to make the testing methods easier more sure, more precise and more general as well as more adopted to the practice of designing and of use of geotextiles.

In view to underline the importance of geotextiles use in civil engineering, the variety of different materials on the market and the importance of testing methods for classification of geotextiles and to get their performances needed for designing, the SM-47 Committee sponsored the special edition of the RILEM Review "Materials and Structures", nr 82, June-July 1981.
The author of this report is indebded to Mr M. FICKELSON, RILEM General Secretary and Chief Editor of Materials and Structures, for a rapid and careful edition of nine papers devoted to Geotextiles designing, use and to test their performances.

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Recommendations of the PIARC on Methods to be Used for Testing Geotextiles

Recommandations de l'AIPCR sur les méthodes d'essais à utiliser pour l'emploi des géotextiles

The paper considers and describes summarily the most appropriated test methods in the field of roads works. The treated tests are : Resistance to tensile stress and elongation, Tear resistance, Permeability, and Porometry.

La communication envisage et décrit sommairement les méthodes d'essais jugées les plus opportunes pour la technique routière. Les essais traités sont : la résistance à la traction, la résistance à la déchirure, la perméabilité et la porométrie.

INTRODUCTION

Un des objectifs actuels du Comité Technique des Essais des Matériaux Routiers (A.I.P.C.R.) consiste à publier des recommandations pour l'exécution des essais. Etant donné - l'emploi de plus en plus fréquent de géotextiles en technique routière;
- le nécessité (grand nombre d'essais pratiques) d'aboutir à une standardisation;
- l'inadaptation à la technique routière de certains essais préconisés par l'industrie textile;

le Comité Technique précité a décidé de procéder à l'élaboration de ces recommandations pour les géotextiles.

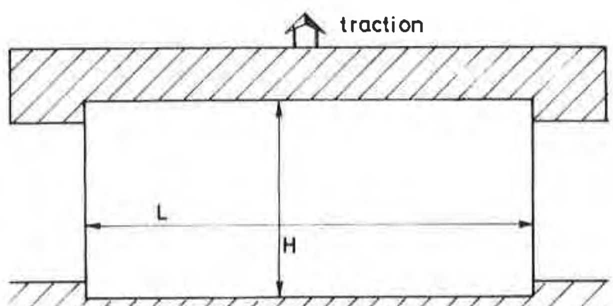
Ces recommandations ne sont pas à proprement dit des modes opératoires détaillés mais elles donnent et décrivent succinctement les essais qui sont les mieux adoptés à la mesure de la caractéristique envisagée.

Au stade actuel, seuls les essais suivants ont été envisagés :
- résistance à la traction et allongement;
- résistance à la déchirure;
- perméabilité;
- porométrie.

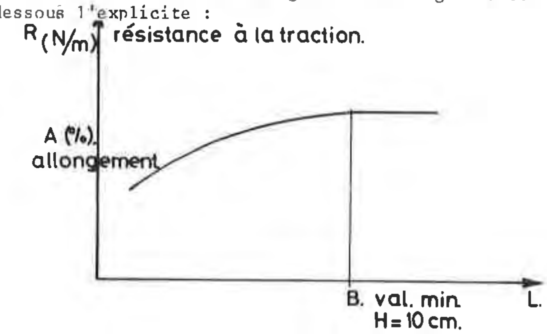
RESISTANCE A LA TRACTION ET ALLONGEMENT

L'essai retenu est celui qui utilise des éprouvettes rectangulaires et dont les dimensions peuvent être définies par :

- hauteur (H) (distance entre pinces) : minimum 10 cm.
- largeur (L) telle que L/H $\geqslant$ 5.



Il peut toutefois être dérogé à ce rapport L/H $\geqslant$ 5 si il est démontré que la diminution de ce rapport n'engendre pas de variation de la valeur de la résistance à la traction et/ou de l'allongement. Le diagramme ci-dessous l'explicite :



Excepté les dimensions des éprouvettes (voir supra) le mode opératoire se réfère essentiellement à la norme AFNOR G 07.001.

Cinq éprouvettes sont donc soumises à l'essai et la moyenne arithmétique des cinq résultats constitue la valeur consignée.

En ce qui concerne l'allongement, la formule suivante est d'application :

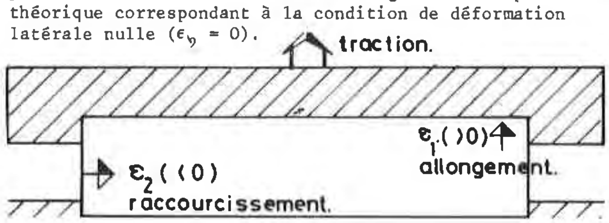
$$\epsilon_R = \epsilon_1 + \epsilon_2 + \epsilon_1 \times \epsilon_2$$

où : ϵ_R = valeur recherchée (exprimée en %)

ϵ_1 = la moyenne arithmétique, exprimée en % des déformations à la rupture obtenues sur chacune des cinq éprouvettes soumises à l'essai (sens de traction)

ϵ_2 = la moyenne arithmétique exprimée en %, des déformations correspondant à ϵ_1, des cinq éprouvettes soumises à l'essai (sens perpendiculaire ou sens de traction). Ces déformations sont mesurées au milieu de l'éprouvette.

L'application de la formule précitée permet de rapprocher la valeur mesurée de l'allongement de rupture théorique correspondant à la condition de déformation latérale nulle ($\epsilon_\nu = 0$).



RESISTANCE A LA DECHIRURE

Tout comme pour la résistance à la traction, l'éprouvette doit être de grandeur suffisante pour que les forces qui provoquent la déchirure puissent se développer dans la structure.

L'essai retenu est l'essai sur éprouvette trapézoïdale avec déchirure amorcée. Les dimensions de l'éprouvette sont reprises à la figure n° 1.

Cet essai est réalisé à faible vitesse (100 mm/min.) L'effet d'une déchirure dynamique n'est donc pas considéré.

L'A.S.T.M. n° 2263, malgré qu'elle ne soit plus en vigueur, sert de référence de base pour la procédure.

Pour chaque direction (sens production et sens travers du géotextile), la résistance à la déchirure du géotextile (exprimée en N.) est la moyenne arithmétique de la résistance à la déchirure obtenue sur cinq éprouvettes.

La résistance à la déchirure (exprimée en N.) d'une éprouvette est la moyenne arithmétique des 5 forces maxima obtenues en découpant conventionnellement le diagramme "force - allongement" en 5 zones.(voir figure n°2)

PERMEABILITE

La perméabilité dans le plan et la perméabilité normale au plan du géotextile sont envisagées respectivement par la mesure de la transmissivité et de la permittivité.

1. Mesure de la transmissivité

L'essai consiste à mesurer, sous charge hydraulique constante, le débit d'eau traversant un géotextile dans son plan ou fonction de la pression de compression auquel il est soumis.

La transmissivité θ (cm2/s) s'exprime par :

$$\theta = k_p \times H_g = A \times Q$$

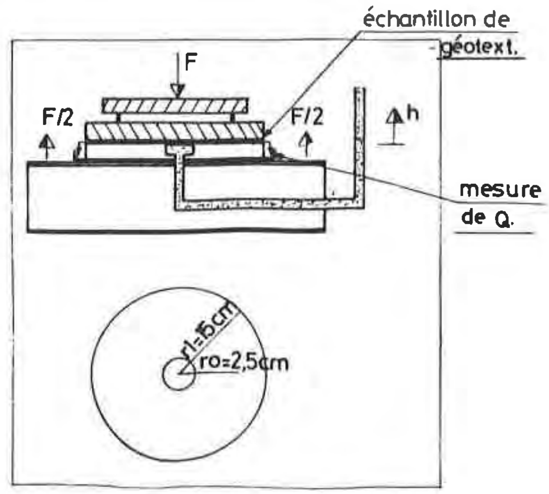
où k_p = coefficient de perméabilité dans le plan du géotextile (cm/s)

H_g = épaisseur du géotextile (cm)

Q = débit d'eau traversant le géotextile (cm3/s)

A = constante de l'appareillage (cm^{-1})

Le schéma de principe de l'appareil permettant cette mesure est repris à la figure ci-dessous:



Disposition de l'essai

L'alimentation en eau s'effectue par le centre du système. La charge hydraulique h est constante et fixée à 50 cm. Le dispositif de compression de l'échantillon s'effectue par un plateau répartiteur sur lequel agit un vérin (maximum 5 tonnes).

La référence de base est : "Mesure de la perméabilité dans le plan des géotextiles non tissés (J.M.RIGO) Revues "Ingénieurs et architectes Suisses" 12 avril 1979.

La procédure peut être résumée comme suit :

- prélèvement de l'éprouvette;
- conditionnement de l'éprouvette;
- mise en place de l'éprouvette dans l'appareillage (élimination des bulles d'air)
- application de la charge hydraulique h = 50 cm;
- mise sous pression, par palier de 100 k Pa, de l'éprouvette et stabilisation de cette pression (10 minutes);
- mesure du débit d'eau traversant l'éprouvette.

Ce débit est ramené à une température de 10°C.

La transmissivité est calculée par la formule suivante :

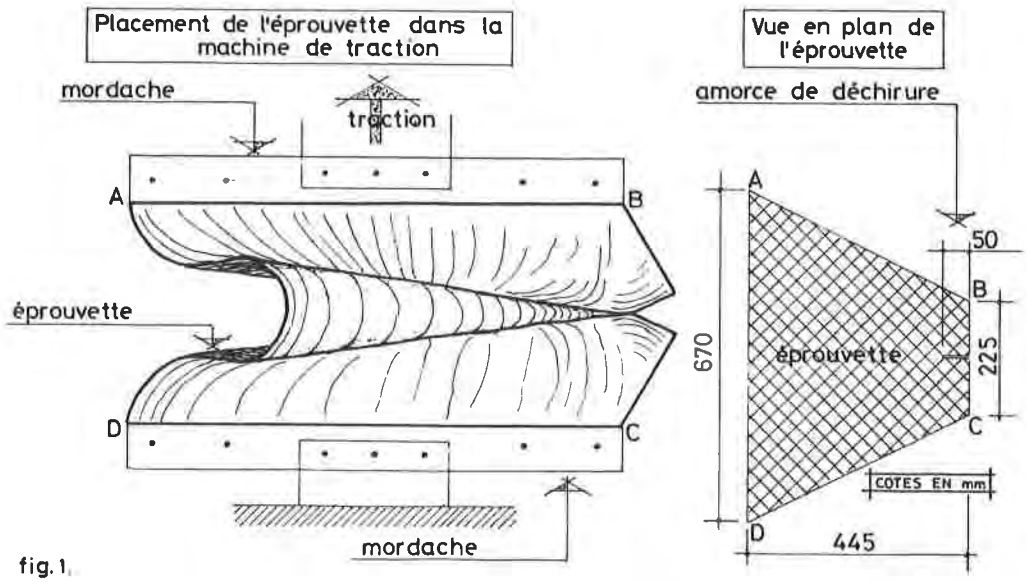


fig. 1

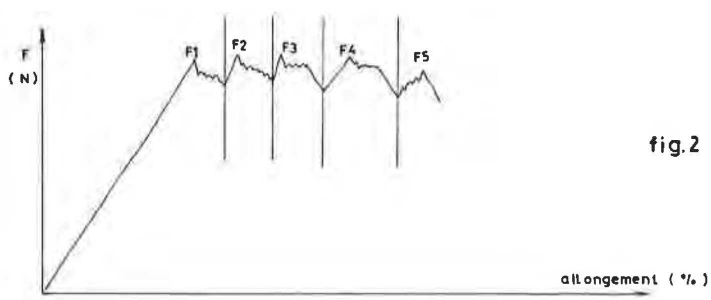


fig. 2

$$\theta = \frac{Q}{2\pi h} \ln \frac{r_1}{r_0} \quad (cm^2/s)$$

où : h = charge hydraulique (cm);

r_1 = rayon des plaques métalliques servant à mettre en compression l'échantillon (15 cm);

r_0 = rayon de la zone d'alimentation (2,5 cm)

Q = débit d'eau traversant le géotextile (cm^3/s) et correspondant à une température de 10°C.

2. Mesure de la permittivité

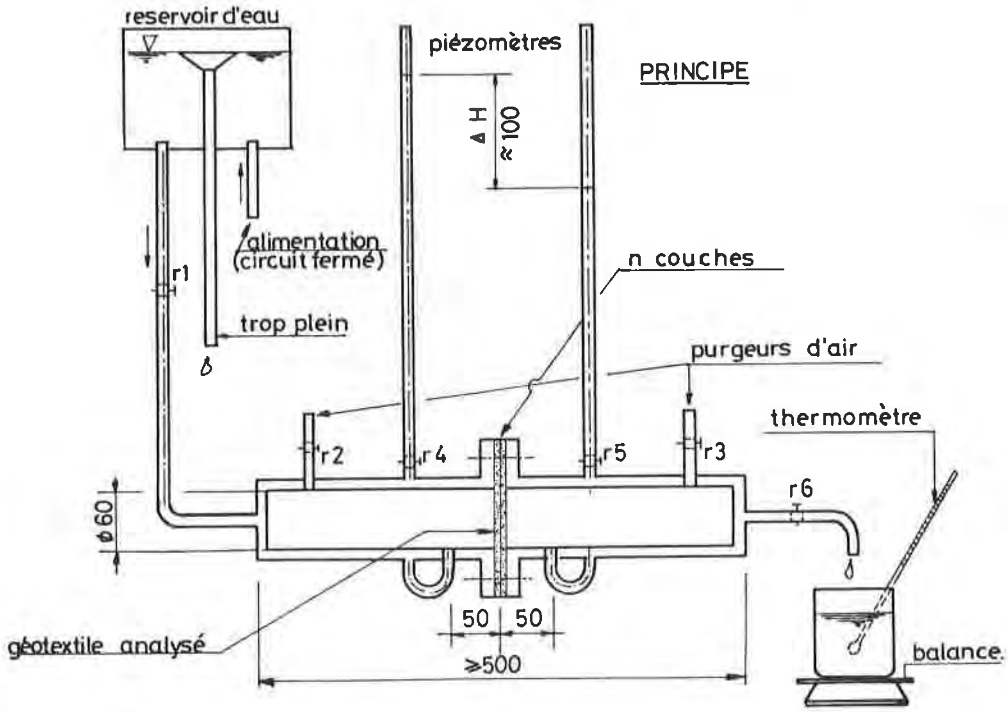
L'essai consiste à mesurer le débit d'eau Q traversant transversalement une section S de géotextile. Cette mesure se fait sous une charge hydraulique h.

La permittivité ψ (s^{-1}) vaut :

$\frac{Q/S}{\Delta h}$ (différence de niveau d'eau entre amont et aval échantillon)

La courbe exprimant Q/S en fonction de Δ h n'est linéaire qu'en écoulement laminaire. Pour ce faire, on maintiendra le débit (par unité de surface inférieur à 0,035 m/s. Cette condition peut toujours être réalisée si on augmente le nombre de couches de géotextile.

L'exemple d'un perméamètre respectant ces conditions est donné à la figure ci-dessous :



La procédure est simple. Après avoir placé l'échantillon (1 ou plusieurs couches), on purge l'appareillage de l'air qui y est contenu. On mesure, ensuite par réductions successives des débits d'eau, les couples de valeurs Q − Δ h.

La permittivité correspond au coefficient angulaire de la partie linéaire du graphique Q/S en fonction de Δ h/n (n étant le nombre de couches de géotextile utilisé)

POROMETRIE

L'essai consiste à caractériser la distribution dimensionnelle des ouvertures du géotextile.

L'essai préconisé est basé sur l' "Equivalent Opening Size" de l'U.S. Corps of Engineers (Federal Highway Administration U.S.A.).

On peut utiliser toutefois, soit des billes de verre, soit des grains de sable.

Chaque fraction calibrée de sable ou de billes de verre est tamisée à sec (5 minutes, fréquence de vibration : 50 Hz, amplitude : 0,75 mm) sur le géotextile qui sert donc de tamis.

Le géotextile est caractérisé par les diamètres moyens (O_{95} et O_{98}) des particules correspondant aux valeurs pour lesquelles 95 et 98% des fractions (de sable ou de billes) sont retenues par le géotextile.

Cet essai n'est pas considéré comme suffisamment précis pour des tissus épais ou lorsqu'il faut tamiser des particules inférieures à 80 µm.

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Development of Test Methods by ASTM

Développement des méthodes de test par ASTM

This report presents the history and significant developments of the American Society for Testing and Materials efforts toward adopting standard test methods for geotextiles. It explains the type of input from members, the qualifications and background of the membership and typical format for adopting test methods. The division of effort is outlined and the relationship to the two parent groups within ASTM presented. Major controversies are discussed such as the questions of geotextile pore size in field performance, the significance of sample width in tensile tests, the meaning of geotextile permeability and the desirability of adopting index tests. Test method areas that will probably be investigated in the future are presented. A summary of accomplishments and work to be done is outlined.

Ce rapport expose l'histoire et les développements significatifs des efforts entrepris par la Société Américaine pour le Contrôle, concernants l'adoption des méthodes d'essai standard pour les géotextiles. Ce rapport explique le type de contribution fournit par les membres, les qualifications, les origines de la société et le format type pour l'adoption des méthodes de test. La division de l'effort est soulignée, et la relation avec les deux groupes apparentés á l'intérieur d' ASTM est présentée. Les controverses principales sont discutées, telles que les questions concernant la taille des pores géotextiles et leur fonctionnement sur les gisements, I'importance de la largeur des échantillons d'essai de tension, la signification de la perméabilite géotextile, et l'avantage d'adopter des tests d'index. De domaine des méthodes de test qui seront probablement examinées á l'avenir, est aussi présenté. Un résumé des réalisations et du travail á faire est exposé.

INTRODUCTION

The American Society for Testing and Materials has been actively developing test methods for geotextiles since 1977. Individuals working on the ASTM effort represent geotextile manufacturers, users, contractors and design engineers. Participation is primarily from North America and Europe. The group meets twice each year and uses the intervening months to draft standards, participate in interlab testing programs and vote on proposed methods.

The goal of the ASTM geotextile group is to adopt meaningful test methods after careful study and critique from the membership. This has proven to become a long and difficult task because of the uncertainties of a new technology and the needs to satisfy scientific concerns from different points of view.

Although the geotextile group has met 10 different times since its beginning, there are more standards to be adopted than have been written; there are more controversies to resolve than have been resolved. This paper will present the efforts that have been underway and includes several of the most important questions needing to be answered.

THE BEGINNING

A questionaire was distributed by ASTM early in 1977 requesting those having an interest in geotextiles to comment on their experiences and to respond to the question of the need to establish an ASTM subcommittee on geotextiles. This first step was followed by scheduling a meeting on geotextiles in New York City in October of 1977. The meeting resulted in establishing a geotextile subcommittee organized by the ASTM main committee on textiles.

The first year and a half for the new subcommittee could be characterized as a learning period with the three meetings consisting of seminars and discussions. Much work had been done by many individuals on the subcommittee including developing a number of different test methods and procedures. The membership could generally be placed in one of two groups. The textile scientists tended to be interested in coverting existing textile testing methods to match the requirements of geotextiles. The members with the geotechnical engineering training were more likely altering existing soil test methods to measure geotextile properties. The textile person was more familiar with small sized samples of fabric tested in special devices that resulted in an index value useful to compare one product with another. The geotechnical person preferred larger size samples with large, slow moving test apparatus. The samples were often tested in contact with soil and the resulting test value was often used in a design equation.

The geotextile subcommittee has grown since 1977 to over 100 members. In June of 1980 the membership decided to gain more input from the user community and began meeting during the time scheduled for the main committee on soil and rock. When this change was made, the textile subcommittee officially became a joint ASTM subcommittee under both textiles and soils. This joint relationship with two main groups within ASTM is unique. It allows the early input of the textile scientist to review those test methods having to do with the geotextile alone. Test methods that include the use of soil or attempt to model field use are reviewed by the geotechnical members in the soil and rock committee of ASTM.

THE INTERMEDIATE STAGE

After the first year and a half, the subcommittee began the activity of developing different test methods and procedures. Most of the meeting time was devoted to reports on specific tests that had been written. After the method was critiqued in committee, the draft standard was sent to a main committee editorial body and returned to the author for revision. The standard was then reviewed by the geotextile members or sent to ASTM headquarters for a formal ballot.

The geotextile subcommittee was divided into four different sections with a chairman to coordinate and report the efforts of each section. The sections are 1) terminology, 2) mechanical properties, 3) endurance properties, and 4) permeability. The division of effort was arbitrary and only for the purpose of better handling the efforts. A test method that was completed in a relatively short testing time that concentrated on measuring a basic parameter such as weight per unit area or tensile strength was developed in the mechanical properties section. A test method to measure a long range property, such as a biological effect, was discussed in the endurance properties section. All tests having to do with the flow of a fluid through the geotextile or a method to measure the pore size or distribution of pores within the fabric were handled by the permeability section. As test methods and practices were developed, special terms would enter the discussions that needed to be defined. This portion of the subcommittee effort has proven to be one of the most difficult tasks of all.

Each time a test method is written or ammended, the meaning of certain terms becomes a major part of the discussion. The time to complete the review and approval for a single term is more than doubled because of the nature of the joint subcommittee. Having to please individuals who review the work from two different backgrounds and who do not necessarily take part in committee meetings and discussions is a difficult task. For example, the subcommittee has yet to officially adopt an approved definition of the word "geotextile". The problem with this term is that it must be used in the methods that are directed through the textile and geotechnical memberships. It would seem reasonable to include the word "geotechnical" in the definition since this is the root of the prefix to geotextile. Although the geotechnical engineers have no problem with the word geotechnical, some textile scientists believe that geotechnical is not commonly understood by the textile community and should not be used in a definition.

The mechanical section has been working on a number of test methods to include those presented as follows Table I. Some of the methods under development are revisions of existing ASTM published standards; others are new methods.

Some workers have suggested that a more appropriate test for measuring field performance would be to test a much larger sample of fabric than the size prescribed in the standard "grab tensile" test. The wide width tensile test should better fill this need. Stiffness is primarily an indication of the ease of installing the material. Seam strength is a new area of study by the section to measure the field property of the integrity of the complete geotextile structure. Puncture, Impact and Friction testing measure the fabric resistance to stresses during installation and end use. Trapezoidal tearing strength is a modification of an existing ASTM method. Basis weight, thickness and compressibility are properties often used to detect conformity of a fabric rather than an end use property.

TABLE I: METHODS UNDER DEVELOPMENT BY THE MECHANICAL PROPERTIES SECTION

1. Wide Width Tensile	5. Impact Resistance
2. Stiffness	6. Friction Testing
3. Seam Strength	7. Trapezoidal Tearing
4. Puncture Resistance	8. Basis Weight, Thickness, Compressibility

The endurance property test methods are presented in Table II. Fatigue, creep and abrasion tests are mechanical test methods to measure the ability of the geotextile to specific long term conditions that may be encountered in the field. These methods are specific to geotextiles and require the development of testing devices as well as writing new procedures. Ultraviolet light exposure, hot air and thermal shrinkage, chemical stability and biological resistance test methods are generally modifications of existing standards. These tests consist of exposing the fabric to certain lab conditions and comparing a measured difference in a mechanical property with an unexposed sample. These tests allow the worker to select the mechanical property test of interest.

TABLE II: METHODS UNDER DEVELOPMENT BY THE ENDURANCE PROPERTIES SECTION

1. Fatigue Testing	4. Ultraviolet light exposure
2. Creep Testing	5. Hot air & thermal shrinkage
3. Abrasion Testing	6. Chemical Stability
7. Resistance to Bacteria and Fungi	

The permeability section test methods are presented in Table III. These methods measure the resistance to flow of water or another fluid through the geotextile. Other methods measure and rank the size of the pore openings. The permittivity test was an index method using water as the fluid and neglecting the thickness of the fabric tested. The size of apparent opening test is a revision of a test to determine the size of hole present to give a design engineer an understanding of the use of the fabric as a filter or soil retention device. The soil/geotextile permeability test is to determine if the geotextile acts to restrict the flow of water through the soil with time.

TABLE III: METHODS UNDER DEVELOPMENT BY THE PERMEABILITY SECTION

1. Permittivity
2. Size of Apparent Opening
3. Soil/Geotextile Permeability

CURRENT EFFORTS

It has been observed that the subcommittee has been progressing through three different stages. The first was when the membership were just learning what had been done through seminar presentations. The intermediate stage consisted of dividing the tasks into separate areas and getting test methods from the conceptual state to a membership ballot. The present stage consists of establishing a workable system to transform the first balloted standards into published methods.

To this end, the subcommittee has completed only a few ballots. No test methods have yet been adopted by ASTM. The subcommittee is now in the process of redirecting its efforts toward satisfying negative comments from the membership on currently written draft methods rather than generating new procedures and methods.

SIGNIFICANT CONTROVERSIES

Some of the differences of opinion have been discussed such as the definition problem with the word geotextile. An early problem was to decide the difference between a test method and a specification. Several members understood that when a test method was developed a suggested range or limit would be placed on any value that was determined. The decision by the subcommittee was to delay writing specifications listing specific test result values until after many of the basic test methods were adopted.

There was a significant difference of opinion over the usefulness of a measured value from a specific test. Most of the tests originating from the textile community could be classified as index tests that did not give a number that could be directly used in a design equation. Some of the tests were better indicators of the strength of a fabric used as a garment but had little meaning if the material was installed under railroad ballast. Subcommittee members have decided to make a judgement for each test with regard to the use of the test results to compare geotextiles or to give a design number. For example, the permittivity test is considered an index test while the soil/geotextile permeability test is for design. The permittivity test gives a value that represents the volumetric flow rate of water through a sample of geotextile at a specified hydraulic head. It does not represent common field conditions. The number obtained would be useful to compare with another fabric to determine relative ability to pass water. The soil/geotextile permeability test, on the other hand, more closely represents field conditions by placing the actual project soil in contact with the geotextile and measures a field-like installation at specified conditions.

The permeability-permittivity controversy began when it was pointed out that the thickness of a geotextile was not important in most soil restraint installations. Before the use of geotextiles, the engineer could design an "aggregate filter" that would restrain the soil yet provide drainage. The permeability of the soil and aggregate filter were obtained with the same test method. Permeability is a parameter that indicates the flow of a fluid at a unit hydraulic head per unit time, per unit area, per unit thickness. It seems reasonable to require the geotextile to be at least as permeable as the soil that it drains. Therefore, many workers developed geotextile permeability test methods and rejected fabrics based on the test results. Since only a single layer of fabric is used in most installations, it is not important what the flow rate per unit thickness is but rather the flow rate per layer. This question resulted in developing a new method and a new term called permittivity.

The desire of the geotechnical engineer to test larger size samples at slower speeds has been discussed. However, other than obtaining a different test value, there has not been a positive determination that the numbers generated by the smaller sized test cannot be used in design. One problem with standardizing test sample size and travel speed is that the two technologies have different machines. The geotechnical engineer would prefer to test fabric samples on soil test devices that operate slower and accept larger size samples than those devices in fabric testing labs. One of the major problems with testing large size samples is to make certain that the device is testing the fabric and not the ability of the grips to hold the fabric. The newer products are often very thick with multiple layers. This type of fabric increases the problems of grip slippage.

Pore size determinations have been important points of dicussion to the design engineer to ensure that a fabric does not have a pattern of openings that will allow a significant loss of soil or will clog. It has been assumed that one would have a reasonably clear understanding of the ability of a particular fabric after obtaining a pore size distribution and permittivity value. However, there is much controversy over determining pore size and distribution, what it may mean, and how to use the result from a permittivity test. The controversy over how to measure and use pore size information is the greatest issue before the subcommittee.

NEW AREAS

There are several areas that have not been covered by the subcommittee that will probably be addressed in the future. These include tests for the integrity of seams through sewing and sealers, asphalt retention tests, and other dynamic strength tests.

SUMMARY OF ACCOMPLISHMENTS AND WORK TO BE DONE

Over the past five years the ASTM joint subcommittee on geotextiles has passed through the stages of grasping the state of the art of geotextile testing to the long process of adopting standards. Currently, the subcommittee is working with 20 different test methods from the conceptual through the first ballot of the membership. There is considerable rewriting to be done and much consideration before the first geotextile standard will be published in the ASTM book of test methods. It is expected that the first series of tests may be approved next year.

The work that has been done is a credit to the many hardworking individuals who have given their time and talents to the effort. The work can only continue if the subcommittee continues to receive the support from dedicated professional.

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Trial Use Results and Experience Using Geotextiles for Low-Volume Forest Roads

Resultats des usages experimentaux des géotextiles dans les voies peu circulation des forets

In 1973, for the first time, geotextiles were considered for use in the Pacific Northwest Region of the U.S. Forest Service. The initial investigations were to use woven monofiliment fabrics as a replacement for graded aggregate filters. These early investigations were followed by construction of two instrumented trial use installations; one was a fabric reinforced retaining wall in 1975, and the other was a subgrade installation on soft ground in 1976. The concepts developed or discovered in these investigations and used in the trial use installations formed the basis for a guideline report published in 1977 for the use of fabrics in low-volume road construction and maintenance. As a result of this work, design methods and specifications have been developed and are being implemented to effectively use geotextiles to build and maintain more reliable roads at lower cost.

En 1973 on a examiné pour la première fois l'usage des géotextiles dans les forêts de la région nord-ouest des forêts nationaux. Dans les premières expériences on a employé les matières tissés à un fil (monofiliment) our remplacer des substances employées avant comme des filtres matériau granular. Ces premiers essais ont ete suivi de la construction de deux installations d'épreuve, réglées à l'usage des instruments, dont l'un, en 1975, était la construction d'un mur à retention renforcé de textile, et l'autre, en 1976, était l'installation des textile sous l'agrégat sur la terre molle. Les idées developés ou decouerts au cours de ces expériences sont à la base d'une dissertation publiée en 1977 au sujet de l'usage des textiles dans la construction et l'entretien des voies peu circulation. Le resultat de cet ouevre était le développement des methodes et des spécifications dont on se sert pour bien employer ces "géotextiles" pour construire et entretenir des voies aux prixs plus favorables.

INTRODUCTION

Geotextiles were first seriously considered for use in Region 6, of the US Forest Service in 1973. Initial use was to replace graded aggregate filters. Geotechnical engineers hired on several National Forests in the late 1960's designed graded aggregate filters based on soil gradations replacing ungraded rock drains. Graded aggregate filters proved very successful, but properly graded aggregates were difficult and expensive to acquire and install, particularly in remote areas with silty soils. Fabrics were looked at as a method of providing reliable underdrains at a low cost. Initial projects were favorable so investigations into new uses and methods continued. Today, geotextiles are specified and used extensively on low-volume aggregate and bituminous paved roads for drainage, road subgrades and retaining walls.

The Pacific Northwest Region (referred to as Region 6 or R-6), includes 19 National Forests containing approximately 11 million hectares or 27 million acres. The Region has a 137,000 km (85,000 mile) transportation system with plans for another 19,000 km (12,000 miles). Annually, 2,500 km (1,600 miles) of roads are constructed or reconstructed at a cost of $160 million with another $35 million spent on maintenance. Most of these roads are located in remote areas with mountainous terrain. About 96% of the transportation system is either aggregate surfaced or unsurfaced.

Road design and construction supervision is performed by National Forest personnel. Technical assistance and program monitoring is provided by the Regional Office. Most roads are constructed for log haul.

Papers written concerning geotextile use are based on observations, tests, experiences, and viewpoints of the authors. Typically, manufacturers and marketers relate to the benefits of their fabric, university professors present theoretical aspects and new ideas, consultants present applications to specific projects, often very large projects, and users relate to what was done and how well it worked. This paper presents some history, experience and philosophy concerning use of geotextiles in the Pacific Northwest Region of the U.S. Forest Service. The viewpoint in this paper is that of a user knowledgeable in many of the theoretical aspects of geotextiles and the very real challenge of effectively using these materials in widely varying field conditions. Design methods and specifications are aimed at cost effective utilization of geotextiles within current operating conditions. Short and long term benefits are considered, with an emphasis on minimizing challenges for designers, construction inspectors, and road builders.

This paper discusses the work done in Region 6 to establish current design methods and specifications. The information should be useful to both geotextile users and suppliers.

DEVELOPMENT OF GEOTEXTILE USE WITHIN REGION 6

The first geotextiles used in the Region in 1973 were woven monofiliment fabrics, meeting the US Army Corps of Engineers specifications based on Calhoun's research (1). The fabrics were used to replace graded aggregate filters in subsurface drains along roadways and in landslide stabilization. The fabrics worked very well, helping stabilize several areas that had been unsuccessfully stabilized using other methods. The fabrics were found to be easy to use once the contractors got over the shock of using "new methods." The costs were much lower than using graded aggregate filters and the reliability judged much higher (2).

The uses that followed in 1974 and 1975 were subgrade restraint, subgrade separation, and retaining wall reinforcement. Limited amounts of fabric have also been used for erosion control and pavement cracking.

Restraint is the use of a geotextile to increase system strength and reducing aggregate thickness, whereas separation is the process of preventing two materials from mixing. Initially, fabric samples were donated by manufacturers for limited trial subgrade uses and laboratory testing. The subgrade installations were based on a combination of several manufacturer's recommendations. The retaining walls were built using design recommendations from Bell (3).

The early fabric trials included limited testing to gain understanding about the fabrics and their properties. Due to limited budgets and facilities, testing was usually limited to that necessary to answer immediate questions, rather than to develop and validate standard procedures. Test methods and procedures have changed as a result of testing by ourselves and others.

Field visits and discussions with several researchers and manufacturer's representatives resulted in construction of two instrumented trial use projects, including different fabric brands and weights. An instrumented fabric test wall was built on the Shelton District, Olympic National Forest in 1975. Several instrumented fabric road test sections were built on the Quinault District, Olympic National Forest in 1976.

This work with geotextiles resulted in a report "Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads", and reference notebooks for Region 6 National Forests in 1977. The publication was "written to increase awareness and improve understanding of the function and uses of fabrics" (2). The rapidly moving technology and increasing availability of fabrics for use in construction made the publication necessary. The remainder of this paper deals with materials evaluations, design methods, and specifications, with primary emphasis on road applications.

LABORATORY EVALUATION OF MATERIALS

A difficult task has been to find fabric tests that have wide acceptance by users and producers, are easy to perform, and have been correlated with the intended performance. Tests meeting these criteria are still being pursued.

A number of tests have been performed in our laboratory to gain information, to confirm testing by others, or to check for compliance with specifications. Tests our laboratory has performed are:

- Grab (ASTM D-1682)
- Cut strip 25.4 millimeter (1 inch) (ASTM D-1682)
- E.O.S. (Equivalent Opening Size)
- Percent Open Area
- Weight/Unit Area
- Gradient Ratio (4)
- OSU Ring Test (5)
- Wide Cut Strip Test (up to 381 millimeter or 15-inch width)
- Seam Strength (ASTM D-1683)

Visual tests have included determining the type of fabric construction, type of fiber or filiment, the uniformity of fiber distribution, and effectiveness of the bonding process. Grab and cut strip tests have been performed on several fabrics before installation and after 1-3 years of service.

The standard grab strength and 25.4 millimeter (1-inch cut) strip test results have no direct correlation with the strength required in a retaining wall to resist imposed static and dynamic loads. The OSU ring test (5) and the wide cut strip test were performed to obtain a better measure of fabric strength when used as a wide tensile member, such as reinforcement in a retaining wall. Both tests are relatively simple, and can be performed using standard tension/compression testing machines. The ring test is performed using a converted CBR mold and plunger as shown in Figure 1. The wide cut strip tests are performed on fabric widths less than or equal to the jaw width. Figure 2 shows the results of wide cut strip tests performed with two different fabrics. For nonwoven fabrics, the cut edge greatly effects the average strength per inch of width for strips less than about 152 millimeters (6 inches) wide. The wide cut strip test and ring test yielded similar results for samples greater than 152 millimeters (6 inches) wide. We abandoned the ring test in favor of the wide cut strip test since the strength measurement is more conventional and direct. The cut strip test is used to check fabric strength for retaining wall applications.

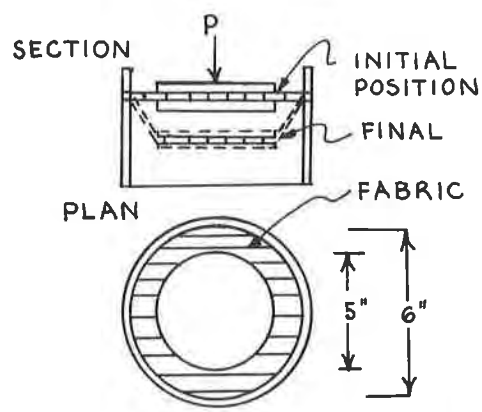

FIG. 1 OSU RING TEST

Figure 2-Fabric Width vs. Strength (nonwoven, needle punched)

The gradient ratio test (4) was performed with a number of fabric and soil types. The soils tested had very low permeabilities when placed in the test apparatus, resulting in extremely long testing periods. The test was judged impractical for routine evaluation of soil/fabric systems on a project basis.

Grab, cut strip 25.4 millimeter (1 inch) and weight per unit area tests were performed on many samples from 1974 through 1976. These tests documented the range of fabric strengths and weights available, and checked the values reported in sales literature. The results were reported for users in the guidelines report (2).

Seam strength tests were performed on samples from the Quinalt test road (6). Seams were sewed with a portable sewing machine using polyester thread. No problems were encountered with the seams in the field.

The E.O.S. test using glass beads, percent open area using the slide projection method, grab, cut strip 25.4 millimeter (1 inch), weight per unit area, thickness, and visual identification of filiment or fiber type have been performed to verify or check specification compliance. Other tests such as burst, puncture, and abrasion have not been performed in our laboratory.

FIELD EVALUATION OF MATERIALS

Field evaluations of materials have been made on several projects. Two of these projects; the Shelton retaining wall, and the Quinalt test road were instrumented to monitor performance.

The Shelton wall shown in Figure 3 was 53.7 meters (176 feet) long by 6 meters (19.5 feet) high. The Quinalt test road consisted of 1,434 meters (4,700 feet) of road containing 16 fabric types or weights of fabric. The test wall and test road segments were incorporated into scheduled projects for timber harvest. Both test projects were instrumented and sampled to monitor performance. Results of both test projects have been reported elsewhere (6, 9).

Samples of fabric for the Shelton wall were exposed at the site with and without treatment and with emulsified asphalt for weathering protection. Coupons were tested periodically for the first year to measure strength changes. Strength decrease was very rapid for the untreated, non ultraviolet stabilized polypropolene. Deterioration of the lightly ultraviolet stabilized polyester fabric was less rapid than for the polypropolene. Both fabrics require ultraviolet protection for long life. Emulsified asphalt appears to be an effective ultraviolet protection when sprayed on the surface of the fabric. The wall was recoated with emulsified asphalt in 1979 (4 years after construction) to ensure a continuous protective coating.

A 106.8 meter (35-foot) long by 30.5 meter (10-foot) high fabric wall was built on the Siskiyou National Forest in 1974. This wall, which preceded the Shelton wall, was built to confirm the laboratory model tests by Bell (7). Although the wall was not instrumented, it did confirm that a fabric wall could be constructed in the field. The face of this wall was treated with a gunite (sprayed concrete) for ultra violet light and vandal protection. The wall has performed very well.

Another fabric property of concern in reinforcement applications is fabric creep under long term loads. Polypropolene and polyester nonwoven and needle punched fabrics were both used in the Shelton test wall to determine if creep properties under field loadings were represented by laboratory creep tests. Laboratory creep tests (8) indicate polypropolene fabrics have as much as 5-10 times as much creep as polyester fabrics. Typically, laboratory tests are performed with nonlaterally restrained specimens as opposed to almost total restraint of fabrics in retaining walls. Retaining wall movements measured during the first 2 years after construction showed no difference in movement, stretching, or creep of the two materials. All movement of the wall face or stretching of the fabric occurred during the first 6 months after the wall was constructed (9). Neither fabric stretched or elongated as much as was predicted from laboratory load tests and internal fabric stress calculations. The wall was designed and stresses calculated using methods proposed by Bell (3).

Figure 3. Photograph of
The Shelton Fabric Wall

The fabric road test sections were built to answer a number of questions. Some of the questions were:

- What weight, strength, or type of fabric is best and most cost effective (manufacturers all claimed, "mine is the best fabric for your project")?

- How do we design, specify, and construct a project with fabric on the subgrade (again, manufacturers, university professors, engineers, and sales people all had different methods)?

- What are the economic considerations?

- What are the social considerations (acceptance of new ideas)?

A brief summary of the findings from the test road and other projects are·

- All of the fabrics used in the test road performed equally and satisfactorily. The fabrics were all nonwoven with weights between 136 and 420 g/m2 (4.0 and 12.4 oz./yd.2), grab strengths between 3.8 and 19.7 KN/m (21.6 and 112.5 lbs./in.), cut strip strengths 25.4 millimeter (1 inch) between 4 and 25.4 KN/m (22.6 and 145 lbs./in.), and fabric elongations of 40-200 percent.

- The Barenberg (10) method of thickness design using field cone penetrometer or vane shear strenghts is a reasonable method of establishing design depths of cover aggregate. Barenberg's method and design curves discussed later in this paper have been modified for our use.

Specifications based on the minimum properties of fabrics used in the test road provide good field performance, free competition between fabric suppliers, and permit contractors to install fabric with minimum inspection.

- Fabrics use on subgrades is usually economical for aggregate surface roads over low strength soils (CBR less than 2 or 3). Fabrics use on higher strength subgrades may be economical, but is very dependent on aggregate costs and performance expectations.

- The construction season can be extended into wetter seasons when fabrics are used on the subgrade.

- The weakest condition of the subgrade is during construction. The subgrade gets stronger with use due to soil consolidation. It may, therefore, be economical to remove and reuse a portion of the cover aggregate after the road has been used for a period of time.

- The thickness design method establishes a design thickness of cover aggregate. The most economical road for a project can be achieved by adjusting the cover thickness in the field based on rutting under construction traffic.

DESIGN METHODS

Subsurface Drainage: Drainage applications are designed using conventional groundwater seepage calculations as outlined in conventional text books (11). Fabric selection is based on the expected service condition and the particle size of the protected soil. The E.O.S. and percent open area of the fabric are determined using criteria originally proposed by Calhoun (1). Use of very high permeability aggregates are encouraged for the drainage layer. This design method is used because it works, it is simple enough to be understood and use in the field, it minimizes testing on hundreds of small projects each year, and the resulting design has a high chance of success.

Woven monofiliment fabrics are specified to ensure the fabrics will survive installation without tears, punctures, or deformation and that the actual fabric installed will function as designed. Nonwoven fabrics are not currently specified for drainage applications due to the wide variation in fabrics available (and liable to show up on a project), and the lack of a widely accepted and proven method of design and specification. Nonwoven fabrics are used by other agencies for drainage applications. To use nonwoven fabrics on our projects with our current capabilites would require extensive testing for specific projects to ensure that the selected or specified fabric would function. Few of our projects are large enough to warrant the extra cost of this work. We are looking to other users and researchers to do the testing and development work in this area.

Geotextile Reinforced Retaining Walls: Fabric retaining wall construction is shown in Figure 4. Laboratory model tests (3) and field experience indicate (7) the walls can be designed with reasonable confidence using the active Rankine case.

A very important consideration in fabric walls is defining the appropriate fabric properties. Although no creep or long term deformation of the fabric test wall has been observed (9), creep and fatique properties of fabrics continues to be a concern. Current evaluations of fabrics for wall reinforcement follow recommendations by Bell (12): Test wide strip tensile specimen after soaking, 203 millimeter (8-inch) width, 50.8-101.6 millimeter (2-4 inch) jaw spacing, 10% per minute. The design strength is taken as the lesser of the strength at 10% strain or 40% (polypropolene) or 60% (polyester) of the ultimate strength.

Soil-geotextile friction is taken as 2/3 ϕ for granular material on both sides of the fabric. A minimum factor of safety of 1.5 is used, with higher values for less certain loadings.

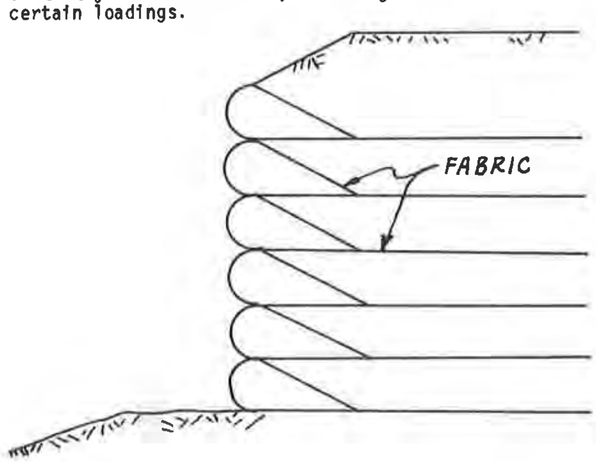

FIGURE 4 - FABRIC RETAINING WALL

Subgrade Restraint: The method outlined in the guideline report (2) has proven adequate for design of geotextile/aggregate structural sections over low strength soils (CBR less than 2 or 3). The basic procedure recommended by Barenberg (10) uses the undrained shear strength of the subgrade soil and the bearing capacity formula used in foundation design:

$$q = CN_c$$

where, q = the stress level in the subgrade
C = undrained shear strength of the subgrade soil
Nc = a bearing capacity factor

The bearing capacity factors normally used are 2.8 without a geotextile and 5.0 with a geotextile when a large number of axles (probably greater than 1,000 80.1 kn or 18 kip axles) and very little rutting (less than 50.8-76.2 millimeters 2 or 3 inches) is expected. These bearing capacity factors may be increased to 3.3 and 6.0 (about a 15% increase) when a small number of axles (probably less than 100) and greater rutting (greater than 76.2 millimeters or 3 inches) is expected. The amount of cover aggregate required is determined from design curves for dual tandem wheel loadings derived from Boussinesq's equation for vertical pressure under a circular load applied at the ground surface. Figure 5 is an example of one of the design curves.

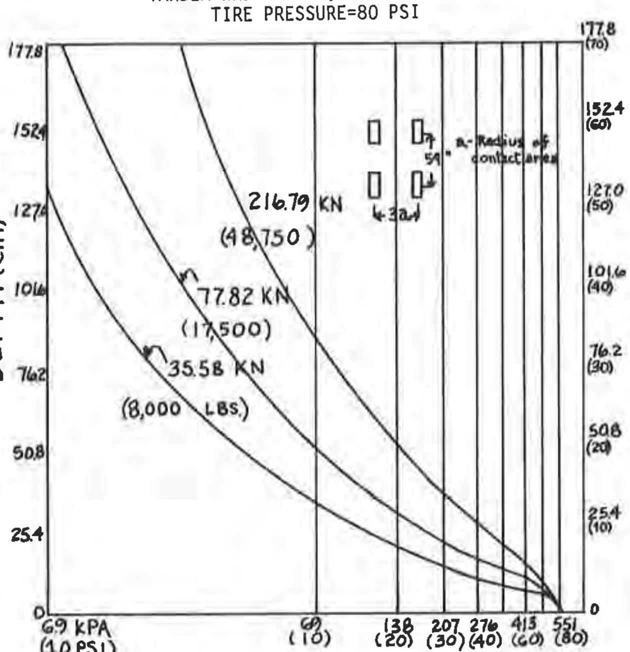

FIGURE 5

The undrained shear strength of the soil is measured in the field using a cone penetrometer or vane shear. Multiple measurements are recommended at several representative locations. Due to wide variability in the field measurements within a small area, the strength value at the 75th percentile (75% of the strength readings are higher than this value) is used for design. Theoretically, 25 percent of the road area will be underdesigned.

Where possible, the actual thickness should be adjusted during construction to obtain adequate performance under construction haul traffic. The design thickness using the bearing capacity method identifies a starting point. Ajust the thickness in the field until a 50.8-76.2 millimeter (2-3 inch) rut occurs under a few axle repetitions (probably 5-10). Increasing the thickness 15-20% over the depth should yield an adequate road structural section. This method works because the worst condition is usually during construction before the subgrade consolidates and increases in strength.

IMPLEMENTATION

A large amount of research has been done on subgrade soil/fabric/aggregate interaction to explain what happens and how to design the most cost effective road. The challenge for the Forest Service and other users is to select and utilize those methods and materials that can be used effectively in the field with a high chance of success under limited guidance. Methods that require extensive soil sampling and testing, highly technical designs, tight specification, and intensive construction inspection and supervision are not compatible with our low-volume road construction program.

New ideas, methods, and materials are constantly being explored, evaluated, and tried. Some evaluations are through formal evaluation programs, but most are in response to solving a particular problem. If the problem solution shows promise of wide application and savings within our skill capabilities, we will evaluate it for general use. If general use is feasible, we will modify our specifications and procedures to include the method or material.

Several specifications (13) have been developed and implemented for the use of geotextiles. The degree of control contained in the specification for geotextile properties and installation procedures depends on several factors. The key factors are design procedure, chances of success, consequence of failure, costs, and construction and inspection capabilities.

The most controversial specification is Standard Specification 720 - Woven Plastic Filter Cloth (13). This specification is used for subsurface drainage applications. The specification requires that the cloth be a pervious sheet of plastic filament woven into a uniform pattern with distinct and measureable openings. The material properties are similar to those specified by the U.S. Army Corps of Engineers in 1976 (4), with a 50 percent strength reduction for nonabrasive conditions. The woven filament fabrics specified are compatible with the design methods used and apply to a wide range of soil conditions.

These fabrics are easily recognized and have strength to survive severe construction conditions. Although the nonwoven fabrics normally cost less per unit area than woven filament fabrics, the compatibility with the design criteria, the ease of recognition, and the strength properties of the woven filament fabrics lead to a higher expectation of survivability of the woven filament system. This is particularly the case for the many small installations in a wide range of soil and construction conditions encountered in the National Forests.

Although the woven filament fabrics are easily recognizable, we have had difficulty preventing the use of slit film or ribbon fabrics. The slit film or ribbon fabrics, which are widely used on subgrades, generally have very low percent open areas and irregular openings which result in poor drainage capabilities. Several drainage installations have been made using the wrong fabric (usually slit film or ribbon) resulting in poor performance or, more usually, extra cost to the contractor or fabric supplier to correct the error. To our knowledge, the proper mill certificate or affidavit had not been supplied by the contractor prior to installation of the nonspecification fabrics.

All of our current specifications require a mill certificate or affidavit from the supplier stating that the geotextile complies with the physical and chemical requirements of the specification. Certificates for drainage fabrics, subgrade restraint, and separation fabrics can be based on previous testing and quality control testing by the supplier. Certificates for fabrics used for retaining wall reinforcement must be based on tests performed on the lot of fabric actually shipped to the site. Certificates are required prior to installation.

The grab and strip 25.4 millimeter (1 inch) test methods are usually specified since they are routine tests performed by most manufacturers. The acceptable test result is adjusted for each specification. The test requirements for subgrade treatments (woven, slit film, or nonwoven fabrics) are:

Weight: 136 g/m2 (4.0 oz./yd.2) minimum
Thickness: 0.38 mm (15 mils), minimum
Width: 3.66 meters (12 feet), minimum
Grab Tensile Test: 21 KN/m (120 lbs./in.), minimum (ASTM D-1682)
Elongated at Failure: 25% minimum
E.O.S.: Smaller than the #70 sieve (E.O.S. requirement waived for subgrade restraint)

The subgrade treatment specification is based on the minimum properties of the fabrics successfully used on the Quinault test road sections and modified to permit slit film and ribbon fabrics. Slit film and ribbon fabrics have been used successfully on many subgrade projects both inside and outside the Forest Service. The minimum elongation at failure will probably be reduced to 20 percent minimum with the next specification revision.

The subgrade treatment specification has worked very well allowing a wide range of manufacturers to compete. Unlike drainage and retaining wall reinforcement applications, difficulties or failures of the fabric in the subgrade applications usually show up during construction and are easily corrected.

The drainage and retaining wall reinforcement specifications are more restrictive since problems with the fabrics or the installation probably will not be recognized until months or years after construction. Any problems that are encountered in drainage or retaining walls may involve the entire road section or structure, making costs for correction very high.

FUTURE APPLICATIONS

A major challenge to everyone in the geotextile field, whether user, manufacturer, or researcher is to interpret the great volume of information being developed to produce a cost effective end use. Standard writing groups such as ASTM and AASHTO have a difficult task in developing performance related standards to aid in cost efficient use of geotextiles. The challenges and tasks are worth working on when we consider the cost saving potential in the use of geotextiles.
The potential savings using fabrics in Region 6 of the Forest Service was estimated to be 4-11 million dollars per year in 1977 (2). Geotextile usage has increased greatly since 1977, but due to inflation and improved applications, and knowledge, the potential savings is probably still in the 4-11 million dollar range.

A key factor in actually realizing the potential savings is developing enough successful experience for management, design, and construction personnel to accept and recommend geotextile applications. Another key factor is having credible technical information and specifications in understandable and usable formats. The future will see more geotextile options as fabrics are manufactured and accepted for specific applications. The third key factor is keeping up with and implementing improvements without excessive failed trial uses. The project designers and constructors must be trained and kept up-to-date on geotextile uses.

The increased knowledge about geotextiles and confidence in their successful use has recently resulted in some very cost effective solutions to difficult problems. Fabric reinforced retaining walls were built across two unstable areas using lightweight wood fiber backfill. The lightweight backfill reduced slope loading by up to 1/2, and the fabric reinforcement permitted large wall deformations without structural damage. On another project, discovery of endangered wildlife required relocation of a road away from a nesting area and limiting activity during the nesting season. The result was a requirement that the road subgrade be completed and base rock placed during wet weather. A geotextile was successfully used on the subgrade as a restraint layer for weaker areas and as a separation layer to prevent the shallow layer of mud on the surface from contaminating the aggregate.

More of these "unique uses" will be developed in the future, limited only by our willingness, and patience to try something new and learn from both successes and failures.

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Mechanism of Geotextile-Aggregate Support in Low-Cost Roads

Mécanisme de support géotextile-agréggats pour des routes

The mechanism of failure of aggregate surfaced roads on very weak subgrades was investigated by (1) study of selected road failures, (2) small scale load tests, (3) large scale static load tests simulating wheel loads and (4) full scale, moving vehicle loading. The geotextile provides tensile restraint for the aggregate, which enhances the load spreading to the subgrade. This reduces elastic deflection with a light load and increases the load causing failure after one or two load repetitions. The geotextile separates the porous aggregate and the underlying soft subgrade, preventing soil intrusion that destroys the aggregate load spreading upon repeated loading. After repeated heavy loading plus rut repair, the geotextile sags below the wheel path and bulges adjacent to it. Load support is more than doubled by catenary support below the wheel, catenary restraint of the subgrade bulge, and by enhancing the aggregate load spreading.

Le mécanisme responsable pour l'écrasement des routes ă surface couverte de granulats ou graviers sur faibles sous-sols a été examiné par 1) étude de certains écrasements de routes 2) essais avec petites charges 3) essais avec lourdes charges statiques 4) et échelle complète de chargements produits par voiture mobile. Le géotextile impose une réstriction sur la resistance de tension des aggrégats, ce qui accroît la diversion du chargement jusqu au sous-sol. Ceci réduit la déflection élastique due ă une ou deux répétitions de chargement. Le géotextile sépare l'aggregat poreux du faible sous-sol sous-jacent empêchant l'intrusion du sol qui détruit la propagation du chargement exercée sur l'aggrégat. Avec de lourds chargements répétés, des réparations d'ornières, le géotextile ploie sous le parcours de la roue et forme des bombements de chaque côté. La résistance ă un chargement est plus que doublée par un support caténaire en dessous de la roue, une restriction caténaire du bombement du sous-sol et par l'augmentation de la répartition d'un chargement appliqué sur l'aggrégat.

INTRODUCTION

Although geotextiles have had increasing use for road support for nearly two decades, the designs are still largely based on emperical rules (1), design charts without technical documentation, (2) and extension from flexible pavement design (3)(4). Giroud and Noiray (5) present a rational method for design based on unpublished research in France and limited studies at the U.S. Army Engineer Waterways Experiment Station.

The authors, as principal engineers for the Law Engineering Testing Co., undertook a comprehensive study of geotextile contribution to low-cost road capacity under a contract with the Monsanto Company in 1976. Although the work was completed in 1977, a contractural agreement of non-disclosure has prevented a written presentation of the results of that work until now.

OBJECTIVES OF INVESTIGATION

The investigation had two objectives: (1) Determine the mechanism by which geotextiles contribute to load support for gravel or aggregate roads on weak subgrades and (2) develop analytical expressions for wheel load support to be used in design. The investigation was limited to soft clayey subgrades and roads of cohesionless materials: natural gravel and crushed stone, termed aggregates. A range of geotextiles was investigated. The emphasis was on Bidim, a needle punched fabric in several weights marketed by Monsanto in the United States. In addition, two widely used spun-bonded materials, Typar and Mirafi, were tested for comparison and to help verify the general applicability of the work.

EXAMINATION OF TYPICAL INSTALLATIONS

The authors and members of the Monsanto Company staff examined a number of failing roads as well as roads incorporating fabric at selected sites in the continental USA. Those selected were predominately inorganic and clayey subgrades. Most were temporary logging roads. Traffic was typically infrequent, a few vehicles per day, but the loads were heavy. The soil strengths were variable but poor. The roads were poorly drained.

These sites were examined while vehicles were operating to observe deflection under moving wheel loads, surface deterioration under successive wheel loads, and evidence of distress such as pumping and rutting. At some sites it was possible to exhume the geotextile. Its physical condition was noted and its deflection from a plane measured. The strength of the subgrade was measured, either by vane shear or by a static cone penetrometer developed for this project.

GEOTEXTILE ENGINEERING PROPERTIES

A lack of data on the engineering properties related to pavement function has been a major obstacle to understanding the role of geotextiles in pavement performance. Most published data are in terms of standard textile tests: strain and strength of narrow strips subject to short-term tensile loading, elongation under a constant tensile load (usually 40% of the failure load), bursting as a diaphram under pressure, and tear resistance. Non-textile tests include puncture of the strechted fabric with cylinders ranging from 7.9mm (5/16 in.) to 127mm (5 in.) in diameter.

Two new tests were developed to more nearly simulate the stretching of the fabrics observed in the existing installations. The first was a triaxial loading in which the geotextile was stretched in 3 dimensions spherically, by an inflated rubber membrane similar to half a balloon. Subsequently, the Monsanto Company developed a modified balloon test with a rectangular specimen expanding in a half-cylinder. They conducted a comprehensive testing program, under the direction of G. Raumann, the Monsanto technologist who served as laision advisor to the authors.(9)

MODEL TESTS

Small scale tests were made in the laboratory to model the subgrade - geotextile - aggregate interaction under circular plates. The test apparatus included a plexiglass box with a volume of approximately 0.03 m^3 (1 ft^3) and loaded plates with diameters of 5 to 7.5 cm (2 to 3 inches). The model did display deflections and failure that qualitatively resembled those seen in the field installations that were inspected. However, the comparative deflections and failure loads with and without the geotextile were greatly different from the comparative loads and deflections observed in the field.

On the basis of these results further model tests to quantify failure and support mechanisms were stopped. All subsequent experimental work was confined to full-scale tests.

FULL SCALE TEST FACILITY

In order to perform tests on full-scale sections of aggregate roads under controlled conditions a special test facility was constructed. It consisted of a 5.5m (18 ft) by 9.1m (30 ft) by 1.2m (4 ft) deep reinforced concrete pit spanned by a movable truss and load system capable of exerting vertical loads up to 130 KN (30,000 lb). The pit was filled with very soft clay. The geotextile was placed at ground level. The aggregate road surfacing was placed above the ground level with sloping approaches to allow loaded trucks to traverse the pit lengthwise in one lane. The materials used are described in Table 1.

Construction of a typical test section began by remixing the clay subgrade to achieve uniformity. The geotextile, marked with a reference grid to allow measurement of permanent strain at the end of testing, was then positioned over the leveled subgrade. Numerous test sequences were performed with the thickness of aggregate varying from 305 to 610 mm (12" to 24"). Control test sections with aggregate surfacing but without the fabric layer were similarly constructed and tested.

FULL SCALE STATIC TESTS

Plate load tests were made on the various test sections. They involved applying a vertical static load to circular and rectangular plates of various sizes simulating tire contact areas. Most tests employed a dual plate, sized to match the print of a dual wheel assembly of 10.00 - 20 truck tires.

One purpose of the load tests was to obtain comparative data on load-deflection, repeated load-deflection and failure load with and without geotextile support for a range of aggregate thickness. A second purpose was to observe the mechanical behavior of the aggregate and the geotextile at different stages of loading, including failure.

TABLE 1. MATERIALS FOR FULL SCALE TEST SECTIONS

Aggregates:
1) Crushed Stone
Maximum Particle Size: 44.4 mm (1-3/4 inches)
Gradation: 40-50% Gravel Size
40-50% Sand Sizes
Less Than 10% Fines (-0.075mm)
Inplace Density: 85-90% Maximum Dry Density
(Modified Proctor Compaction)
2) Washed River Gravel
Maximum Particle Size: 52mm (2 inches)
Gradation: 45% Gravel Sizes
55% Sand Sizes
Inplace Density: 80% Maximum Dry Density
(Modified Proctor Compaction)

Fabrics:
Continuous Filament, Non-Woven, Needle Punched
Polyester (Bidim)
.15 kg/m^2 (4.4 oz/yd^2)
.27 kg/m^2 (8.0 oz/yd^2)
Spun Bonded Fabrics
Mirafi: .14 kg/m^2 (4.1 oz/yd^2)
Typar: 0.14 kg/m^2 (4.0 oz/yd^2)

Subgrade:
Soft Saturated Clay (Commercial bentonite)
Shear Strength (undrained):
Cohesion (c) = 3.4 KPa (0.5 psi)
Internal Friction (∅) = 0
Moisture Content (w) = 450%

Typical test results for repeated loading with geotextile is shown in Fig. 1. In the first sequence the load was increased to impending failure and the rut or depression repaired. This was followed by 50 cycles of load-unload at 80 percent of the failure load. The load was then increased until failure was again impending. The curve shows that the failure load increased by 15 percent. The repair, cyclic loading at less than failure and reloading was repeated. The failure load increased by 29 percent of the original failure. The results were similar for the four different geotextiles tested. Without the geotextile, repeated loading following initial failure lead to rapidly increasing deflection and loss of support at loads less than the initial failure load.

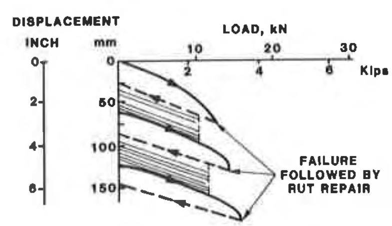

Fig. 1 Typical plate load test results with geotextile.

The subgrade surface (and geotextile when present) was exhumed in all test sequences: sometimes after repeated loads smaller than failure, sometimes immediately after initial failure and in most cases after sequences of repeated loading.

The load tests with the three dimensional stress spread and subgrade deflection do not simulate moving wheel loads and their two dimensional rutting. However, the results served as a guide to the mechanisms of support and the effects of aggregate type, thickness and geotextile in the behavior of the same systems under moving wheels.

TRAFFIC TESTING

Traffic loading was simulated by trucks with variable loading. Most of the testing involved two different load sequences on aggregate surfaces, with geotextiles. The incremental load sequence involved two passes of the wheels, measurement of rut depth and any distress, followed by two more passes at increased wheel loads. The loads were increased until the rate of rut depth increase per sequence greatly exceeded the rate of load increase. In the cyclic loading sequences, the truck load was maintained constant for large numbers of passes: rut depths were measured at intervals. More than 100 passes were included in some sequences, with loads ranging from 1140 to 3640 kg (2500 to 8000 lb) per dual wheel. In some sequences the ruts were repaired by relevelling the aggregate when the rut depth exceeded 4 in. After each test sequence the subgrade or geotextile surface was exposed to determine the displacement. In some of the tests, the progressive deflection was monitored by induction cells; however, the results became difficult to interpret when the displacements increased.

The effect of the geotextile on deflection is shown in Fig. 2. At light loads (about half the load for initial failure) repeated loading produces rapidly increasing deflection and ultimately failure without the geotextile. With the geotextile there is increasing deflection but no failure. At heavy loads (approximately failure) without the geotextile there is rapidly increasing deflection and usually punching of the wheel into the aggregate and subgrade with 2 or 3 passes. With the geotextile there is increasing deflection and rutting but no punching or destruction of the aggregate surface.

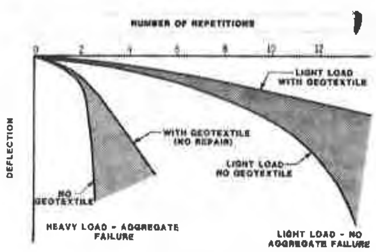

Fig. 2 Typical load repetition - deflection curves comparing aggregate subgrade and aggregate-geotextile subgrade support.

The effect of rut repair with heavy loads on aggregate surfaces underlain by geotextiles is illustrated in Fig. 3. Repair re-establishes the continuity of the aggregate and increases its thickness beneath the wheel. The effect of repair after the initial rutting (from 2 or 3 passes) is to reduce the rate of deflection. After a second repair, deflection develops at a much slower rate.

Removal of the aggregate discloses large permanent deformation of the fabric-subgrade interface, as illustrated in Fig. 4. There is a sag trough below the wheel path and bulges on both sides. Because the soft clay subgrade is incompressible during short term loading, the sag volume equals the bulge volumes (to be discussed later). In a few cases of repeated, very heavy loading there was rupture of the fabric. This was infrequent. In most cases an equilibrium sag was reached with the geotextile stretching across the wheel path but not parallel to it.

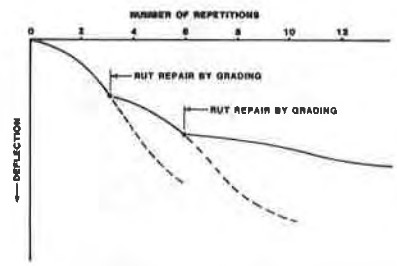

Fig. 3 Typical load repetitions deflection curves showing effect of rut repair by regrading.

Fig. 4 Catenary sag and bulges of geotextile beneath rutted aggregate surface.

The plate load tests reproduced the traffic loading qualitatively. However, the moving wheel loads produced greater deflection and precipitated system failure at smaller loads than the plate load tests. Conversely, the effect of rut repair was more pronounced in the traffic tests than in the static load tests.

SUPPORT MECHANISM WITHOUT GEOTEXTILE

The mechanism of load support without a geotextile was studied to provide insight into how the geotextile enhances support. Distress and failure in the tests were seen to be controlled by wheel load and number of repetitions, by the aggregate surface thickness, and the strength of the subgrade. The progress of failure is illustrated in Fig. 5.

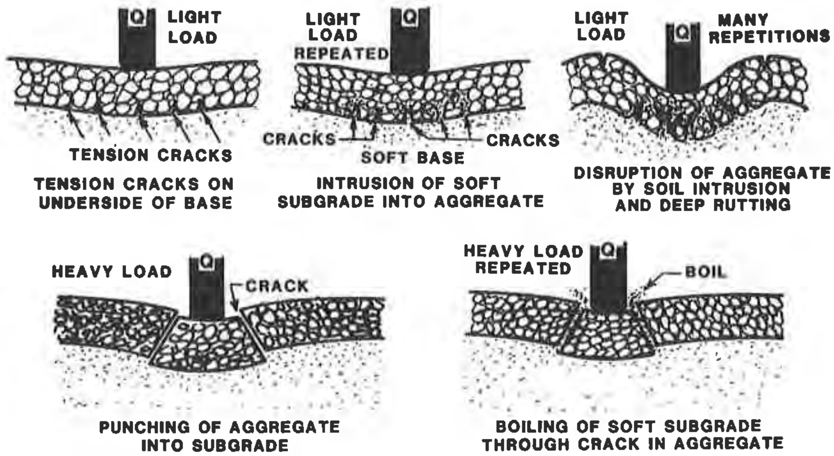

Fig. 5 Destruction of aggregate surface by soft subgrade intrusion and punching.

At light loads the aggregate deflects downward, spreading the load of the wheels on an area of subgrade much wider than the tire print. The authors approximate this as is sometimes done in foundation analysis (6) by assuming the wheel is supported on a truncated pyramid (or cone), Fig. 6, and is wider than that which approximates the Boussinesq stress distribution.

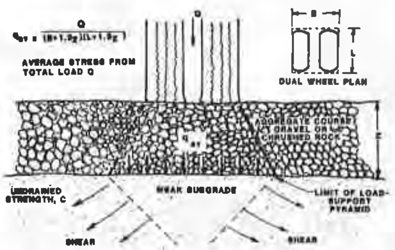

Fig. 6 Average stress on soft subgrade from wheel load.

The aggregate cannot withstand tension; it cracks on its lower surface below the wheel and ultimately on the upper surface at some distance from the wheel. With repeated loading the soft subgrade is extruded upward into the cracks and the larger pores. The aggregate particles no longer make point-to-point contact -- they float in the soft subgrade. The aggregate continuity, strength, and load spreading are reduced. The pavement eventually fails, by total disruption of the aggregate accompanied by subgrade shear.

With a very heavy wheel load, the aggregate pyramid punches into the subgrade. The pyramid shape is well defined by cracks. The loads required for failure can be computed by the methods used for foundation bearing capacity. With reloading after a failure, the soft subgrade squeezes or boils upward along the pyramid sides, entering the larger voids and sometimes coming to the surface. The load bearing approaches that of the subgrade alone.

EFFECT OF THE GEOTEXTILE

The large scale plate load tests and moving wheel loads demonstrate that the geotextile alters the system behavior. At light loads the geotextile offers restraint to the deflecting aggregate, enhancing its load spreading and reducing the deflection. This confirms observations by other investigators (7)(8). More important, the geotextile prevents the intrusion of the soft subgrade into the cracks and voids of the aggregate. Thus the load spreading ability of the aggregate is maintained. At heavy loads the geotextile tension continues to help maintain the load spreading of the aggregate. The wheel load bearing capacity is increased about 10 percent. Moreover, the geotextile continues separating subgrade and aggregate.

With failure and surface rutting at heavy loads, the geotextile is forced downward in a sag curve, similar to a catenary, as shown in Fig. 4. The sag is accompanied by bulges adjacent to the wheel path. Bulging was not noted in field installations in very fibrous organic soils but occurred in highly organic clays. When the rutting is filled in before the aggregate continuity is destroyed, the load carrying capacity is increased dramatically. The pyramid of load spreading becomes wider and the catenary tension in the geotextile contributes to both direct load support and bulge restraint.

LOAD SUPPORT WITH GEOTEXTILE

The following equations form a basis for designing field installations with the fabric-tensioning rut repair concept. Certain simplifying assumptions are required. However, calculated truck load-deflection curves are in reasonable agreement with our data.

The deflections in the aggregate, geotextile and subgrade during truck loading are modeled in Fig. 7. The observations of plate load test data and truck loading concluded that with fabric the subgrade could carry a uniform pressure of:

$$p = 5.14\ (S_u) \tag{1}$$

Where S_u is the subgrade undrained shear strength, and the factor of 5.14 is the common bearing capacity factor for cohesive soils. This uniform subgrade pressure is acting over the base of a stress-pyramid with dimensions B' x L.

Additional load carrying capacity is provided by the geotextile. The shape of the deformed geotextile is assumed as a parabola, and the geotextile would then provide a uniform uplift over the width B'. This uplift pressure is calculated as:

$$p' = \frac{8T\Delta}{B'^2} \qquad (2)$$

p' = uplift pressure
T = minimum tension in geotextile = tension in geotextile under the center of loaded area.

The effective geotextile deformed width B', deflection Δ, and tension T are all related to Δ', the permanent surface rutting, as described in subsequent paragraphs. The permanent surface rutting, Δ', is easily measured in field installations and is the basis for checking predicted and observed performance.

The fabric is deformed over a width which approximates the base width of the stress pyramid shown in Fig. 6. An analysis of the strains during truck loading indicates that the effective deformed geotextile width at small deflections is less than the B + 1.3z shown in Fig. 6, and is only slightly larger than the tire print width B. As the surface deflections increase, the effective deformed width B' of the geotextile increases and surpasses the dimension B + 1.3z. The following equation and table relates the observed width of the deformed geotextile to surface deflection.

$$B' = n\,(B + 1.3\,z) \qquad (3)$$

TABLE 2

Δ'/B	.05	.10	.15	.20	.25	.30	.35	.40
n	.70	.80	.90	.96	1.00	1.04	1.08	1.12

The geotextile deflection under load is greater than the permanent deflection measured after load removal by an amount equal to the subgrade and geotextile rebound. Deflections in the field will be measured after truck loading. Load carrying capacity is related to deflection under load, so rebound must be estimated. A rebound of one inch has been used, based on observations. The relationship between permanent surface deflection, Δ' and geotextile deflection under load, Δ, was calculated by balancing volumes of displaced material as shown in Fig. 7. The average rut width is about 1.2B, and the rebound is δ, and yields:

$$\Delta = \frac{1.8B\,(\Delta'+\delta)}{B'} \qquad (4)$$

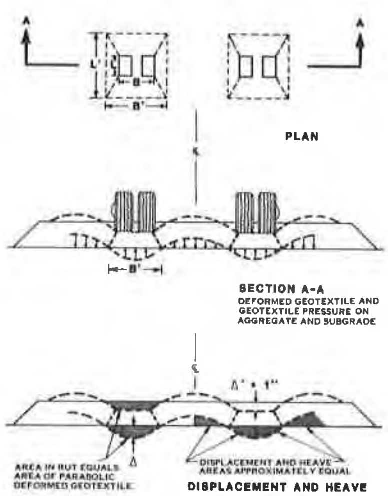

Fig. 7 Displacement, heave, enlarged base pyramid, catenary support and catenary restraint.

The tension in the fabric is strain dependent. The strains are computed as:

$$\varepsilon = \frac{\left[\left(1+\left(\frac{4\Delta}{B'}\right)^2\right)^{\frac{1}{2}} + \left(\frac{4\Delta}{B'}\right)^{-1} \text{ARCSINH}\left(\frac{4\Delta}{B'}\right)\right]}{2} - 1 \qquad (5)$$

The calculated value of strain is used with particular geotextile stress-strain curves to compute tension in the geotextile.

The strain and corresponding tension are nonuniform. The above strain is assumed as an average for which an average tension T_{avg} is computed. The ratio of maximum Tension (T_{max}) to minimum Tension (T_{min}) is expressed as:

$$\frac{T_{max}}{T_{min}} = \left[1 + \left(\frac{4\Delta}{B'}\right)^2\right]^{\frac{1}{2}} \qquad (6)$$

from which the ratio of T_{min}/T_{avg} can be computed, as shown in the following Table 3.

TABLE 3

B'	Tmax/Tmin	Tavg/Tmin	Tmin/Tavg	Tmax/Tavg
.05	1.02	1.01	.99	1.01
.10	1.08	1.04	.96	1.04
.15	1.17	1.08	.92	1.08
.20	1.28	1.14	.88	1.12
.25	1.41	1.21	.83	1.17
.30	1.56	1.28	.78	1.22
.35	1.72	1.36	.73	1.26
.40	1.88	1.44	.70	1.31
.45	2.06	1.53	.65	1.35

This ratio is applied to T_{avg} to yield T_{min}, and used as T in Eq. 2.

The geotextile stress-strain curves used in this study were produced from the results of two dimensional balloon tests [9]. The test produces a circular arc deformation of the fabric, with uniform strains. The data from actual truck loading, however , indicates strain increases from the center of the loaded area outward. This corresponds to strains produced by a parabolic deflected shape.

The geotextile uplift pressure calculated by Eq. 2 provides an additional force on the base of the stress pyramid and increases truck loading capacity. The capacity is further increased in an indirect manner by the geotextile restraining effect on the subgrade bulge outside of the stress pyramid. The geotextile provides a surcharge pressure to the subgrade, equal to the uplift pressure calculated by Eq. 2. This surcharge allows the subgrade itself to support an additional load greater than would be calculated by Eq. 1. Although the "surcharge" increases truck loading capacity through interaction with the subgrade, it is directly fabric dependent. Thus, the increased loading capacity due to this effect has been combined with the directly applied fabric uplift pressure. The net result is that the increased pressure on the stress pyramid, due to the geotextile, is twice that shown in Eq. 2. The total pressure on the base of the stress pyramid is therefore:

$$P_{Total} = 5.14\,(S_u) + \frac{16T\Delta}{B'^2} \qquad (7)$$

This pressure acts over a base dimension of B' x L'.

This procedure allows determining of the fabric strains that will occur with a specified axle load. The procedure is as follows:

1) For an assumed rut depth and aggregate thickness, the deformed width B' is determined from Table 2 and Eq. 3.
2) The geotextile deflection is determined from Eq. 4.
3) The average geotextile strain is determined from Eq. 5, and is used with appropriate stress-strain curves to determine the geotextile tension.
4) Table 3 is used to determine T_{min} and T_{max}/T_{avg}. The ratio T_{max}/T_{avg} is multipled by the stress determined from Step 3 to evaluate the maximum geotextile stress (strain). This should be smaller than the failure stress (strain) by an appropriate factor of safety.
5) T_{min} is used in Eq. 7, along with Δ, B', L', and S_u to determine the load capacity.
6) The process is repeated to determine the load/rutting characteristic of a particular design.

Fig. 8 is an example of the above procedures compared to actual truck load testing. Each data point represents the first time the load reached a new maximum, and was preceded by several smaller load repetitions. The effect of these smaller load repetitions has not been accounted for in this derivation, but is minor.

The comparison between predicted rut development and truck loading shown in Fig. 8 is reasonable, and demonstrates the validity of this procedure. The calculated strains at the end of truck loading vary from 14% at the center of the catenary support, to 25% at the inflection point, and agree with the measured geotextile strains. Fig. 8 also demonstrates the large contribution made by the geotextile when used with very soft subgrades. For stronger subgrades the relative contribution of the geotextile decreases.

Additional evaluations of load versus rut development, similiar to Fig. 8, have been made for two other non-woven fabrics (Typar and Mirafi). The general agreement between predictions with this theory and measured performance is good. However, for these other geotextiles the predicted performance was conservative. Extensions to woven geotextiles, with their generally higher modulus stress-strain behavior, is cautioned until further testing is performed. For fibrous organic subgrades that compress under load, the bulge-restraint effect should probably not be included.

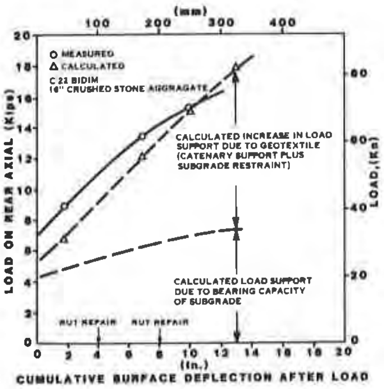

Fig. 8 Comparison of computed and observed cumulative deflection with truck loading.

The geotextile tensioning, rut development, rut repair procedure described in this paper has one major drawback: it dictates that truck loading must remain centered over the catenary support. Trucks cannot wander over the road and maintain the required support. This is not a serious drawback in low volume access roads, where drivers would tend to remain in the ruts anyway.

ACKNOWLEDGEMENTS

The field studies, small scale and full-scale experimental work on which this paper are based were conducted by the Law Engineering Testing Co. in 1976 and 1977, under contract with the Monsanto Company. The authors directed, conducted and analyzed the results which were presented in confidential progress reports and in a final report of November 8, 1977. Mr. Bill Hamilton of Monsanto was Monsanto's representative for investigation of field installations. Ms. Trudy Raumann was Monsanto's laision for the full scale experimental work, providing suggestions for testing and helping interpret fabric behavior. Monsanto, under the direction of Ms. Raumann extended the fabric testing procedures conceived by the authors and provided the engineering data on the fabrics tested. The analyses of the mechanisms of fabric support were undertaken during the Monsanto contract and presented in the November 7, 1977 report. They have been refined by the authors, based on theoretical studies made since then.

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The Strengthening Effect of Geotextiles on Soil—Geotextile—Aggregate Systems

L'effet de renforcement des géotextiles sur des systèmes sol-géotextile-agrégat

A two-dimensional model of a geotextile-reinforced unpaved road was constructed and used to evaluate the strengthening effect of the geotextile on the system. The laboratory test equipment, procedures, and results are described in previous reports. This report outlines the parameters needed for evaluation of the structural reinforcing effects of the geotextile in both laboratory tests and field installations. A technique for calculating the tension in the geotextile at any point within the profile is presented along with empirical factors for estimating the shape of the deformed surface at the geotextile.

Un modèle à deux dimensions d'une route non revêtue et renforcée d'un géotextile a été construit et utilisé pour évaluer l'effet de renforcement du géotextile sur le système. La description des matériaux, les procédures et les résultats de l'essai de laboratoire se trouvent dans des rapports précédents. Cet article présente des paramètres intervenant dans l'évaluation des effets de renforcement du géotextile dans les essais de laboratoire et sur le terrain. Une technique pour calculer la traction du géotextile en tous points dans la section est présentée avec des facteurs empiriques pour estimer la forme de la surface déformée au droit du géotextile.

1. INTRODUCTION

A two-dimensional laboratory model of an unpaved road was constructed and used to evaluate the strengthening effects of a geotextile in a soil-geotextile-aggregate (SGA) system when rutting occurs. The test apparatus consisted of a lucite box 1.5 m long, 0.15 m wide and 0.92 m deep. The bottom 0.59 m of the box was filled with soft clay. A geotextile was placed over the clay and covered with 76 to 229 mm of aggregate ranging in size from 6 to 25 mm. A 0.5 second duration load was applied to a 100 by 150 mm flat plate on the surface of the aggregate. Displacement measurements were made photographically through the side of the lucite box on markers placed throughout the system. A complete description of the test configuration, procedures, and results are given by Kinney (1979), and Kinney and Barenberg (1980).

A procedure was developed for analysis of the test data and projection of that data to field conditions. The general model was quite comprehensive and included the following:

* The normal stress on the subgrade induced by the goetextile.
* The shear stresses induced by the geotextile.
* The strain energy stored in the geotextile.

Using the normal stress and strain energy portions of the model, it was possible to accurately predict the behavior of the laboratory tests (Kinney 1979). When only the normal stress portion of the model was used to predict the performance of field tests, the model underestimated the beneficial effects of the geotextile (Kinney and Barenberg 1979).

This report presents the model and describes in detail some of the parameters needed in the model, including a technique for calculating the tension in the geotextile and methods of estimating the system geometry.

2. GEOTEXTILE TENSION MODEL (GTM) CONCEPT

The GTM is a mechanistic model of the behavior of the geotextile in a soil-geotextile-aggregate (SGA) system. It includes methods to determine the geotextile-induced shear and normal stresses, and the strain energy stored in the geotextile.

The normal stress created by the geotextile being stretched over a curved surface increases the stability of the system by decreasing the normal stress on the subgrade directly under the load, and by applying a downward stress on the heaved upward portions of the subgrade on each side of the rut. The net effect is to spread out the load on the subgrade, which mobilizes the resisting shear stress over a larger volume of the subgrade, Kinney and Barenberg (1980).

The shear stress on the subgrade caused by the geotextile decreases the strain naturally developed in the subgrade under the load and therefore increases the stability of the system. The shear stress on the aggregate caused by the geotextile increases the confining pressure in the aggregate, thus allowing it to distribute the load on the subgrade more effectively. It also decreases lateral spreading and subsequent rutting in the aggregate itself.

The strain energy stored in the geotextile acts in the same fashion as the strain energy stored in the soil upon loading. The elastic portion of this energy is released during unloading to cause rebound. If an SGA system undergoes rutting, then a portion of the strain energy in the soil is elastic and a portion is plastic. The strain energy stored in the geotextile reduces the strain energy in the soil, thereby reducing the plastic strains and rutting (Kinney 1979).

The geotextile-induced shear and normal stresses can be calculated if the tension in the geotextile and the deformed shape of the geotextile are both known. The sum of the shear stresses on the two sides of the geotextile is equal to the rate of change in tension per unit width of the geotextile along its length. The distribution of shear stresses between the top and the bottom of the geotextile is not as easily determined unless the geotextile has slipped with respect to one or both of the materials. If slippage has occurred, the shear stress on that side is equal to the maximum available between the geotextile and the adjoining material.

The geotextile-induced normal stress is equal to the tension in the geotextile per unit width divided by the radius of curvature.

The strain energy in the geotextile can be calculated if the tension-strain history of the geotextile is known. The general expression for the strain energy is as follows:

$$E_f = \iint_S \left[\int_E T d\varepsilon \right] ds$$

where:

E_f = Strain energy stored in geotextile
T = Tensile force per unit width
E = Strain
S = Surface area of geotextile

Strain energy is stored permanently in the geotextile due to the strain caused by the permanent deformation of the system. Additional energy is stored during the dynamic loading of the system. A complete detailed analysis of an SGA system from its construction through many load pulses would require calculation of both components of the strain energy. In the authors' experience, the permanent strain energy is comparatively small in relation to the total strain energy absorbed by the system; and therefore, only the dynamic strain energy need be considered.

If the assumption is made that the dynamic tension-strain relationship for the geotextile is linear and the problem is considered two dimensionally, the following relationship results:

$$E_f = \frac{W}{2E_s} \int_L (T_p^2 - T_i^2) dl$$

where:

W = Width of the geotextile being stressed
T_p = Tensile force per unit width under peak load
T_i = Residual tensile force per unit width between loading
E_s = Linear relationship between the unit tensile force in the geotextile and the strain in the geotextile
L = Length of geotextile

The integration can be done graphically by plotting T_p^2 and T_i^2 on the vertical axis and the distance along the geotextile on the horizontal axis. The strain energy is the area between the two curves.

3. TENSION IN GEOTEXTILE

The tension in the geotextile is a function of the strain history throughout the system and the tension-strain response of the geotextile. These aspects are discussed in detail below.

3.1 General Behavior of the Entire System

Analysis of laboratory tests on model SGA systems (Kinney 1979) indicate that when there is no slippage between the geotextile and the materials above and below, the entire system behaves as a continuous unit and displacements throughout the system are proportional. After slippage occurs, the elements act more independently. The permanent displacements in the aggregate under the load and those in the subgrade still appear to be proportional to the displacement of the load, although the proportionality constants are different. The displacements throughout the geotextile and the aggregate to either side of the load, however, are significantly changed in both direction and magnitude.

Analysis of movies from the same laboratory tests indicates that there is very little relative movement between the aggregate directly under the load and the geotextile during loading. Slippage that occurs in this area appears to occur upon unloading. This is reasonable since during loading a high normal stress is built up across the boundary, creating the potential for high shear stress. Upon unloading, the potential for high shear stress is lost, and if there has not been sufficient rebound strain in the system to release the tensile stress, then slippage will occur.

3.2 Tension-Strain Response of Geotextile

If the strain-time history of the geotextile is known, the tension-time history can be determined experimentally by reproducing the strain-time history in the laboratory while measuring the tension. These tests would require very sophisticated equipment and detailed knowledge of the strain-time history of the geotextile in the field. The response can be approximated by comparatively simple tests measuring three properties of the geotextile.

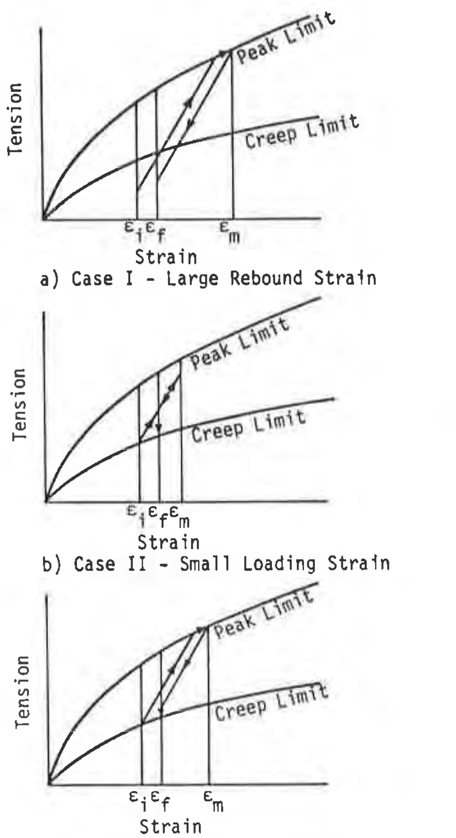
The first property is the creep limit, which is the maximum tension that can be retained in the geotextile at a given strain. A reasonable estimate of this limit may be obtained by holding a constant strain and measuring the decrease in tension with time. It may also be possible to obtain similar results using the traditional approach of holding a constant tension and measuring the increase in strain with time.

The second property is the peak repeated load tension-strain relationship. When a geotextile is strained repeatedly, there is some maximum tension that can be obtained before it will yield permanently. The results of tests performed by the authors show that a definite peak repeated load tension-strain limit is established in a relatively few number of load cycles.

There is some logic to the assumption that the peak repeated load tension-strain limit should approach the creep limit, but the experimental evidence suggests something much higher.

The third property is the relationship between the tension and the strain during the dynamic loading. Results of tests performed by the authors indicate that a fairly linear relationship is reached after a few load cycles, and that the relationship is independent of stress level.

Given these three properties, the tension in the geotextile for any strain-time history can be reasonably estimated. Figure 1 shows three possible tension-strain relationships that may exist under conditions of repetitive loading of constant magnitude. In Case I the rebound strain is sufficient to lower the tension below the creep limit. The residual tension between loadings will be below the creep limit and could be zero. The tension under peak load will be on the peak limit.



a) Case I - Large Rebound Strain

b) Case II - Small Loading Strain

c) Case III - Large Loading Strain and Small Rebound Strain

Fig. 1: Typical Geotextile Tension-Strain Relationships

In Case II the loading strain is insufficient to increase the tension from the creep limit to the peak limit. Upon unloading, the instantaneous residual tension is greater than the creep limit, hence the geotextile relaxes with time to the creep limit ready for the next load. In this case the length of time between loadings is significant. If the loadings are separated by long intervals, the residual tension between loadings will always be at the creep limit and the peak tension will be below the peak limit. Short intervals between loadings, however, will cause the peak tension to increase to the peak limit, and the residual tension between loadings to be above the creep limit.

In Case III the peak strain is more than sufficient to increase the tension from the creep limit to the peak limit; however, the rebound strain is insufficient to reduce the tension below the creep limit. The response in this case is not significantly affected by the time between loadings since the peak stress is independent of the residual starting point.

The geotextile response to any repetitive loading situation of constant magnitude will reduce to one of these three cases after a few load repetitions. If small loads are applied following a large load, the response would be controlled to a large extent by the response under the large load. If the large load caused a response as in Case I, the response under the small loads would be elastic, following the response curve from unloading the large load. When the residual tension between the small loads exceeds the creep limit, either Case II or III will control.

3.3 Calculation of Geotextile Tension

Calculation of the geotextile tension is done in two stages with two parts to each stage. First, the tension in the geotextile between loadings is determined, followed by the dynamic increase in tension during loading. In each stage the assumption is first made that there is no slippage between the geotextile and the base or subgrade. Under the condition of no slippage, the strain in the base and subgrade at the interface is estimated, giving the geotextile strain. The geotextile tension is calculated from the tension-strain relationship for the geotextile, and the tension in the geotextile is differentiated along the length of the geotextile to give the total shear stress on the geotextile. If this shear stress is greater than the maximum available, then slippage must occur. An iterative graphical procedure is described herein to demonstrate the method of determining the extent of slippage and the subsequent tension in the geotextile.

In somewhat more detail the analysis proceeds as follows:

3.3.1 Tension Between Loadings, Assuming No Slippage

The assumption of no slippage would be correct if the shear stress between the soil and the geotextile was sufficiently great. If the geotextile is soft or the deformation small, the actual shear stress developed may be high enough to validate the assumption. The geotextile tension is calculated using the following steps:

1. Determine the general tension-strain response of the geotextile.
2. Estimate the strain in the aggregate and subgrade at the interface.
3. Calculate the geotextile tension. Assume that geotextile response Case II or III is applicable and that the tension in the geotextile between loadings falls on the creep limit. Refinements for large recoverable strains, mixed loading, and rapid loading intervals can be made later if it is considered warranted.

3.3.2 Tension Between Loadings with Slippage

The analysis is then continued to determine if slippage has occurred using the following steps:

4. Determine the total shear stress on the geotextile necessary for the no-slippage condition to hold; differentiate the geotextile tension with respect to length along the geotextile.

5. Determine the maximum shear stress available between the geotextile and the materials above and below it.

If the shear stress required for no slippage exceeds the maximum available shear stress at any point in the profile, then slippage must occur and be considered. If slippage does not occur, steps 6 through 12 are omitted, and the calculations for the dynamic strain increment are started with step 13.

If slippage occurs, the following three basic rules of statics and compatibility must be satisfied in an SGA system.

* If there is no slippage, the strain in the geotextile equals the strain in the base and subgrade.
* If there is slippage, the shear stress on the geotextile will equal the maximum available.
* The total elongation within the zones of slippage must be the same for the geotextile and the materials above and below the geotextile unless the ends of the geotextile also slip.

By combining these rules in a trial and error procedure, it is possible to determine the amount of slippage at various points in the section, and to determine the tension in the geotextile.

The elongation of the geotextile can be calculated by integrating the strain in the geotextile. The strain in the geotextile is related to the tension in the geotextile, and the tension is the integral of all the shear stresses on the geotextile starting at the end. If all these relationships are lined up as shown in Figure 2, the solution becomes apparent in light of the three statics and compatibility concepts outlined above. Unfortunately there does not appear to be a closed-form solution for determining the location and width of the area of slippage. Hence, it becomes necessary to use an iterative scheme with the following steps:

6. Draw Figures 2a through 2e using the assumption that there is no slippage.
7. Estimate the location of one end of the zone of slippage.
8. Find the maximum available shear stress on the geotextile in the zone of slippage, Figure 2b.
9. Calculate the tension by integrating the shear stress from the assumed start of slippage, using the calcualted tension at that location. The end of the zone of slippage is determined when the calculated tension at that end within the zone of slippage is equal to the calculated tension just past that point where slippage has not occurred, Figure 2c.
10. Determine the strain throughout the geotextile from the tension, and the tension-strain relationship, Figure 1.
11. Integrate the strain to get the elongation and compare the elongation to the base and subgrade elongation, Figure 2e.
12. If the geotextile elongation does not coincide with the soil elongation in the areas of no slippage, then repeat steps 7 through 11 with a new location for the start of slippage.

Three special conditions may arise, but are all easily handled in turn. First, the end of the geotextile may slip. The end of the geotextile is a fixed boundary condition of zero tension and therefore controls the rest of the analysis. Second, the geotextile may slip through the entire central portion of the profile between the two wheel paths. If this happens, the geotextile can carry tension and have strain, even though the soil appears unstrained and is not slipping against the geotextile at one central point in the section. The third condition arises when the geotextile slips throughout the entire area under the wheel path. If this occurs, the controlling boundary condition is such that there can be no tension discontinuity in the geotextile.

Once the geotextile tension has been determined for the conditions between loadings, the analysis proceeds to the conditions that exist under peak load.

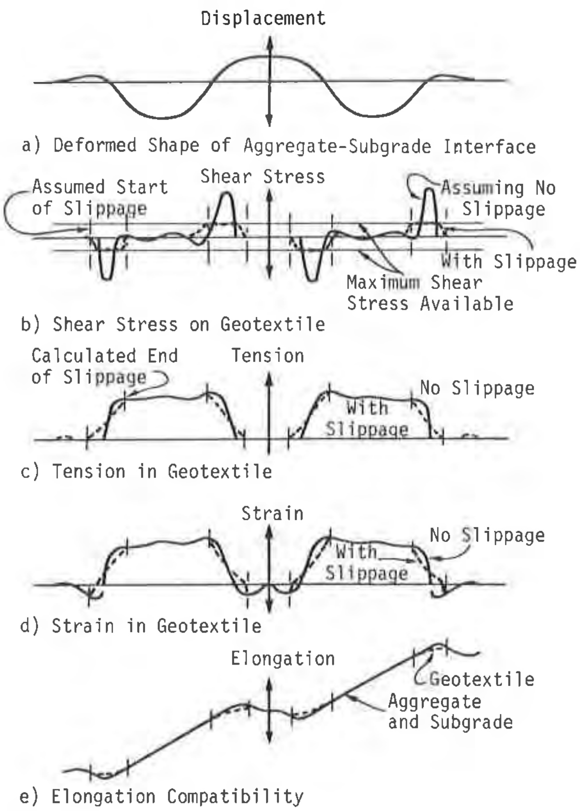


Fig. 2: Flow Diagram for Geotextile Tension Calculations Under Conditions of Local Slippage

3.3.3 Tension in Geotextile Under Peak Load

The tension in the geotextile under peak load is obtained by adding the increment of tension caused by the dynamic load to the tension remaining between loadings. The following steps are used to calculate the increment of tension caused by the dynamic load.

13. Estimate the dynamic strain increment in the aggregate.
14. Determine the tension in the geotextile, assuming no slippage between the aggregate and the geotextile during loading.

The analysis should continue to determine if slippage occurs during loading. Steps 4 and 5 are applicable; however, the maximum available shear stress is calculated using the normal stress under the peak load. If the calculated shear stress on the geotextile exceeds the maximum available at any point in the profile, then slippage has occurred. Steps 6 through 12 could be repeated to determine the extent of slippage and, hence, the geotextile tension. If the effect of slippage under peak load appears to be localized and primarily on the steep portion of the deformed interface, then revision of the geotextile tension for slippage during loading is probably not warranted.

4. DETAILS REQUIRED FOR GTM ANALYSIS

Many of the steps in the general GTM procedure require knowledge of various properties of the system that are not usually available. This section is devoted to providing the means to estimate the necessary properties to make a reasonable analysis using the GTM. The methods presented for determining the various aspects of the system geometry are based on laboratory tests, field tests, and geometrical analysis. They appear to adequately represent the system geometry.

4.1 Three-Dimensional Effect

Before application of the wheel load, the rutted profile is two dimensional and the geotextile is not under tension in the direction parallel to the rut. The response of the base and subgrade to the wheel load is three dimensional.

Based on the laboratory tests, Kinney (1979), it seems reasonable to assume that the section is two dimensional with an effective width equal to 3/4 of the width of the depressed portion of the subgrade surface.

4.2 Maximum Shear Stress Available

The maximum available shear stress on the geotextile is the sum of the maximum available shear stresses on the top and bottom of the geotextile. Each may be reasonably represented by a friction angle and an adhesion. The normal stress on the geotextile must be calculated considering the tension in the geotextile and the loading conditions at that time and location.

4.3 Recoverable Displacement of Load

The traditional solution developed by Burmister is based on a uniform circular load on a two-layer elastic system. Figueroa (1979) developed the following regression equation based on an extensive set of finite element analyses which may be more applicable:

$$\log \frac{d}{25.4} = -0.92 - 0.00067\,t - 3.6\,E_r$$

where:

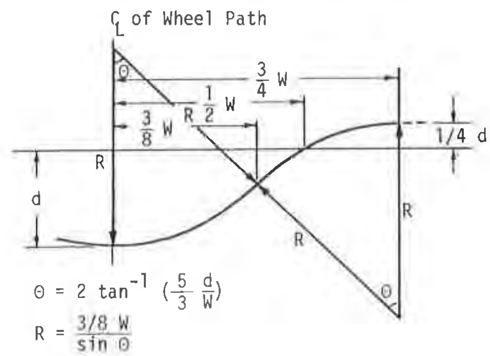
- d = Resilient displacement of load - mm
- t = Thickness of the base - mm
- E_r = Resilient modulus at a deviator stress of 41 kPa - KPa

4.4 Deformed Shape of Interface

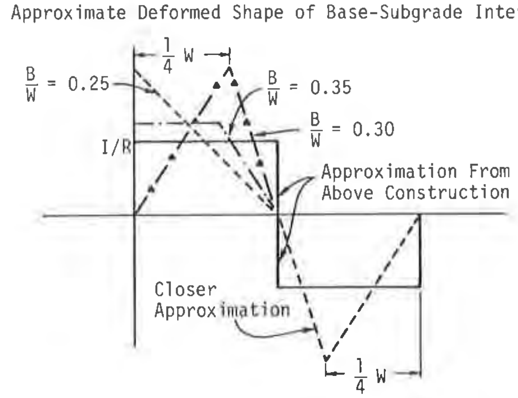
A fairly simple graphical construction produces a surprisingly accurate estimate of the deformed shape of the geotextile in laboratory tests, Kinney (1979). The deformed shape in the field should be similar to that measured in he laboratory.

Width of Rut in Subgrade - The width of the depressed portion of the rutted subgrade is a function of the surface rut width, the aggregate thickness, and the geotextile stiffness. The relationship for estimating the width is given in detail in Kinney (1979) and Kinney and Barenburg (1980).

Shape of Rut in Subgrade - The shape of the rut can be approximated fairly accurately by using three circular arc segments of equal radius as shown in Figure 3a.



a) Approximate Deformed Shape of Base-Subgrade Interface



b) Radius of Curvature of Base-Subgrade Interface

Fig. 3: Approximate Deformed Shape of Granular Base-Subgrade Interface

Radius of Curvature of Rut in Subgrade - Although the deformed shape is quite closely approximated by circular arcs, the true radius of curvature can vary significantly from this. The variability appears to be primarily dependent upon the ratio of the width of the surface rut to the width of the depressed portion of the geotextile. Possible modifications to the basic model are shown in Figure 3b.

Elongation of Base and Subgrade at Interface - Throughout the laboratory tests, Kinney (1979) and Kinney and Barenberg (1980), the shape of the elongation curves for the aggregate and the subgrade were similar, particularly near the center. Both materials appeared to undergo nearly constant strain throughout the center portion under the load and very little strain outside this area. The total amount of elongation is the difference between the length of the curved surface of the rut in the subgrade and the original horizontal length of this same surface. The average strain is therefore the total elongation divided by the length over which most of the elongation takes place. The relationships for elongation and strain are given in Figure 4.

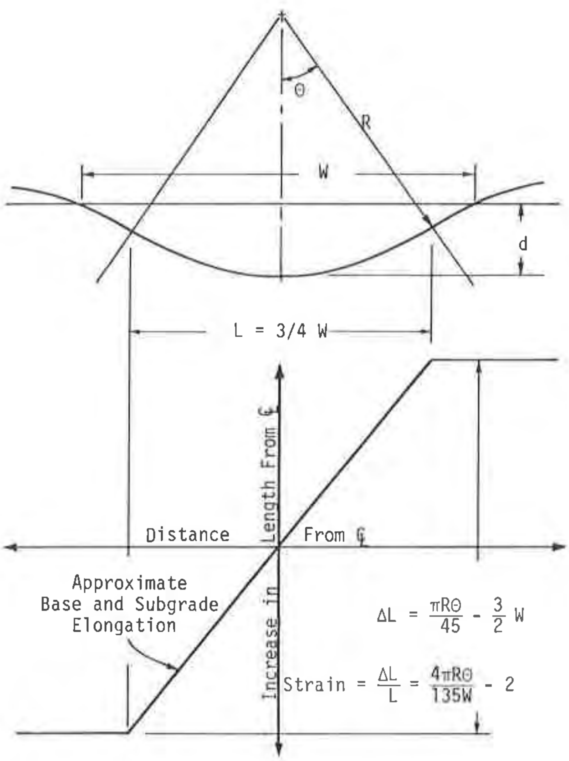


Fig. 4: Approximate Elongation and Strain in the Base and Subgrade Along the Interface

Depth of Rut in Subgrade - The amount of vertical displacement in the base subgrade interface is controlled by the depth of rutting at the surface, plus the volume change and lateral spreading of the granular materials under the load. If the surface is regraded, or aggregate sloughs off the sides of the rut into the bottom, the displacements in the subgrade will no longer resemble the displacements at the surface.

The amount of volume change of the granular material under the load is dependent upon the aggregate properties, the placement conditions, the system geometry, and the conditions of loading. It appears reasonable to assume a 5 to 10 percent total reduction in volume in the absence of any actual data. The decrease in thickness due to lateral spreading of the granular layer under the load can be calculated from the geometry of the system, based on the change in the dimensions of a rectangular block immediately below the load.

5. CONCLUSIONS

The concepts and data correlations presented herein provide valuable insight into the behavior of a geotextile in a rutted unpaved road. A technique is presented which can be used to calculate the tension in the geotextile at any point under the influence of the wheel load and empirical correlations are presented to estimate the shape of the surface of the geotextile in the area of the rut. These are combined into an outline of a design procedure which includes:

* Normal stress on the geotextile.
* Shear stress developed by the geotextile on the aggregate and the subgrade.
* Strain energy stored in the geotextile.

The performance of model tests have been accurately predicted using only the upward-directed normal stress caused by the geotextile and the strain energy stored in the geotextile. Using only the upward-directed normal stress caused by the geotextile underestimates the beneficial effects of the geotextile in field applications. More work is necessary in developing the ideas presented herein and applying them to road design. In particular, the effects of the shear stress caused by the geotextile need refinement.

6. ACKNOWLEDGEMENTS

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Geotextile Performance at an Access Road on Soft Ground near Rio de Janeiro

Comportement d'un géotextile dans une voie d'acces sur sol compressible près de Rio de Janeiro

This paper presents a case history on performance evaluation of a non-woven textile as a reinforcement at the base of a low embankment access road, constructed across a very soft soil. Four test section, in which the geotextile type and installation method were varied, were built. The instrumentation installed in each test section allowed the observation of vertical and horizontal displacements. The number of trucks passing on each test section was recorded. Borings were dug through the fill for the definition of its thickness. The results were compared with similar measurements made at sections with no reinforcement. Measured geotextile performance, expressed in per cent reduction of fill material consumption seems to vary between 10 to 24% in volume. This difference seems to depend mainly on the way the geotextile was laterally fixed. An economic evaluation, considering local costs, has shown that this geotextile application would not be economically advantageous unless the reduction in volume of fill material exceeds 30%.

Cet article présent la description de la construction et du comportement d'un remblai de basse hauteur construit sur un sol três compressible utilisant un geotextile non-tissé pour son estabilisation. Quatre sections experimentales variant le type et la méthode d'installation du geotextile ont été construites. La instrumentation instalée permetait la determination des déplacements horizontales et verticales sous le remblai. Le nombre de camions passant sur le remblai etait precisé et son épaisseur determinée plusiers fois. Les resultats ont été comparées avec ceux d'un remblai construit directement sur le sol argileux. L'influence du geotextile, indiqué par la reduction de volume necessaire de remblai, fait de 10 a 24% en function de la forme de fixation du geotextile à coté du remblai. Une étude économique a indiqué que, pour une application pratique, la reduction de volume devrait etre au moins de 30%.

INTRODUCTION

A comprehensive research program on the behaviour of embankment founded on soft soil has been conducted by a research team from the Federal University of Rio de Janeiro and the Catholic University of Rio de Janeiro with financial support from the Highways Research Institute. In order to carry out full scale experiments. a testing site, located about 10 km north of Rio de Janeiro, was selected where a thick deposit of about 11m deep of a very soft gray clay occurs. This site is a part of a large swampy area, locally known as "Fluminense Plains" covering an area of about 150 km^2 around Guanabara Bay. At this site, two main instrumented trial embankments have already been built (see figure 1 and 2). The first one was built in December 1977 and reached failure. An analysis of the observed field behaviour was made by Ramalho Ortigão (1980) and is also presented by Ramalho Ortigão, Lacerda & Werneck (1982).

Following the scheduled research program, a second trial embankment was recently constructed aiming at testing different types of sand and prefabricated vertical drains. Field measurements are still being conducted and will be presented by Collet (1982). This trial embankment is about 360m long, 35m wide, and 2,5m high, comprising 5 sections in which vertical drains were installed in a square area of 35m x 35m. Two additional end sections were built at the embankment ends to allow comparisons and evaluation of vertical drainage performance. In order to provide access for installation of the foundation instrumentation, placement of 0,5m thick sand mat and the installation

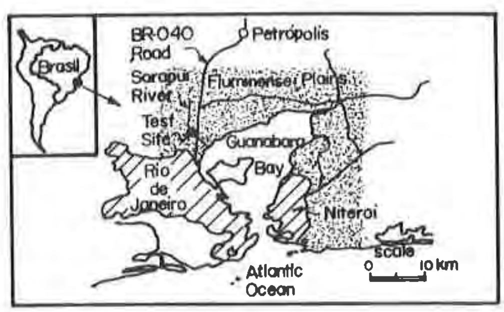

Fig. 1 - Test Site Location

of the vertical drains, a lateral access road had to be constructed. This road consisted of a low embankment, about 1m high and 7m wide, with its axis parallel to the main embankment longitudinal axis, and about 33m apart of it (axis to axis). The construction of this access road consisted of an unique opportunity for testing the influence of geotextile at the base of a low embankment, since this additional field trial could be accomplished with little increase in total costs. This paper gives a brief picture of the field observations made to evaluate geotextile performance. A complete description can be found in Palmeira (1981).

SOFT FOUNDATION PROPERTIES

The foundation soil consists of a 11m deep deposit of soft Rio de Janeiro Gray clay overlying sandy layers. Clay properties were studied by Pacheco Silva (1953) and recently covered in more details by Lacerda et alii (1977), Werneck et alii (1977), Costa Filho et alii (1977), Ramalho Ortigão and Costa Filho (1982), among others.

Figure 3 summarizes geotechnical properties. Liquid limit varies from 150%, near the top, to 90% near the bottom, the in situ water content being slight higher than these values. Plasticity index is about 80%. Field vane tests results ranging from 5 to 15kPa have shown a decrease in undrained strength at the clay crust, the minimum value being recorded at a depth of 2.5m, below which the undrained strength seems to increase linearly with depth. A mean value of undrained strength is about 10 kPa. Field vane tests also indicates the clay sensitivity is in the order of 2 to 4. Stress history was evaluated from several high-quality oedometer tests, which have shown the over-consolidation pressure to be slightly higher than the in situ vertical effective pressure, as shown in figure 3.

TEST SECTIONS

From an initial suggestion by Rhodia Company (the Brazilian subsidiary of the French Rhône Poulenc Group) the experiment was subsequently designed. It comprised four sections, in which the geotextile and installation method were varied. Two geotextiles were employed at the test sections. They consisted of non-woven needle punched polyester fabric types OP-30 and OP-40, commercially known as BIDIM, and their main characteristics are as follows:

TABLE 1 - GEOTEXTILE PROPERTIES

	OP-30	OP-40
mass per unit area (g/m^2)	300	400
thickness (mm)	3,5	3,8
Monodirection tensile strenth (kN/5 cm)	0,80	1,05
elongation at failure (%)	50-70	50-70
Bidirection tensile strength (kN/m)	23	31
elongation at failure (%) 2-D test	27-30	27-30
approximate local price in US dollars per m^2 (Jan. 1981)	1,20	1,60

Each test section comprised an area 7m wide, 20m long at the base of the embankment. At the first test section, named S1, the geotextile type was OP-30, while at the others sections, named S4,S5 and S6, the OP-40 type was laid down at the base of the embankment. Two additional instrumented sections, names S2 and S3, were set up with no geotextile in order to provide data for evaluating geotextile influence. Figure 4 shows the way the geotextile was laterally fixed in each section.

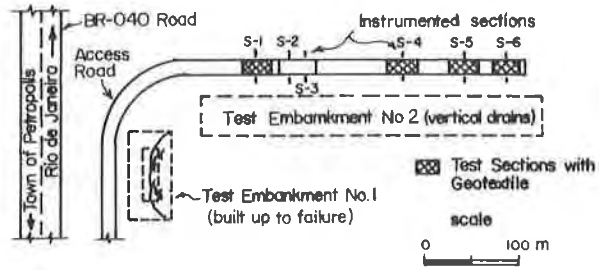

Fig. 2 - Test Site

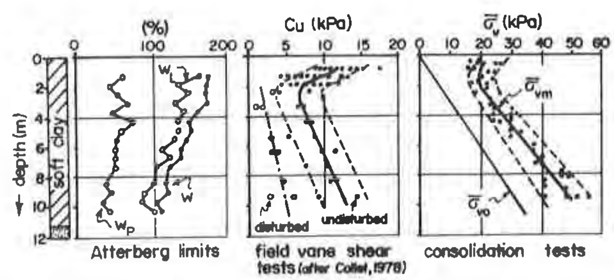

Fig 3 - Summary of Geotechnical Properties - Rio de Janeiro Soft Gray Clay

ROAD EMBANKMENT DESIGN AND CONSTRUCTION

Main characteristics for the design of the access road were as follows: lane width about 4m and truck load of 71kN. Entering this information plus the necessary foundation data in the design charts available from geotechnical manufacturers, this generally yielded embankment thickness as low as 30cm, when geotextile reinforcement was adopted. A similar value for the embankment thickness was also obtained through Giroud & Noiray's -1981) charts. Such a small thickness, following design recommendations, could only be accomplished if aggregate material was employed in order to better distribute truck load to the soft foundation. However, at the test site, the type of fill material available at a reasonable cost differed too much from the recommended one. It consisted in a residual clayey soil of gneissic origin, excavated from a borrow pit about 20km away from the test site. Its main characteristics are as follows:

$w_L \cong 49\%$, $w_p = 31\%$, $\gamma_t = 20.5$ kN/m^3 and $w = 20 \pm 4\%$

at placement conditions. Grain size distribution, has shown 67% in weight finer than 0,075mm in diameter. Sites where an easily available softer fill material is used, rather than a costly aggregate, are not uncommon. In such a situation, current design recommendations for geotextile reinforcement do not apply. Therefore, the devised field trial was also an opportunity for the evaluation of the efficacy of geotextile reinforcement applied to a relatively soft embankment material. Prior experience at the test site indicated that minimum height to support low traffic was about 0,8m. This was then, the designed height for the access road.

Placement was accomplished by means of a light bulldozer and no direct compaction was exerted on the fill. It was difficult to keep the road level under an accurate control, since the only available placement equipment, a light bulldozer, was not the most appropriated equipment to level the embankment. In addition, as construction progressed, maintainance had to be exerted of the road, already crossed by the trucks which delivered fill material to the placement area. Indeed, the actual road level was always kept to minimum necessary to support traffic load, since embankment maintainance refilling operation was carried out as necessary only to ensure trafficability.

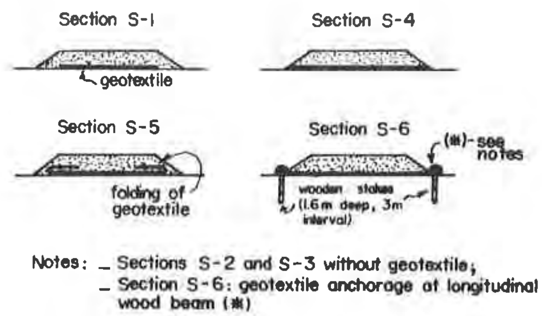

Fig. 4 - Geotextile Installation

INSTRUMENTATION

An instrumentation was installed to observe embankment deformation. The relatively small dimensions of the embankment cross section limited the type of instrument that could be employed for monitoring field behaviour. An horizontal magnetic extensometer and a profilometer (a full profile settlement gauge) were, then, chosen to monitor horizontal and vertical displacements, respectively. A typical lay-out of an instrumented section is presented on figure 5. Plastic plates 30 cm x 30 cm containing a built-in magnetic ring were laid down around a plastic access tubing, allowing the slide of a magnetic sensor to locate the position of the magnets. This permitted the calculation of horizontal displacements in the foundation soil just below the geotextile. In addition, magnetic rings around another plastic access tubing, were directly attached to the fabric allowing measurements of geotextile horizontal displacements.

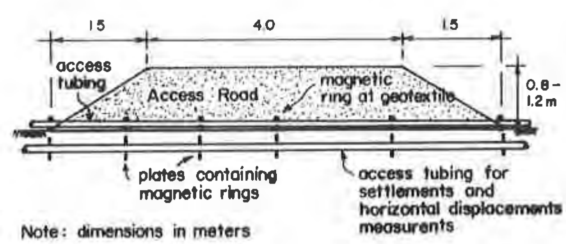

Fig. 5 - Typical Instrumented Section

A vertical displacement profile was obtained by sliding the profilometer sensor device through the same access tubing of the magnetic extensometer. This instrument, which was specially devised for this work, (Palmeira & Ramalho Ortigão, 1981), was hydraulically operated, and yielded good results. Prior experience with the described instruments allows an evaluation of the accuracy of the measured deformation. Accuracy of horizontal displacements is in the order of ± 3mm, while the accuracy of vertical displacements is estimated to be in the ± 17mm range, with a confidence level of 95% (Palmeira & Ramalho Ortigão, 1981). In addition to instruments monitoring, field data included recording the thickness of the embankment through boreholes drilled through it. The number of trucks passing on each test section was also recorded during six months after construction of the access road.

FIELD OBSERVATIONS

Many initial readings were carried out, on the instrumentation prior to fill placements. However, at the placement of the first layer, which had a minimum thickness of about 50cm, necessary to support the bulldozer load, some disturbance, or sometimes damages, ocurred to the instrumentation. As a result a new rezeroing operation had to be conducted in all the instruments. At section S3, initial placement provoked a major damage to the instrumentation and, thus, this section had to be abandoned.

An important observation was made through the comparison of field horizontal displacements obtained from magnetic rings attached to the geotextile and magnetic plates of the same position. It was observed that the effect of fill placement was more evidenced by the magnetic plates. Also, these plates have shown a more pronounced deformation at the first passage of the bulldozer or a truck than the magnetic rings. This behaviour was atributed to relatively large dimensions (30cm x 30 cm) of the plates when compared to the low embankment height. The above observation was made at the test sections which were firstly built. This has led to a modification of the instrumentation of the S6 section, the last one to be constructed. In this section, in addition to the magnetic rings attached to the geotextile, some other rings were installed with no attachment at all, just embedded in the fill material above the geotextile. By comparing the results of the horizontal displacements of the fixed and free rings, no significant difference could be detected. Therefore sliding at the fill-geotextile interface was supposed to be negligible.

Field data processing included preliminary plots of horizontal and vertical displacements against time and against the number of trucks passing on each section. Time was initially taken into account since heavy traffic was mainly concentrated in the last three months of the observation period. However, these plots have shown that measured displacements were mainly traffic dependent. Time effect was therefore disregarded in the analyses.

From another set of plots, it was observed that the calculated strains at the base of the embankment, rather than the measured displacements, presented a more consistent pattern of its distribution accross the embankment section. Therefore, horizontal displacements data will not be shown in this paper.

Main field observations carried ou up to 6 months after the completion of the access road, are then, summarized in figures 6 and 8, and in table 2. Figure 6 shows settlement and horizontal strain profiles for an increasing number of trucks (50, 100 and 200) which crossed each test section. The following observations can be drawn from this figure:

1.- Maximum measured settlements were about 150-200 mm and horizontal strains reached values of about 10 per cent, maximum values generally taking place under the tracks of the wheels;

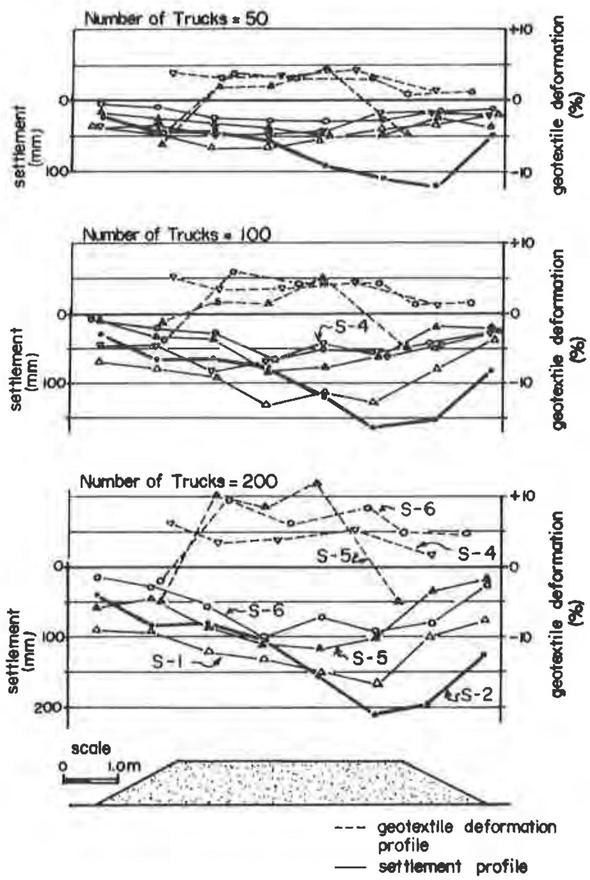

Fig. 6 - Settlements and Geotextile Deformation Profiles versus Number of Trucks at Test Sections

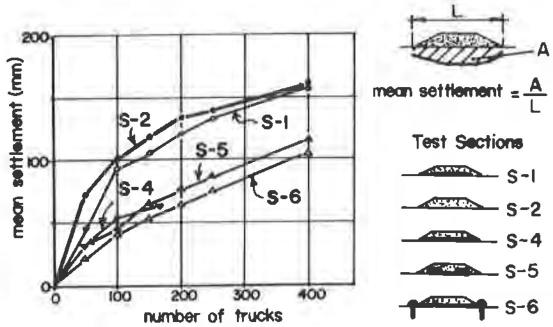

Fig. 7 - Mean Settlement versus Number of Trucks at Test Sections (Full Section)

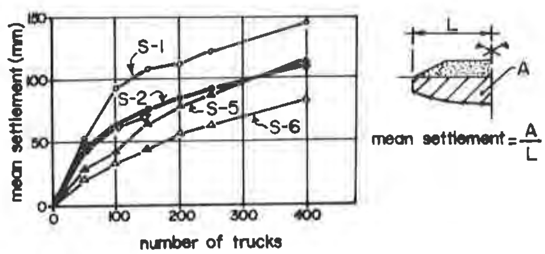

Fig. 8 - Mean Settlement versus Number of Trucks (Half-Section, without Local Failure)

2.- The geotextile installation method, as employed in section S6 (with wooden stakes)yielded more uniform distributed tensile strains, as recorded along the whole cross section and reaching lateral stakes at the foot of the embankment;

3.- The occurrence of a local failure can be observed in section S2 (no geotextile) as evidenced by the large settlements on the right part of the cross section;

4.- Local failure also seems to have taken place on section S1 (geotextile only beneath the embankment platform), where fabric installation method seems not to be efficient;

5.- Other sections with geotextile (S4, S5, and S6) seem not to present a local bearing failure, and this can be attributed to the effect of the geotextile reinforcement.

The observation made can be further emphasized by plotting mean settlement versus the number of trucks, as done in figures 7 and 8. In these plots, settlement distribution was averaged along the full cross section (fig.7) or only for half-sections (fig.8) without local yield. Figure 7 indicates that in sections without geotextile (S2), or where the geotextile was poorly installed (S1), the mean settlement is greater than the settlement at sections where geotextile was extended beneath the embankment shoulder (S4, S5 and S6). On the other hand, when local failure does not seem to occur as at the half-sections shown on figure 8, geotextiles loses efficiency and its application is not worthwhile for minimizing settlements.

Another measurement carried out at the end of the period of observations has been the measurement of the thickness. This was accomplished by drilling Ø 150 mm boreholes through it. These boreholes were located at the tracks left along the access road by the wheels of the trucks. Nine determinations of fill thickness were performed in each test section and the results have been indicated in table 2. This table includes the mean value and the standard deviation of data. The difference between mean values are small, however a simple statistical test have shown with a level of confidence on 95% that all these distributions of embankment height are different from each other. This difference assumed to be due to different reinforcements. Data in table 2 also show the greater meam value of fill thickness taking place where no reinforcement was present (S2 - section). On the other hand, at section S6, where the geotextile was laterally fixed with stakes, and where tensile strains were more uniformly distributed over the fabric, the minimum mean value of fill thickness was recorded. Now, comparing the reduction of mean fill thickness in each section with the results of section S2 (no reinforcement) the following data is obtained.

TABLE 2 - MEASUREMENTS OF EMBANKMENT HEIGHT

Section Nº	Mean Value	Standard Deviation (m)
S1	1.04	0,10
S2	1.15	0.09
S4	0.97	0.04
S5	0.94	0.05
S6	0.89	0.06

TABLE 3

Sections w/ reinforcement	Reduction in mean emb. height (%)
S1	9.6
S4	15.7
S5	18.3
S6	22.6

The main difference in the reinforcement application in section S1 to S6 is that lateral anchorage or lateral fixing of the geotextile is increased, as the section reference number increases (see figure 3). It follows, therefore, that differences in data in the above table can be regarded as a measure of reinforcement efficiency. In fact, in section S6, where the geotextile seemed to be best laterally anchored, a more uniform distribution of horizontal strain was observed and this test section yielded the greater reduction of mean fill thickness. The lowest reduction was achieved in section S1, in which the geotextile was not extended beneath the embankment shoulders, yielding, thus, the poorest reinforcement efficiency. This latter conclusion related with test section S1, disregards the fact that the geotextile at this section was type OP-30, which is thinner and can carry less load than type OP-40 employed at other sections. However, this fact seems to be less important to the overall geotextile efficiency than the way the geotextile was applied (with more or less lateral anchoring).

ECONOMIC CONSIDERATIONS

In addition to technical evaluation,the economic efficiency will play, of course, a major role in the decision of employing or not the geotextile. For the conditions prevailing at the test site, circa at early 1981, fill material cost was evaluated in US$ 5.36 per m^3, which includes: excavation at the borrow pit area, 20 km of transportation distance to the test site and placement. This value was mainly influenced by the cost of transportation.
The cost of the geotextile was shown in table 1. Those values should be added to labour costs for reinforcement installation, which at local prices and due to the type of work, are minimal.

Considering these points, final costs of access road per unit length in each test section were evaluated as shown in table 4.

TABLE 4 - EMBANKMENT COSTS

Section number	Emb. Vol. m^3/m	% vol saved	Emb. cost US$/m	Cost of Reinf. US$/m	Total cost US$/m	Total cost ratio
S2 *	7.2	0	38.59	0	38.59	1.00
S1	6.5	10	34.84	8.40	43.24	1.12
S4	6	17	32.16	11.20	43.36	1.12
S5	6	17	32.16	11.20	43.46	1.12
S6	5.5	24	29.48	14.40**	43.88	1.14

(*) without geotextile (**) Includes cost of wooden stakes

Data shown in table 4, reflecting conditions strictly prevailing at the test site, shows that the geotextile application increased the total costs in 12 to 14%. On the other hand, a simple calculation shows that the geotextile reinforcement, as installed in section S4. would be cost-effective only if savings in fill material exceeds 30% in volume.

CONCLUSIONS

Many observations and conclusions were drawn from the full scale instrumented field trial previously described. Main features will be reviewed below:

1. - An instrumentation for measuring vertical and horizontal displacements at the base of the road was successfully accomplished by means of a profilometer and a magnetic horizontal extensometer.

2. - Experience with the instrumentation and the observations made indicated that the magnetic rings, rather than plates containing rings, should be used as targets for the horizontal extensometer, in order to minimize the effect of shocks at fill placement.

3. - Field observation have shown that the geotextile reinforcement seems mainly to prevent local yield to take place at the foundation, thus minimizing fill material consumption.

4. - Maximum technical efficiency of the geotextile reinforcement was achieved by means of wooden stakes. This implied in the minimum fill material consumption per meter of road length.

5. - The installation methods consisting of laterally folding the geotextile (section S5) or laying it to cover the whole embankment cross section base (section S4), yielded similar results.

6. - Field measurements have shown that geotextile applications have saved 10 to 24% of volume of fill material, this difference depending on the installation method, i.e., the way the geotextile was laterally extended.

7. - An economic evaluation considering local prices have shown that the geotextile reinforcement would be cost-effective only if savings of fill material exceeds 30% in volume.

8. - An effective numerical modelling of such a field trial should consider local yielding at the foundation and the effect of traffic. A first attempt by Palmeira (1981), employing a Finite Element program considering linear-elastic materials and a bar element for the geotextile, did not yield good results.

9. - Available design charts, as those published by Giroud and Noiray (1981), have been developed only for stiff embankments made of aggreagte, not considering the effect of a softer material as employed in this field trial.

ACKNOWLEDGEMENTS

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Jute Fabric in Road Construction

Les "tissés" jute pour la construction des routes

There has been a rapid growth of road development in the developing countries. Exploring successfully the possibility of using cheap and alternative types of locally available fabrics for stabilizing poor subgrade soils in place of the synthetic geotextile versions would mean a lot to these countries in terms of dollars and cents.

The jute fabric appears to hold promise as a cheap alternative to the imported synthetic versions in so far as the filtering ability, efficiency and non-clogging properties are concerned. From the point of view of durability however, long term studies are needed to be carried out.

In this paper, the authors discuss the feasibility of using jute fabric as an alternative substitute to the synthetic non-woven types of application in road construction for the developing countries.

Les pays en voie de developpement connaissent actuellement une importante croissance du reseau routier.

Des enconomies importantes peuvent etre realisées utilisant des geotextiles ayant des materiaux locaux comme counstituants.

Les tissés jute, sont une alternative economique et ayant toutes les proprietes de filtre et couche anti-contaminante.

Quant a la longevite, des etudes a long terme doivent etre realisées.

Cet articles analyse la faisabilite d'utilisation des "tissés" jute en remplacement des geotextiles "non tissés", pour les foundations de routes dont les pays sont en voie de developpement.

1 INTRODUCTION

As the cost of aggregates and haul costs rise, filter fabric becomes an increasingly viable alternative to stabilize the subgrade in road construction. The use of jute fabric increases the soil CBR value and therefore the stability, prevents the intermixing of aggregate and subsoil, improves compaction and permits vertical and lateral drainage. The use of jute fabric permits material saving in the design depth of road structure reduces maintenance costs in terms of materials, labour and equipment, allows construction to proceed in inclement weather and from environmental considerations reduces the cost of excavation and therefore the silt transportation into drains and water courses. It can therefore be said that the jute fabric holds great promise for use as a geotextile in subgrade stabilization in Asian and Southeast Asian countries. The two major jute producing countries of the world are India and Bangladesh. If these two countries make use of their own product in the form of jute fabric to stabilize the subgrade soils for their several thousands of kilometers of rural roads alone for light and medium traffic, at least 10% to 20% of reduction would result by way maintenance costs. It would mean a savings of several millions of dollars which would have been spent otherwise in extra maintenance costs in local currency or in terms of foreign exchange if an alternative imported synthetic fabric had been used.

It has been reported (1) that jute has been extensively used successfully and economically in projects requiring high strength with a caution as to its rapid decay as a result of its water absorptive nature. The tests by authors have confirmed the good aspects of strength qualities of jute fabric in its fresh condition. Under the water content and temperature variations occuring within the subgrades of Singapore, the decay of jute fabric has been observed to be so slow that it does not disqualify its use as a geotextile to separate the subgrade from the sub-base in road construction. An interesting study on the feasibility of using jute fabric as a geotextile in road construction has been reported in this paper.

2 PROPERTIES OF JUTE FABRIC

2.1 Strength

The jute fabric reported in this paper consisted of tightly woven burlap of 850 g/m^2 weight. The threads used were of 3 mm diameter being made out of well twisted bundle of minute hair-like jute fibres. Typical properties of the jute fabric tested were as in Table 1.

Table 1. Properties of Jute Fabric

Grab tensile a strength (wet), N	760
Elongation at break (wet), %	15-20
Trapeziodal tear strength, N	350
Permeability, cm/sec	
- Unstressed condition	more than 10^{-2}
- Under an all round pressure of 500 kN/m^2	10^{-3} - 10^{-4}

Since the strain at failure is relatively small compared with many other non-woven synthetic fabrics, the jute fabric has a relatively high tensile modulus. It is obvious that the higher the modulus, the thinner is the required aggregate layer (2). The reduction of aggregate thickness resulting from the use of any geotextile is known to range from 20% to 60% for a subgrade soil of CBR = 1 for light to medium traffic (2).

The high tensile strength while facilitating rugged handling and damage free installation also guarantees performance ensuring continuity even under heavy compaction. The average properties of the jute fabric were not significantly different either when the fabric was dried up in the hot sun or wetted under pouring rain. Therefore it can be said that storage of jute fabric in the open despite sun or rain may not be forbidden and wet ground or rain does not impede laying of the fabric.

Jute fabric surface is rough enough to provide a good bond either with the subgrade beneath or with the aggregate layer above it.

Single layer of a coarse and heavy sacking jute fabric has been found suitable for packaging with a holding capacity of as much as 10 kN (3). Strength and creep tests have confirmed that jute fabric packs are preferable to the traditional bulk containers of metal or timber.

The strength of individual threads consisting of twisted bundle of hair-like fibres and its light weight makes the jute fabric flexible, easy to handle and to work with. The jute fabric is tough enough to resist damage due to rough handling during installation and tamping of the granular sub-base. The fabric strength prevents propogation of tears and punctures and provides excellent energy absorption capability.

2.2 Drainage

The jute fabric for use as a geotextile should consist of twisted jute fibres into threads of 2 mm to 4 mm in diameter closely woven to form a sheeting of comfortable width around 4 m rolled and supplied to the site for easy spreading on the subgrade. The jute fabric when closely examined reveals tiny square openings cris-crossed by a number of minute fibres which in turn contribute in considerable measure to the further reduction in the side of openings and aid in the filtering process. The size of the openings for the woven fabric itself can be specified to the supplier. An interesting point to note is that apart from the regular tiny openings available in the woven fabric, each twisted jute thread itself can provide a path for water by capillary action. Jute fabric therefore has the ability to absorb water and under-layment use, water moves along the plane of the fabric by siphoning action.

The filter fabric for road construction should have good mechanical-strength, sufficient permeability, capacity to retain fine soil particles, resistance to chemical and biological attack. The only non-synthetic woven fabric which satisfies most the desired properties is the jute fabric.

2.3 Clogging

It has been found by triaxial tests that clay intrustion into the jute fabric when confined alround by soft clay and subjected to gradually increasing alround pressures of up to 100 kN/m^2 was insignificant. The permeability under such conditions was never reduced below a value of 10^{-4} cm/sec.

2.4 Durability

The jute fabric is manufactured from jute fibres which are by nature strong, durable and highly resistant to decomposition under chemicals or adverse environmental conditions. The fibres are extracted from the jute plants by a process of keeping them under water for several weeks and therefore are resistent to deterioration under prolonged exposure in wet soil. The subgrade moisture in Singapore (4) and other neighbouring countries under similar climatic conditons is known to vary between 18% to 30% the year round with not much variation in subsoil temperature which would be around an average of 30°C irrespective of the season. In fact there is no well defined season for countries like Singapore and Malaysia. The jute fabric embedded in soil in such an environment is expected to remain always moist. It is never subjected to conditions such as intermittent wetting and drying or freezing and thawing. Under such conditions, the durability of the jute fabric may not be a matter of concern for a period of at least one year.

The use of jute fabric sacks for storing materials such as common salt, cement, fertilizers etc is quite well known and attests for the chemical inertness of the fibres. Laboratory studies on jute fabric exposed to acidic and alkaline environments (pH = 3 and 12)have shown that in a period of one year the tensile and tear strength are only decreased between 10% to 30%.

It is the opinion of the authors that since the subgrade gets consolidated under the self weight of the pavement as well as under the construction rolling and traffic wheel loads, it is expected to attain the maximum stability within about an year's time. The subgrade so stabilized can take care of itself without the aid of jute fabric in so far as withstanding the stresses and strains are concerned unless the traffic intensity increases beyond the one to which the subgrade has been used to with the aid of jute fabric within its first year of carrying the road on it. The question of durability of jute fabric should not therefore seriously affect its usage as a geotextile for subgrade stabilization. The drainage layer above the stabilized subgrade would stay intact and function as long as the subgrade is not overstressed.

3 PLACING OF JUTE FABRIC

The geotextile reported in this paper is made up of jute fibres woven in the form of coarse and thick (2 mm to 4 mm) fabric which can come in rolls of 4 m or more in width to facilitate easy unrolling on the surface of the subgrade to be treated.

Jute fabric is spread directly over the roughly levelled poor subgrade soil. In the case of clayey subgrades (percentage of fines exceeding 50) it is recommended to spread the fabric after placing a layer of sand of 10 mm to 20 mm thickness. The fabric is then surcharged with granular material preferably sand of 30 mm to 50 mm thickness to act as a lower sub-base and it is rolled initially with light rollers and later if possible with medium to heavy rollers.

A layer of sub-base consisting of coarse aggregate or crushed rock (locally known as crusher-run) varying in thickness from 200 mm to 300 mm may be placed over the sand layer and compacted. Under the surcharge action of sub-base layer and compaction rolling, the subgrade loses water content through the filter fabric and gains strength.

Unrolling of the fabric can be done easily manually and great accuracy in alignment is not required. For multi-lane roads an overlap of at least 300 mm is preferred where necessary. Wastage in overlapping can be reduced by just folding the edges together and stitching longitudinally by using a portable sewing machine. The fabric covering the subgrade may be spiked if necessary by the use of U-shaped spikes driven at random as necessary to keep the fabric in place during construction and rolling.

Proper placement of fabric to ensure lack of continuity with suitable overlapping or stitching wherever required is important. The extreme flexibility of jute fabric allows it to bend, and fold making it quite versatile in easy spreading.

In the event of a tear occuring, the damage remains localized and does not spread progressively like in the case of a woven cotton fabric. In this respect, the jute fabric can be considered to behave much like any other non-woven synthetic fabric. Any accidental damage does not therefore affect the overall performance of the jute fabric.

For unstable and wet subgrades, jute fabric appears to provide a satisfactory solution to stability and drainage problems. Fig. 1 illustrates the jute fabric in position within the road structure.

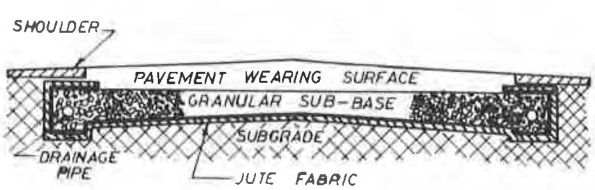

Fig. 1 Position of Jute Fabric in the Road Structure

The jute fabric placed as above acts as a separator to eliminate the punching of aggregate into the soft subgrade as well as to resist the infiltration of fines from the subgrade into the aggregate layer thus arresting any tendency for 'pumping'. The drainage system also maintains optimum performance because the fabric does not get clogged under field conditions. The high tensile strength and tear resistance make the jute fabric to act as a support membrane to reduce localized distress to the road surface by redistributing traffic loads over a wider area of subgrade. This would of courseresult in the reduction of thickness of overall road structure resulting in some reduction in the quantity of earthwork as well. It has been reported (2, 5, 6) that with the protection offered by geotextiles to the subgrade, less sub-base is needed and therefore less subgrade needs to be excavated.

Durability studies have confirmed that the fabric retains sufficient strength for about a year. Long term durability studies and the decaying of jute fabric with time under different environmental conditions are needed for proper long term assessment. In the opinion of authors, the strength and condition of the jute fabric beyond a period of one year after placement should not be of any concern as by that time the fabric would have already played a very important role in providing a self sustaining subgrade for most types of soils. After placement, the jute fabric will strengthen the subgrade by consolidating it by removing the water gradually in a step by step loading starting from the granular surcharge of sub-base, base and road surface layers, roller compaction of various layers and finally under the traffic wheel load compaction at least for a period of 6 months. The gain in strength of the subgrade with time can well be compensated for the loss of strength of the jute fabric within the same time frame. The granular drainage layer placed above the stabilized subgrade would continue to function permanently.

4 LABORATORY STUDIES

In order to study the influence of jute fabric on the strength of clayey subgrade, unconfined compression tests and CBR tests were carried out in the laboratory on samples compacted with and without fabric layers in the saturated and unsaturated conditons. The standard proctor compaction tests were carried out on soils without fabric as well as on samples with 2 layers of fabric embedded at mid-depth. Fig. 2 shows the effect of a fabric layer on the soil compaction characteristics. For the same compaction effort, the soil is seen to be better compactible when the fabric is used.

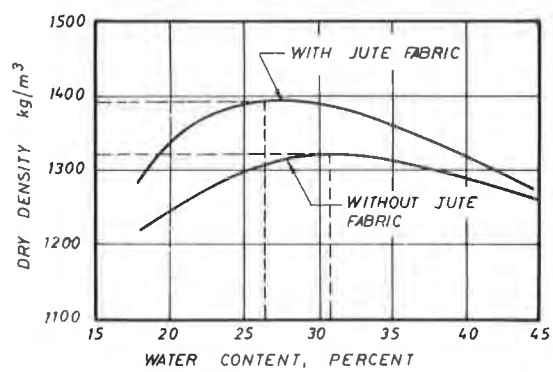

Fig. 2 Effect of Jute Fabric on Compaction Characteristics of Subgrade Soil

The unconfined compression tests were carried out on samples compacted to 100 mm diameter and 200 mm long at the standard proctor compactive effort using optimum moisture content of 25%. For these tests, two layers of jute fabric were introduced within the sample at equal intervals. The CBR tests were carried out compacting the samples in the standard CBR mould at the same moisture content as above. For these tests with jute fabric, two layers were interposed within the samples at equal intervals while compacting. Tables 2 and 3 respectively show the influence of fabric on the unconfined compressive strength and CBR values of samples compacted in the laboratory.

Table 2. Effect of Jute Fabric on Unconfined Compressive Strength

Water content, %		25	30	35
Unconfined Compressive Strength, kN/m^2	without fabric	110	45	36
	with fabric	300	115	65
% strain at failure	without fabric	8	10	22
	with fabric	26	30	42

Table 3. Effect of Jute Fabric on CBR Value

Water content, %		20	25	30	35
CBR Value %	without fabric	5.0	4.7	3.5	2.6
	with fabric	8.0	6.8	5.2	4.5

The laboratory test results conclusively show that the stress-strength characteristics of the soil are better with the jute fabric than without it.

5 FIELD TESTS

Since the laboratory tests gave only a qualitative indication of the beneficial effect of jute fabric, plate load tests were conducted to evaluate the insitu behaviour of the subgrade. The subgrade soil was soft to medium silty clay of natural water content of 35% and vane shear strength (insitu) of 20 kN/m^2. Plates of 300 mm diameter were loaded keeping them directly on the surface of the uncompacted subgrade in the first series and on the surface of jute fabric spread over the subgrade in the second series. The average results are shown in Fig. 3.

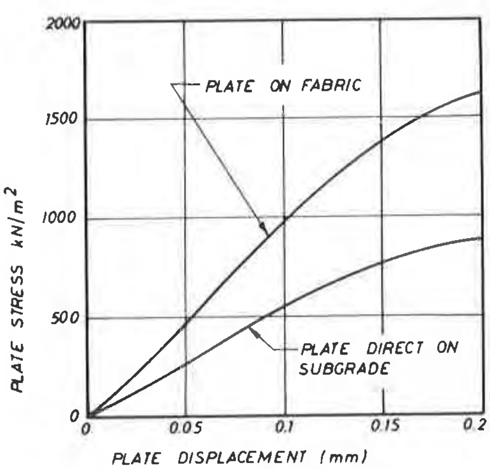

Fig. 3 Effect of Jute Fabric on Bearing Capacity of Subgrade Soil

The plate load tests confirmed that the jute fabric significantly improves the bearing capacity and settlement behaviour of the subgrade soil. The results of tests carried out using jute fabric were in tune with similar tests reported (7) using synthetic fabric.

A separate study of model footing on foundation soil reinforced with jute strips being conducted at the Department of Civil Engineering, National University of Singapore, has also confirmed that reinforcing the subgrade significantly helps in distributing the load over a wider area and thus resulting in enhanced bearing capacity and reduced settlement. In the case of a road, this would result in a reduced overall thickness of road structure and less earthwrok when jute fabric is used as an intervening membrane between the subgrade and the overlying thin granular sub-base layer.

6 JUTE FABRIC DRAIN

A type of flexible drain known as fibredrain made out of jute fabric suitable for consolidation of soft compressible soils under highway embankments etc., has been developed and field tested by the staff of the Department fo Civil Engineering, National University of Singapore. The fibredrain is patented in the United Kingdom. The drain consists of a rectangular strip of 100 mm width and 4 mm thick obtainable in the form of rolls in lengths of 200 m to 300 m. The drain strip itself consists of two fibre strands of 2 mm to 3 mm diameter laid longitudinally parallel to each other and enveloped by one or two layers of jute fabric held together by longitudinal stitches. Such drains can be inserted into the ground vertically at spacings of 1 m to 2 m in a triangular or square grid pattern and the ground surcharged by a preload fill of permeable soil. The rate of consolidation depends upon the type of soil being treated, the drain spacing and the surcharge load. Field tests have indicated (8) satisfactory performance of these drains with respect to drainage, non-clogging and non-deterioration within a period of 2 years of observation. Soft compressible marine clay upto 20 m depth was able to be consolidated to increase the shear strength from 15 kN/m^2 to 50 kN/m^2. These jute fibredrains besides being used for consolidating deep seated compressible soils under highway embankments can also be effectively used for vertical drainage to relieve hydrostatic pressure beind the retaining walls which occur frequently along roads in hilly terrain.

7 CONCLUSIONS

The jute fabric has the potential of being used to serve as a filter fabric as well as a fabric reinforcement to stabilize and protect weak subgrades in road construction. When the jute fabric is palced directly on the subgrade and topped with a granular backfill to form a sub-base for the pavement, it is found to function in a three fold way: (i) it separates the subgrade from sub-base thus preventing the punching of the base material into the subgrade and at the same time the fines from the subgrade are also prevented from gaining entry into the road structure, (ii) it acts as a drainage layer to remove excess water from softening the subgrade, and (iii) it helps to improve the bearing capacity and settlement behaviour of the subgrade by virtue of its action as a fabric reinforcement.

The jute fabric is expected to contribute towards better road performance by reducing road defects with the consequent reduction in maintenance costs. The economy resulting in reduced road thickness design and construction time is an added bonus.

While the jute fabric as a geotextile appears to function quite close to synthetic non-woven fabrics in performance, its durability aspect seems to pose a serious limitation on its use. However, jute fabric is founded to be fairly resistant to deterioration when embedded in wet soil under a narrow margin of annual variation in subgrade water content (18% to 30%) and subgrade temperature (25°C to 30°C) conditions prevailing in the geographical region of Singapore and Malaysia. There is little doubt that the jute fabric is initially very stong and ideal for use as a geotextile material. After it is placed on the weak subgrade, the subgrade stiffens and becomes stronger on consolidation within about a year or so under the action of granular subbase surcharge, self weight of pavement, construction rolling and traffic loads. The jute fabric immensely helps in this rapid subgrade strengthening process in combination with the drainage layer above it. With time, the subgrade becomes less and less dependent on the fabric for its stability and therefore, the long term durability aspect of jute fabric should not deter its use as a geotextile for subgrade stabilization in road construction.

The jute fabric is useful for developing countries of Asia and South East Asia as a money saver as well as a construction expedient. The advantages resulting by its use will more than outweigh the cost of the material and laying. Being in the vicinity of jute producing countries, the developing countries of Asia and South East Asia can harness the benefits of jute fabric especially for subgrade stabilization in road construction. For these countries, the jute fabric could serve as an economical alternative to the imported versions.

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A Full-Scale Experiment on Granular and Bituminous Road Pavements Laid on Fabrics

Expérience à grande échelle avec couches de forme granulaires et bitumineux sur textiles

Granular pavements of two thicknesses were laid on each of five fabrics over a clay subgrade, with two control sections. One fabric section and one control were overlaid with dense bitumen macadam. Surface deformations, strains in all layers and vertical stress in the subgrade were measured under wheel loads up to 5300 kg. Fabric had no effect on the performance of the macadam-surfaced pavement. In the granular pavements permeable fabrics caused reductions of permanent surface deformation and permanent subgrade strain. Transient subgrade strain was only reduced in the transverse direction. Fabrics were torn at surface deformations of 90 - 110 mm. Permanent transverse tensile strain of 1 - 2% developed in the fabrics under the wheel ruts.

Au dessus d'un sol support argileux, on a construit des couches de forme granulaires de deux epaisseurs sur chacun de cinq textiles différents. On a construit aussi deux éléments experimentals de contrôle. On a couvert de macadam de bitume un des éléments avec textile et un des éléments de contrôle. On a mesuré les déformations de surface, les déformations de tous les matériaux et les contraintes verticals dans le sol support, à cause de circulation de charge maximum 5300 kg. Le textile n'a changé pas du tout le fonctionnement de l'élément couvert de bitume. Les textiles perméables ont réduit les déformations permanents de la surfacè et du sol support des éléments granulaires. Ils ont réduit le déformation dynamique du sol support, main seulement dans le direction transversale. Les textiles se sont déchirés lorsque la surface de la route s'était déplacée par 90 - 110 mm vertical. Sous les lignes de circulation des roues les textiles ont supporté des tensions transversales permanentes de 1 - 2%.

INTRODUCTION AND PREVIOUS WORK

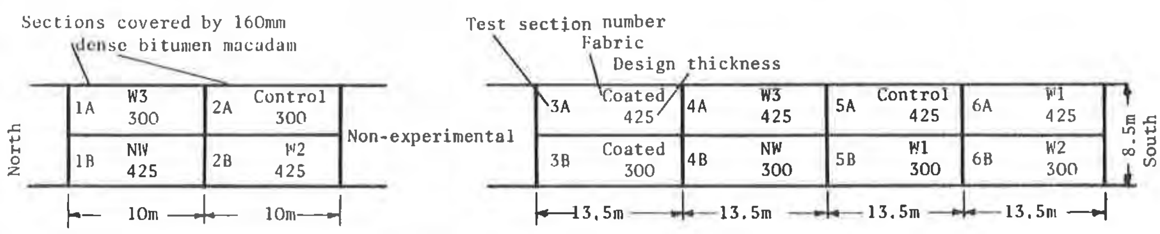
A full-scale experiment was planned to investigate the effects of inclusion of fabric at the subgrade/sub-base interface in granular and bituminous-surfaced pavements. In previous experiments (1,2,3) on granular pavements laid on clays with CBR's from 0.7 to 12%, the permanent surface deformation had been reduced by the presence of fabrics and meshes of differing properties, but only one experiment had sought any understanding of the mechanism of improvement. This was a pilot full-scale experiment by the Transport and Road Research Laboratory in 1977.(3) The pavement was contained within a rigid tank and thus restrained against overall lateral spread. The permanent vertical strain under the wheel-path, both in the subgrade and in the bottom of the sub-base, was reduced significantly by the presence of a melt-bonded fabric. Transient vertical strain in the subgrade was not altered but transient horizontal strains, both longitudinal and transverse, were reduced significantly.

DESCRIPTION OF EXPERIMENT

Site. An experimental road was constructed on a level site at Sandleheath in Hampshire. The subgrade was London Clay, with mean properties as follows:

Liquid limit	51%
Plastic limit	23%
Natural moisture content	28%

Depth below formation (mm)	CBR % (measured with cone penetrometer by Black's method) (4)
0	0.7
75	2.4
150	2.8
225	3.0
300	3.5



1. Layout plan

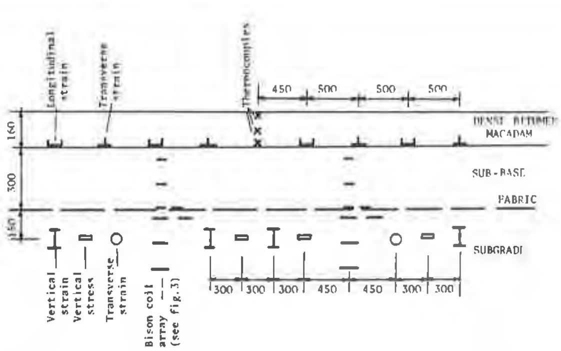
Table 1. Fabrics

Fabric designation	Description	Direction of tests	Ultimate[3] strength kN/m width	Extension[3] at failure %	Supplier
W1[1]	Woven polypropylene tape 96 x 44 per 10 cm	Longitudinal (warp) Transverse (weft)	19.0 13.3	19.0 11.2	Synthetic Fabrics (Scotland) Ltd
W2[1]	Woven polypropylene tape 59 x 37 per 10 cm	Longitudinal (warp) Transverse (weft)	56.6 38.6	13.7 9.2	Low Brothers & Co
W3[1]	Woven multi-filament polyester 112 x 106 per 10 cm	Longitudinal (warp) Transverse (weft)	82.8 84.4	14.7 14.8	ICI Fibres Ltd
Coated[1]	Neoprene-coated nylon balanced weave	Longitudinal Transverse	60.4 42.8	25.0 23.0	MVEE
NW[2]	Melt-bonded polypropylene/ polyethylene	Random	8.1	50.5	ICI Fibres Ltd

NOTES
1. Tested by standard strip tests
2. Tested by ICI plane strain method
3. All test results are means of four

Table 2. Instrumentation.

Test Section No	Design thickness (mm)	Fabric	Measurements
1A	300	W3	Transient stress & strain in the soil, permanent strain in the soil and in the fabric, transient strain and temperature in dense bitumen macadam.
2A	300	None	Transient stress & strain in the soil, permanent strain in the soil, transient strain and temperature in dense bitumen macadam.
4A	425	W3	Transient stress in soil, transient & permanent strain in soil, fabric and granular layer.
5A	425	None	Transient stress in soil, transient & permanent strain in soil and granular layer.
6A	425	W1	Transient & permanent strain in soil, fabric & granular layer. (Bison gauges only in this section)



2.Typical layout of instruments. Longitudinal section along wheel path.

Materials and Construction. The experiment included three woven fabrics of different properties, one non-woven and one impermeable coated fabric. Details of the fabrics are given in Table 1.

Two thicknesses of granular material were used, designed to be 300mm and 425mm after compaction. The material was crushed granite conforming to the Department of Transport (DTp) Type 1 sub-base specification.(5) The 300mm thickness was obtained by spreading in a single layer and compacting by Vibroll tandem vibrating roller to comply with the DTp Specification, Clause 802.(5) For the 425mm thickness a second layer was spread and compacted with the same scheme of passes. The mean dry density achieved was 2.17 Mg/m^3 and mean moisture content 5.39%.

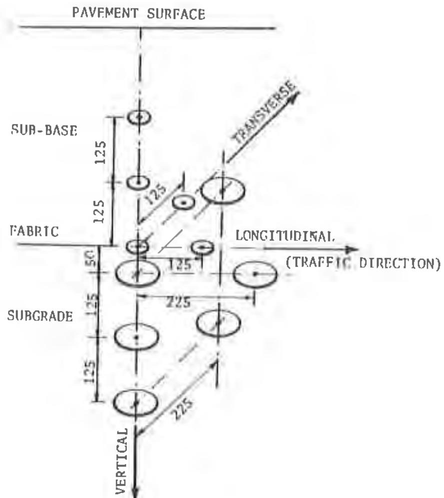
Two of the 300mm thick sections were overlaid with dense bitumen macadam basecourse (5) to a nominal thickness of 160mm. Compaction was by an 8 tonne, three-wheeled, steel-tyred roller to the DTp specification.

Layout and Measurements. A plan of the experiment is shown in Fig 1. Each fabric was used in two test sections, one of 300mm design thickness and the other of 425mm, and there was one control section of each thickness, containing no fabric. The two 300mm sections overlaid with dense bitumen macadam were the control and the W3 fabric section; they were overlaid after very brief trafficking. Instruments for measurement of stress, strain and temperature were installed in five sections, as listed in Table 2 and shown in Fig 2 and Fig 3. The vertical surface deformation was measured at intervals of 300mm across four cross-sections in each pavement throughout the experiment.

Trafficking. For early trafficking on the sub-base, two-axle lorries with dual rear wheels were used. Axle loads were increased in steps to a maximum of 9940kg. For final trafficking, these lorries were replaced by a three-axle dump-truck with single wheels and coarse cross-country tyres on both rear axles, with a maximum axle load of 9460kg.

The line of travel was varied randomly within a 800mm wide path for each wheel, but when stresses and strains were being measured wheels were driven in the middle of the wheel path directly over the gauges.

As the sequence of traffic varied betweeen test sections, a unified scale of trafficking was adopted, the unit of the scale being one pass of the 9940kg axle. This is the scale of 'equivalent axles' used in the results given below. The equivalence factor for every other axle was the ratio of the mean transient vertical strain measured at 150mm depth in the subgrade of sections 4A and 5A



3. Typical array of inductive (Bison) coils.

under the axle to the mean strain measured under the 9940kg axle. This procedure took into account the axle load and the effect of the different tyres and wheel systems.

The dense bituminous macadam sections were trafficked by the two-axle lorry with dual wheels initially carrying a rear axle load of 9940kg. Because no measurable change in road behaviour was recorded after 4600 vehicle passes, the rear axle load was increased to 13600kg and a further 7700 passes were applied.

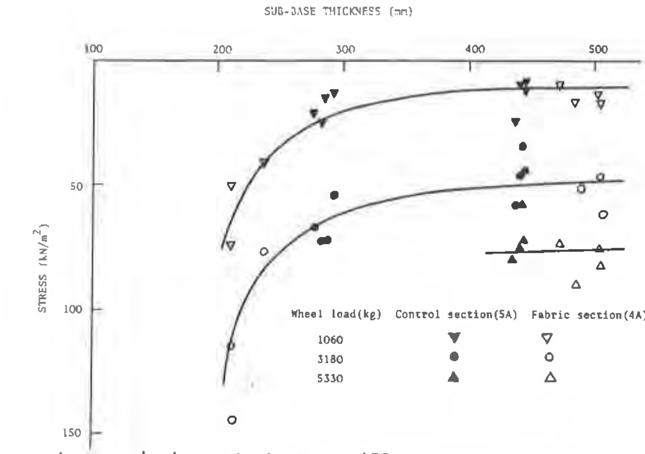
RESULTS

The stresses, strains and deformations described below were measured during and at the end of trafficking. With two exceptions it was possible to discontinue traffic on each granular pavement as soon as it failed.

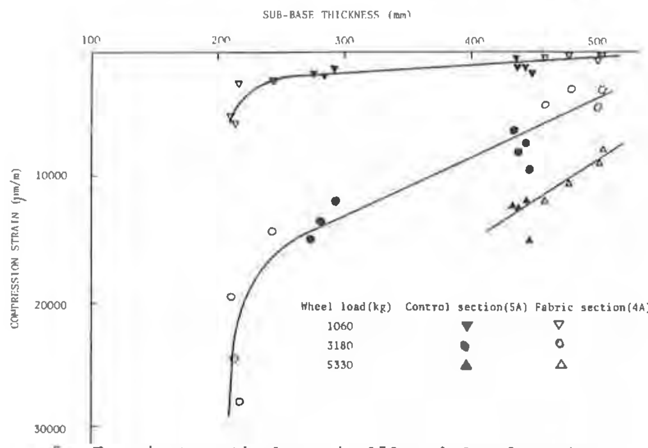
The results from individual stress and strain gauges and from the four cross-sections where surface deformation was measured are referenced by their distances in m from the north end of the experimental section.

Transient Stress and Strain. Many thousands of readings of transient stress and strain were made on the TRRL stress and strain gauges under all vehicle axles. In general, after each increase of axle load, the peak values of vertical stress, vertical strain and transverse strain increased with vehicle passes until constant levels were achieved. The number of vehicle passes to reach these new levels, however, varied from about twenty to several hundred. The peak values obtained in the four instrumented sections (before overlaying sections 1A and 2A with dense bitumen macadam), are plotted against the pavement thickness at each gauge in Figs 4, 5 and 6. It can be seen that thickness has a greater influence on both vertical stress and strain than the presence of fabric and no influence of fabric can be inferred. Figure 6, however, shows that the heavier wheel loads generate considerably less transverse strain in the sections with fabric.

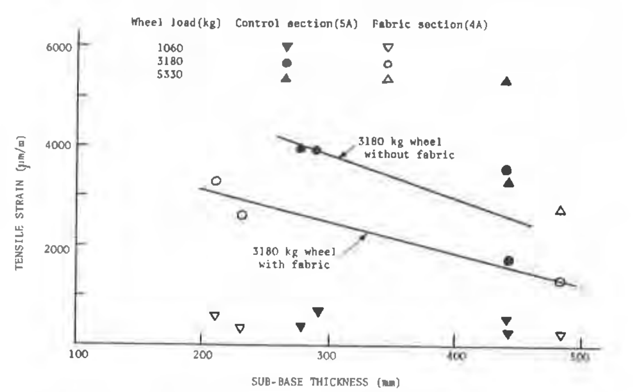
Similar measurements were made on the dense bitumen macadam sections (1A and 2A) and these showed also that the presence of fabric made no difference to the transient vertical stress and strain in the soil and to the transverse and longitudinal strains at the bottom of the bituminous macadam. There was, however some evidence to suggest that the transient transverse strain in the soil was reduced slightly under the fabric.



4. Transient vertical stress 150mm below formation.



5. Transient vertical strain 150 mm below formation.



6. Transient transverse strain 150 mm below formation.

A good general indicator of structural condition is the transient deflection (6) under a slowly moving lorry wheel. Deflections were measured frequently at eight positions on sections 1A and 2A and were found not to be affected by the fabric.

Permanent Strain. The most marked effect of fabric in the unsurfaced sections was the reduction of transverse strain at the subgrade/sub-base interface (Fig 7) and at

50mm below it (Fig 8). In Figure 8 the curve for 6A (3.0m) represents a location where the fabric survived untorn to the end of trafficking and 6A (6.0m) a point where large tears occurred during trafficking. The strain at 6A (6.0m) increased continuously throughout the trafficking, as in control section 5A, but the pattern at 6A (3.0m) of a reducing rate of increase of strain was similar to that observed in section 4A with strong fabric. It is inferred that the presence of either fabric, W1 or W3, caused a large reduction of transverse strain so long as it remained untorn.

Similar differences were observed at 150mm below the interface in sections 4A and 5A where the transverse strains were approximately 1.5% and 7% respectively at the end of trafficking.

In the two sections overlaid with dense bitumen macadam the permanent transverse strain in the soil was less than 0.02% at the two measurement locations in the section with fabric and at one location in the control section. The other gauge in the control section had a thinner cover of bituminous material and indicated a strain of 0.2%.

The permanent vertical strains in the subgrade of section 5A were also greater than in section 4A but the curves for 5A were tending, at the end of trafficking, towards constant values, unlike the transverse strain curves for 5A (Fig 8). After considering variations in sub-base thickness and other factors,(6) it was concluded that in unsurfaced pavements the presence of untorn fabric causes a substantial reduction in permanent vertical strain in the subgrade to at least 150mm below the interface. This effect was not found in test section 6A at (6.0m) where the fabric was torn.

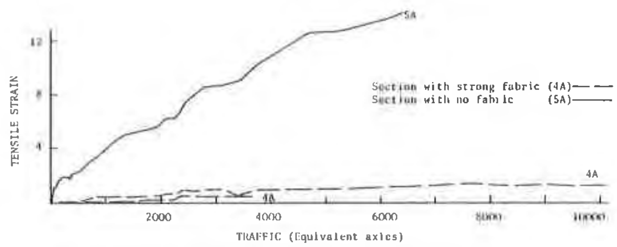
The measurements of permanent strain at a single array of coils are not subject to the effects of variation in pavement thickness or material properties and they thus provide a reliable picture of the distribution of strain in the cross-section. The permanent strains in the cross-section through the wheel path at each array of Bison coils when the vertical surface deformation was 50mm are plotted in Fig 9. The strains in the subgrade were smaller at the arrays with untorn fabric, viz. the arrays in unit 4A and at 6A (3.0m). It is inferred that subgrade deformation at these arrays was distributed through a larger area of the cross-section than at the other three arrays, since the maximum surface displacement was the same at all six positions. This occurred however without reducing the peak values of transient vertical stress and strain (Figs 4 and 5). Even at surface deformations of only 30mm there was a pronounced difference of permanent strains between sections with and without fabric, although not so large a difference as at 50mm (Fig 9).

The wider spread of permanent strain in the cross-section coincides with longer life of the pavements. The number of equivalent axles taken to reach a surface deformation of 50mm at each of the Bison arrays was:

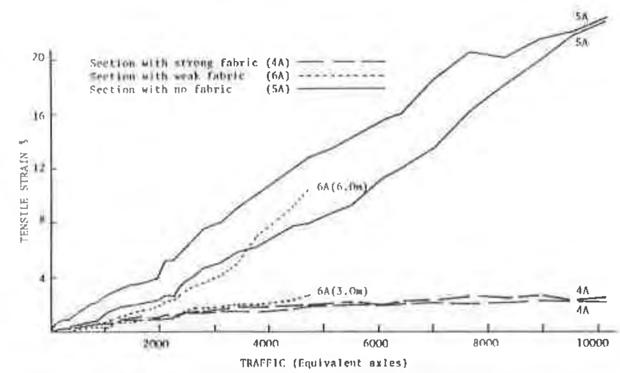
Section 4A,	both arrays	1875	equivalent	axles
Section 5A	(4.0m)	1460	"	"
	(5.8m)	1180	"	"
Section 6A	(3.0m)(fabric untorn)	1830	"	"
	(6.0m)(fabric torn)	1180	"	"

Section 6A suffered much more rapid deformation than 4A after passing 50mm deformation. At that stage, according to concepts discussed in the next section of the paper, it was failing.

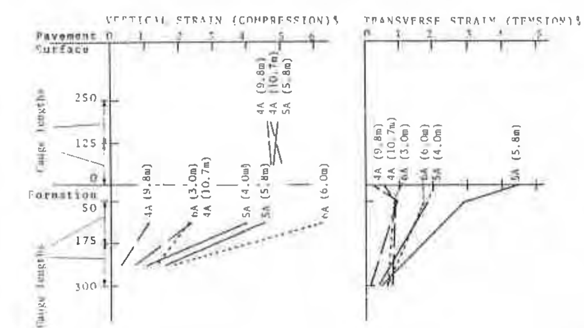
In the sections covered with dense bitumen macadam the



7. Permanent transverse strain at formation.



8. Permanent transverse strain 50mm below formation.

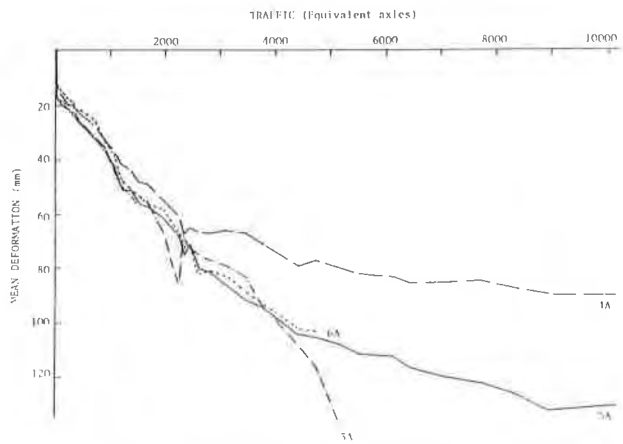


9. Permanent strains at surface deformation of 50mm.

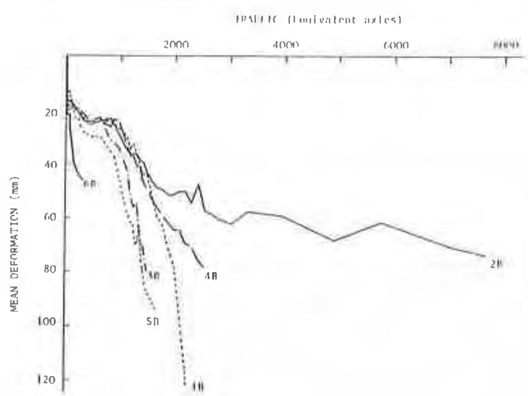
permanent vertical strains in the soil were not influenced by the presence of fabric but they did depend on the thickness of the bituminous macadam. At the end of trafficking the permanent strain was only 0.2% under 160mm of dense bitumen macadam.

Permanent Surface Deformation and Pavement Failures. The development of mean surface deformation measured in the outside wheel paths of all the unsurfaced sections are plotted in Figs 10 and 11. Curves of decreasing slope represent pavements adequate for the wheel loads in use, while curves of constant or increasing slope represent failing pavements. It is clear that all the B sections except 2B failed after the passage of moderate numbers of equivalent axles. Sections 3A and 6A also suffered failure in the outside wheel paths, although the mean deformation curve for 6A does not show it. (The final failures in all sections were concentrated in short lengths, with local deformations more rapid than is shown by the mean curves). The trigger to failure in most of the B sections was the start of trafficking by the three-axle dump-truck, with its maximum axle load similar to the previous lorry but on single wheels with coarse-treaded tyres.

The permanent deformation of the dense bitumen macadam



10. Mean surface deformations of A sections.



11. Mean surface deformation of B sections.

sections depended on the thickness of the bituminous layer and was not reduced by the fabric. For a thickness of 160mm of dense bitumen macadam the surface deformation was 7mm at the end of trafficking.

On completion of trafficking two trenches were dug across each section. One at the cross-section showing maximum deformation and the other where deformation was a minimum. At many places the fabrics were found to be torn in the wheel paths. At all of these, the deformation at the end of trafficking was increasing rapidly, in the way identified in Figs 10 and 11 as a failure mode; but at several locations where the deformation curve was in the failure mode the fabric was not torn, showing that failure may begin before the fabric tears. General observations suggested that this was the case in all the sections that failed.

Although torn fabric was found at one place where the vertical surface deformation was only 55mm, the general evidence was that in sections with sub-base thickness less than 400mm the weaker fabrics were torn if the surface deformation exceeded 90mm, but in sections of thickness 400-470mm they were only torn if the deformation exceeded 110mm.

The influence of the properties of the fabrics on the deformation curves had to be isolated carefully from effects of differing sub-base thicknesses, subgrade properties, etc.(7) The most important observation was that neither of the strong permeable fabrics (W2 and W3) was torn except in the premature failure of section 6B, caused by excessive local subgrade weakness and a thin granular layer. Sections of adequate thickness with these fabrics (2B and 4A) did not fail and underwent less deformation than the control section (5A) which also survived to the end of trafficking without failure. Weaker permeable fabrics were torn during failure of their overlying pavements. The failure of 6A (containing fabric W1) before 5A is attributed to its weaker subgrade, but is evidence that such fabric will not prevent failure of a pavement inadequate for the ruling conditions of subgrade and traffic.

Sections 3A and 3B containing the coated fabric, which was also of high strength but impermeable and of smooth surface, underwent deformations similar to those of comparable sections with weak fabrics. This was attributed to slip between the fabric and the clay as the excavations made on completion of trafficking showed the contact to be wet and slippery. Vane shear tests indicated a sharp reduction of strength at the surface of the subgrade. In contrast, the subgrade strength in the wheel paths directly under permeable fabrics almost doubled during trafficking.(7)

12. View of fabric in trench across section 4A.

The deformation of some of the unsurfaced pavements varied with the distance of the wheel path from the unrestrained pavement edge. Comparisons between the development of deformation near the edge and near the centre of the road showed that the rate of deformation was increased if the middle of the wheel path was less than 1.4m from the edge of the running surface.

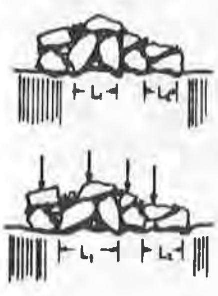
Final Condition of Fabric and Sub-base. The separation of the sub-base from the subgrade was effectively maintained by all the fabrics, only a slight coloration from the subgrade showing in the bottom of the sub-base over permeable fabrics. In the control section the sub-base and subgrade were intermixed to a depth of about 50mm over the whole section.

In most of the trenches across the pavements the fabrics, if not already torn, were stretched taut across the wheel paths. Elsewhere, both between the wheel paths and outside them, wrinkles were observed parallel to the wheel paths but at varying distances from them (Fig 12). The deformation of the subgrade/sub-base interface in the wheel paths was enough only to stretch the fabric by about 1 or 2%, but the strain necessary to cause rupture of any of the fabrics would be more than 10%. The largest permanent tensile strain measured in an untorn fabric was also less than 2% (see Fig 7). An explanation of these observations is offered in the discussion below.

When the sections overlaid with dense bitumen macadam were excavated, fabric W3 on section 1A was untorn. Moreover no measurable deformation had occurred at the top of the soil and hence no stretching of the fabric was observed.

DISCUSSION

The formation of a rut in the granular pavements under repeated loading involves incremental plastic deformations in both sub-base and subgrade. Deformation of the sub-base proceeds by non-reversible rearrangements of particles, in which the relative movement of individual particles may exceed the average strain measured over a greater length. Figure 13 shows how general deformation of the sub-base can induce large strains in small areas of fabric. The lengths labelled L1 and L2 are both strained much more than the average measured over a typical gauge length because the large particles bite into the fabric and move it with them. If the local strain in the fabric rises to the rupture strain, a tear is initiated; the existence of the tear reduces horizontal restraint on the sub-base, and under repeated loading allows greater deformation and progressive extension of the tear.



1. Initial stable arrangement of sub-base particles.
2. Downward load forces particles into a new arrangement. When the load is removed the particles will remain in the new arrangement. Because the particles bite into the fabric there has been a large percentage extension of the lengths L_1 and L_2.

13. Local extension of fabric by sub-base deformation.

In observations of the movements of cracks during trafficking it appeared that portions of the sub-base bounded by cracks might be rocking on the subgrade as rigid bodies. It is suggested that in this rocking motion fabric can be fed gradually away from the wheel paths to form the longitudinal wrinkles seen on either side in the excavations.

The wrinkles between the wheel paths showed that the fabric was not in tension and this, on a saturated clay subgrade, is contrary to an important assumption of Giroud and Noiray's design method.(8) Some measurements of subgrade strain made just outside the wheel paths in this experiment, suggest that the angle of effective load distribution within the granular layer is smaller than Giroud and Noiray assumed. In addition this experiment has shown that permanent tensile strain can exist in fabric across a rut, a feature not allowed for in Giroud and Noiray's assumptions. This applies a transverse prestress to the loaded area, which must be beneficial. How large the permanent strain can become without risk of rupture remains to be determined. The tensile strain and prestress will occur more readily in pavements where traffic is confined to relatively narrow wheel paths.

CONCLUSIONS

In granular pavements the presence of a permeable fabric reduces the rate of surface deformation provided that the fabric is not torn. The fabric reduces permanent vertical strains in the granular layer and in the soil although transient vertical stress and strain in the soil are not changed. Transient and permanent horizontal strains in the soil are reduced by the presence of fabric.

The structural behaviour of the pavements containing bituminous bound layers was not improved by the presence of fabric.

All the fabrics used were effective as separating layers between the clay subgrade and well graded sub-base material.

ACKNOWLEDGEMENTS

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Design of a Temporary Road Structure with the Use of a Textile Membrane

Conception d'une piste avec membrane textile

Field and laboratory studies have shown that a textile membrane has a considerable influence on a stressed-strained state of the system "soft soil-granular" and allowed to propose design schemes of the system for wheel load, these schemes being adopted depending upon specific properties of the soft soil.

The design schemes take into account the formation and propagation of ruts on the soft soil due to consolidation high compressible soils (peat soils) or due squeezing out low compressible soft soils. Possible decrease in the strength of some soils under the action of repeated loadings is also taken into consideration.

Les essais en laboratoires et in situ ont montré que la membrane textile influe d'une manière importante sur l'état contrainte-déformation du système sol mou- matériau granulaire et ont permis de proposer des schémas de calcul par rapport aux charges de roue prises en considération selon les particularités des propriétés du sol mou.

Au dimensionnement des schémas ont tient compte des conditions de formation et de développement des ornières en sol mou du fait de la consolidation des sols très compressibles (la tourbe) ou de l'extrusion des sols mous minéraux peu compressibles. On prend en considération également la possibilité de réduction de la résistance des sols sous l'influence des charges répétées.

The use of synthetic materials with low moduli holds the greatest promise for construction of various types of access and temporary roads over soft soils. These roads are of special importance for economic development of highly swamped areas in West Siberia.

A problem of temporary road design is in practice a design for minimum thickness. A structure of the temporary road consists of a fill-up soil layer of h_f thickness laid over geotextile spread on the surface of weak soil mass.

When designing a road structure of such type the principal aim is to determine a required minimum thickness of the fill-up soil layer h_f in order to provide the passage of vehicles over the soil. Here the designation of the textile membrane is to allow the construction of embankment of minimum thickness (0.3 to 0.8m).

The following considerations may be used as the basis for minimum thickness design.

If the material of fill-up layer has a sufficient strength and low compressibility at given loading parameters, then the successful performance of the structure considered will be dependent only on conditions of the weak soil performance at the bottom of fill-up layer. In this connection, of interest are, first of all, two possible processes which are as follows:

- lowering the strength of weak soil for the reason that the tangential stresses due to the external loading exceed the shear resistance of the weak soil;
- development of the process of weak soil consolidation under the action of compressive stresses.

As the result of the first process there occurs more or less gradual lateral squeezing-out of the weak soil to the outside from under the loaded area. It is evident that with increase in the number of wheel passages a rut will appear, which may be referred to as a rut of squeezing-out.

As the result of the second process a rut of consolidation, which is of different nature, may develop. The latter is characteristic of the high compressible soils (peats, etc.).

It is obvious that for adequate performance of a structure the depth of the rut of either nature should not exceed some allowable value taking into account the condition of vehicle passage.

If at the boundary of the fill-up layer and weak soil mass there is a membrane of the tension-resistant material, the behaviour of the structure will change, i.e. under certain conditions the membrane will act so as to restrain development of the rut of either nature.

Strain pattern analysis of the structure based on observation on models with transparent membrane has allowed to propose as a first approximation a design scheme to estimate quantitively the effect of textile membrane.

The main point of the scheme is as follows (Fig.1).

When a load P_o is applied to the surface of structure at the level of the membrane location there appear normal stresses P_z which are uniformly distributed over the area of a circle of diameter $D_1 = D_o \sqrt{k}$ where k is a coefficient of stress distribution taken in accordance with the known solutions of the theory of linearly deformed media.

Under the action of these stresses there occurs settlement of the area AB by value S. Due to high normal stresses and, hence, high friction forces there is no sliding of the membrane at this section, i.e. there is joint movement of the soil and geotextile, and the latter is not involved in work.

At the same time outside of the area AB the friction forces initially are conditioned only by stresses from the dead weight of fill-up layer (q) and therefore they are small. Because of this at some length l_d sliding of the membrane in soil occurs, and this results in the membrane tension.

Force Q applied to the membrane at point b resolves into component P tending to stretch the membrane and component N (manifested in friction) which will hinder tension as well as the friction forces due to dead weight q will.

The value l_d may be found from the condition of shear equilibrium:

$$P = N f_{1.s} + 2 q l p f_{av.} \tag{1}$$

where $f_{1.s.}$ = friction coefficient between the lower surface of membrane and the soil;

$f_{av.}$ = average friction coefficient between the soil and the upper or lower surface of membrane.

Taking into account that an average tensile force in membrane is

$$P_{av.} = \frac{P}{2} \tag{2}$$

and hence the tension of the membrane is

$$\Delta = \frac{P}{2E_{fab}} \cdot l_d \tag{3}$$

as well as geometric relation between $l_{d.}$ and $+l_d$ it is not difficult to obtain formula connecting value P and settlement S:

$$S = \frac{P\left[\sqrt{\frac{P_{tex}}{2E_{tex}}\left(\frac{P}{2E_{tex}}+2\right)} - f_{e.s.}\right]}{2g \cdot f_{AV}} \tag{4}$$

where $E_{tex.}$ = geotextile tensile modulus (N/cm). Then considering relation between P and Q and replacing the unit force Q (per unit of membrane operational width) by the stamp unit load, an expression may be obtained to determine the value of stamp load (P_{tex}) which is compensated by forces appearing in the geotextile at tension:

$$P_{tex} = \frac{4P}{D_o \sqrt{K}} \sqrt{1 + \frac{1}{\frac{P}{2E_{tex}}\left(\frac{P}{2E_{tex}} + 2\right)}} \tag{5}$$

Formulae (4) and (5) allow to establish relation between $P_{tex.}$ and settlement S. In practice these relationships may be used by means of monograms the example of which is given in Fig. 2. At specified $E_{tex.}$, $f_{av.}$, $f_{1.s.}$, q, D_o and a specified (allowable) value of settlement S the monogram permits to determine the magnitude of that portion of the load on stamp that is accepted by the geotextile due to its tension and friction against the soil.

Thus, as it follows from the above, the designation of the textile membrane is in that it removes a port of stresses arising from the external loading and acting at the surface of weak soil mass, thus reducing the stresses by value of $P_{tex.}$

With the above in mind, structural design may be done by the following way:

1. Design in respect of squeezing-out rut development is reduced to checking the condition:

$$P_z^o \leq P_{cr} + K P_{tex} \tag{6}$$

where P_z^o = vertical normal stresses from the external load and the dead weight of ill-up soil acting at the level of membrane location;
P_{cr} = critical load on weak soil.
Value $P_{cr.}$ can be determined by means of solutions according to the limit equilibrium theory using the relationship of the following kind:

$$P_{cr.} = M_1 D_1 \gamma_w + M_2 \gamma_f h_f + M_3 C \tag{7}$$

where M_1, M_2, and M_3 = functions of the angle of internal friction for weak soil;
γ_w and C = unit weight and cohesion of weak soil;
γ_f and h_f = unit weight and depth of fill-up layer
Value $P_{tex.}$ is evaluated from Eqs. (4) and (5) at S = $S_{allow.}$

2. Design in respect of consolidation rut development, is reduced to the check of the condition:

$$S_{des.} \leq S_{allow.} \tag{8}$$

A design depth of the consolidation rut may be estimated from the relationship:

$$S_{des} = \frac{P_z \cdot D_1 \cdot U_c}{E_w} = \frac{P_o \cdot D_o \sqrt{K} \cdot U_c}{E_w} \tag{9}$$

where: P_z, D_1, P_o, D_o = values according to Fig. 1;
E_w = modulus of deformation of weak soil;
U_c = coefficient less than one taking into account an actual degree of consolidation that may be reached during structure life (For peats it is often assumed to be 0.6).

In some cases a depth of the rut due to squeezing out the plastic soil from under the wheel should be well below an allowable value according to the condition of vehicle passage. This is the case with soils in which the squeezing-out leads to a sharp drop of strength and progressive rutting as well as with temporary roads when the requirements to carriageway evenness are increased.

As practice and experiment (1) show, when placed at the contact of granular material and weak soil, geotextile, even having not high geotextile tensile modulus (uniaxial tension) - of the order of 10 kH/m, can substantially ffect rut development under the multiple load application (Fig. 3). This is explained by improving the conditions of weak soil performance near the contact surface with fill-up material. Stabilizing effect of the geotextile membrane can be represented, as a first approximation, in terms of reduction of of tangential stresses in the weak soil due to provision of a smooth contact in the two-layered semi-space loaded with circular stamp.

The condition of limit equilibrium in soil as determined under the centre of stamp, is as follows:

$$\frac{1}{2\cos\varphi_w}\left[(\sigma_1-\sigma_3)-(\sigma_1+\sigma_3)\sin\varphi_w\right] \le C_w \qquad (10)$$

where: σ_1 and σ_3 = the highest and lowest principal normal stresses φ_w and C_w - angle of internal friction and cohesion of weak soil.

This condition is checked for a point situated under the stamp centre; it may be written as

$$\tau_w - \tau_g \le \frac{1}{K_N} C_w \qquad (11)$$

where: τ_w = active (not compensated by friction) part of tanential stresses in weak soil caused by temporary load;

τ_g = correction to shear stresses considering the structure dead weight; stresses induced by the latter are distributed according to hydrostatic law;

K_N = 1 + lgN = factor taking into account an increase in severity of load action under N repetitive applications

Values τ_w and τ_g may be found from the chart in Fig. 4 based on solutions according to the theory of elasticity for two-layered system with smooth contact between the layers.

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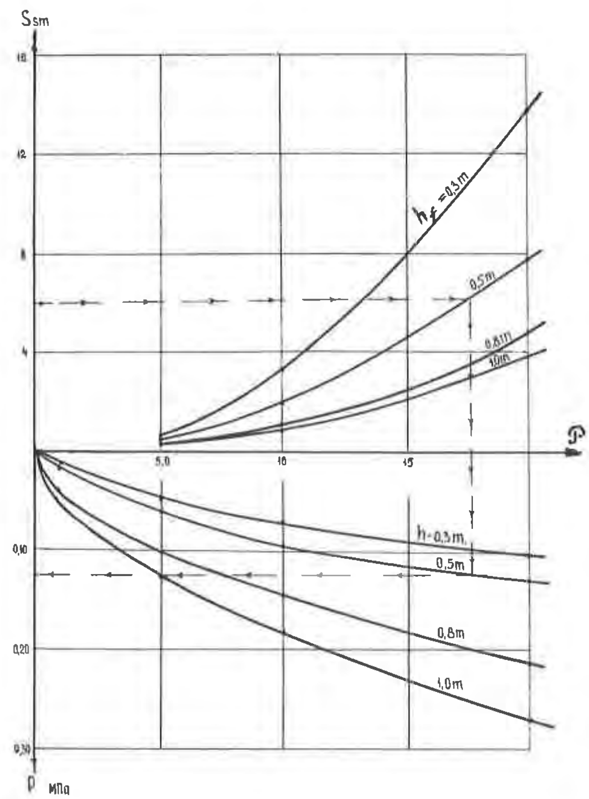

Fig. 2 Nomogram for determination of value $P_{tex.}$ at specified rut depth in weak soil (S) and various thicknesses of fill-up layer (h_f)

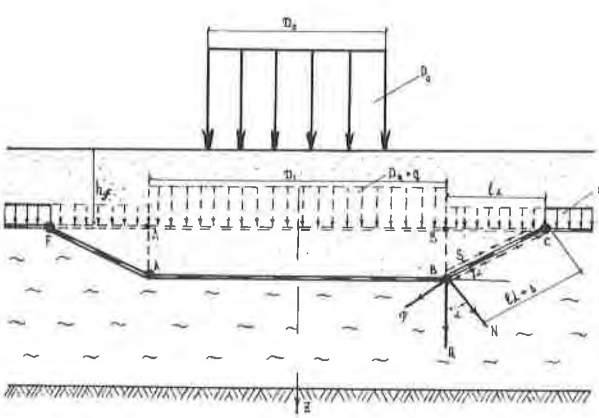

Fig. 1 Scheme of membrane performance in thin-layered structure of temporary road.

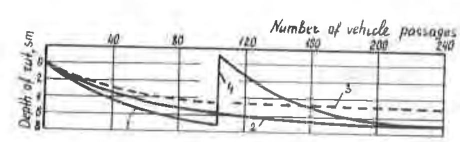

Fig. 3 Relationship between rut depth and number of load applications: 1 - gravel layer, 40 cm thick, no membrane; 2 - gravel layer 70 cm thick, no membrane; 3 - gravel layer 40 cm thick, membrane of non-woven fabric; 4 - grading operations.

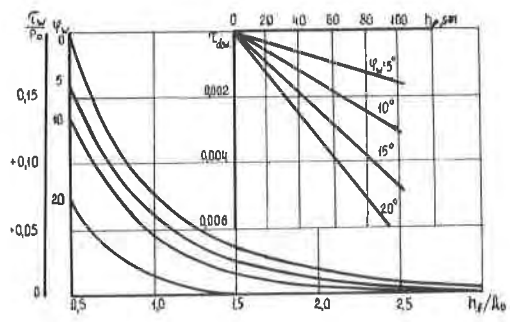

Fig. 4 Relative active shear stress τ_w/p_o and correction to soil dead weight γ_{dw} versus thickness of granular layer h_f.

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Experimental and Theoretical Behavior of Geotextile Reinforced Aggregate Soil Systems

Comportement expérimental et théorique des systèmes agrégats-sol renforcés de géotextiles

The development is briefly described of a general finite element computer program for analyzing a plane strain or axisymmetric fabric reinforced soil continuum. The results are also described of an extensive series of model tests conducted in 0.9 and 2.5 m dia. circular test tanks simulating an aggregate surfaced haul road. Fabric reinforcement was found to be effective in reducing rutting partly by changing the state of stress in the soft clay subgrade. After large rut depths develop, measured vertical stresses in fabric reinforced systems on the centerline beneath the load averaged about 83% of Boussinesq stresses compared to 88% for a nonfabric system. Fabric reinforcement caused a substantial reduction in permanent strain (and hence rutting) in the soft clay subgrade for a depth equal to about one loading diameter. Other consequences of fabric reinforcement are also discussed.

Le mémoire décrit brièvement le développement d'un programme à l'ordinateur, composé des éléments finis généraux, pour l'analyse d'une déformation plane ou d'un continuum axisymmétrique du sol renforcé par un tissu. On présente en outre les résultats d'une série étendue d'essais en modèle, dans lesquels on a simulé une route de camionnage en employant des cuves d'essai circulaires aux diametres de 0.9 m et de 2.5 m. Les essais montrent que le tissu affaiblit effectivement le sillonage, en partie à cause d'un changement de l'état de résistance dans le sous-sol d'argile tendre. Après l'apparence des ornières assez profondes, les résistances verticales measurées sur le trait de centre sous la charge, dans les systèmes renforcés en textile, donnent une moyenne à peu près 83% des résistances Boussinesq, par rapport à 88% dans les systèmes sans tissu. Le renforcement en textile suscite une diminution importante de la déformation permanente (et, par consequent, du sillonage d'ornières) dans le sous-sol d'argile tendre, jusqu'à la profondeur approximative d'un diamètre de charge. Le menoire traite aussi d'autres suites du renforcement textile.

INTRODUCTION

At the present time relatively little research has been undertaken to define the basic mechanisms which control the performance of fabrics in geotechnical applications. An understanding of the mechanics of fabric reinforcement in this type application is required to develop rational design procedures compatible with the actual mechanics of fabric reinforcement behavior.

This paper describes the development of a general analytical model for fabric reinforced continuum problems, and summarizes some of the findings of an extensive series of model tests conducted in 0.9 m and 2.5 m (3 ft. and 8 ft.) dia. circular tanks. Although the GAPPS7 finite element program developed is quite general, the verification and parametric studies presented are limited to haul road applications.

FINITE ELEMENT MODEL

Introduction. The GAPPS7 finite element program accurately models a layered soil continuum reinforced with fabric and subjected to multiple load applications (1,2). Additional important features of the GAPPS7 program include (1) the ability to consider either small or large displacements which occur under multiple wheel loadings in a haul road, (2) modeling of materials exhibiting stress dependent behavior including elastic, plastic and failure response, (3) modeling of the fabric interfaces including provisions for slip or separation, (4) a two-dimensional flexible fabric membrane element not capable of taking either bending or compression loading, (5) no-tension analysis for granular materials, and (6) provision for solving either plane strain or axisymmetric problems. The GAPPS7 program does not consider either inertia forces or creep, and repetitive loadings are applied at a stationary position (i.e., it does not move along the continuum). Material properties can, however, be changed for each loading cycle to consider time and/or load dependent changes in properties.

To make the GAPPS7 program practical for routine use, an automatic data generation program MESHG4 was developed. In addition to handling material properties, MESHG4 completely generates the finite element mesh from a minimum of input data. To check the generated mesh and assist in interpreting the large quantity of data generated by GAPPS7, a plot program was developed called PTMESH. Use of these programs greatly facilitates performing finite element analyses and also checking for errors in the data.

For a typical 44 to 56 element mesh the computer running times vary from about 500 to 750 system seconds on the CDC CYBER 74 computer; actual cost is about $10 to $12 (U.S.) for an economy run. The running times and costs are for nonlinear material properties, 8 to 10 load increments, and the small displacement option.

Eight Node Isoparametric Element. In applying the finite element technique the mass is divided into a discrete number of elements small enough in size to insure satisfactory accuracy (Fig. 1). The soil continuum (such as the soft clay and aggregate in a haul road) is modeled by an eight node, isoparametric element. This element has the capability to model curved boundaries and has a quadratic variation of displacement. The stiffness of each isoparametric eight node element is obtained by integration using nine sampling points.

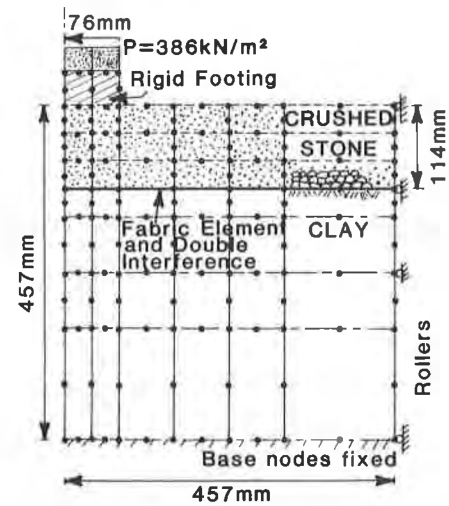

Fig. 1 Typical Finite Element Used to Model Laboratory Tests.

Nonlinear and Large Displacement Analysis. The solution strategy for nonlinear and large displacements is based on the solution of incremental formulations of the problem. A stress dependent elastic modulus E can be used having the form (1) $E = K\sigma_\theta^n$ where K and n are material properties and σ_θ is the sum of the principal stresses or (2) $E = f(\sigma_1 - \sigma_3)$ where σ_1 and σ_3 are the major and minor principal stresses, respectively. Load is applied to the model in small increments, and computations of incremental and total stresses are performed by solving a system of linear, incremental equilibrium equations for the system. Both geometric (large displacements) and material nonlinearities can be included as desired in the formulation. Geometric nonlinearities are caused by the change in geometry of the body as it undergoes large displacements.

Using the large displacement option the updated Lagrangean approach (1,6) is employed in the analysis. This formulation considers the change in geometry of each element with each load increment. Stresses, strains, and displacements are referred to the immediately previous configuration of the body. Unbalanced forces are reversed and applied to the nodes after each iteration.

No-Tension Analysis. The no-tension analysis considers the inability of granular, cohesionless materials such as crushed stone to take tension. These materials have no true cohesion and can take only limited amounts of tension in the horizontal direction due to interlock, friction and passive soil resistance outside the loaded area. A wheel loading causes relatively large vertical stresses which force the grains together at point contacts. Because of these normal stresses the granular material can take some friction before a tensile failure occurs (3). Special consideration must therefore be given to the bottom portion of a granular layer placed above the fabric in a haul road because of the existance of a tensile state of strain.

An attempt was made to extend the no-tension analysis for rock joints by Zienkiewicz, et al. (4) to the soil-fabric problem. Slow convergence of the technique was observed, with seven iterations per load increment not being sufficient to equilibrate the system even at low stress levels. The Zienkiewicz approach was therefore abandoned after several attempts to adapt it, and subsequently the simplified method for handling tension of Raad and Figueroa (5) was successfully implemented.

The principal stresses and strains are computed using GAPPS7 for each of the 9 integration points of the eight node isoparametric element used to model the granular material. The existance of failure is checked at the end of each iteration at each sampling point. To define failure the extended Drucker-Prager failure criteria (1,6) is used as a three-dimensional extension to the Mohr-Coulomb failure law for axisymmetric conditions. In apparent contrast to the Raad and Figueroa's approach, plastic strains are accumulated during the analysis.

This admittedly empirical approach for handling tension is illustrated in Fig. 2 for one stress condition. Assume the granular material is at a state of failure due to low lateral confining pressure, σ_3, at any one of the integration points as shown by Mohr circle C in Fig. 2. At failure the Mohr circle must just touch the failure envelope and cannot extent beyond it. Therefore, when the calculated failure circle extends beyond the envelope it is pushed back to just touch the envelope keeping σ_1 the same. Equilibrium of the system is checked after each iteration. After the modification the principal stresses comply with Mohr-Coulomb failure criteria, and the material will be in a perfectly plastic failure state. Use of small load increments produces stress states that

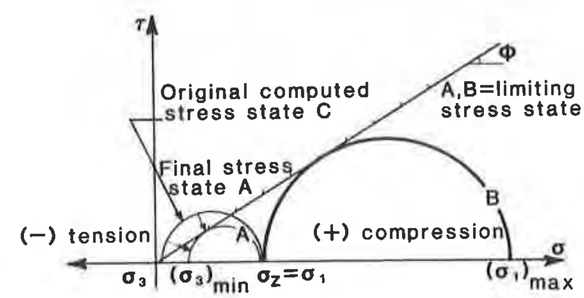

Fig. 2 Approximate No-Tension Correction Used in Aggregate.

are close to the failure envelope and hence only require a small amount of correction.

Interface Element. Slip between the fabric and adjacent material is of considerable interest in studying the behavior of fabric reinforced solids. Special elements illustrated in Fig. 3 were therefore developed to model the fabric interfaces using the work of Wilson, Goodman, Herrmann and others (1). In the figure two interface elements are shown on each side of four fabric elements.

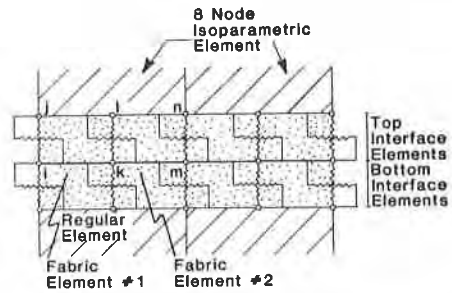

Fig. 3 Soil-Fabric Finite Element Model.

The interface elements are also attached at the common nodes to the isoparametric elements. The interface element ijklmn (Fig. 3) is formed by six linear springs and six nodes, and satisfies compatibility requirements with adjacent eight-node elements. In the model the concentrated normal and shear springs replace the uniformly distributed foundation type modulus, k, which is the ratio of stress divided by the displacement.

The computed shear and normal stresses at the interface define the mode of behavior along the interface: no slip, slip, or separation. Depending on the behavior mode, the actual forces developed at the interface are applied to the system. The maximum allowable shear at the interface is given by the Mohr-Coulomb Failure Law

$$\tau_{max} = c_a + \sigma_n \tan\delta \qquad (1)$$

where:

- c_a = adhesion between the interface materials
- δ = friction angle between the interface materials
- σ_n = normal stress to the interface

The interface material properties c_a and δ have been determined in the laboratory by performing dynamic, direct shear tests simulating the fabric-soil interface. If the shear strength of the material in contact with the fabric is less than the adhesive strength given by Eqn. (1), (which in general was found not to be true in the tests performed) the soil shear strength controls along the interface.

Fabric Element. The fabric element (Fig. 3) models the reinforcing fabric placed for a haul road application between the aggregate and subgrade. Linear, two-dimensional, axisymmetric and plane strain fabric elements were developed as extensions of a one-dimensional pinned end bar element. To give a good approximation of the behavior of the interface and the fabric, two fabric elements are placed adjacent to one interface element (Fig. 3). The fabric element can only take tension in the plane of the fabric; compression and bending is not permitted. As a result, the fabric element when carrying a tension load must remain straight between node points. Since small fabric elements are used, a good approximation is obtained to the curved profile actually assumed. The friction forces that are developed along the fabric are applied at the nodes of the element. In application the fabric-interface model has been found to give quite good results. As the load is increased in increments and iterations are carried out, a nonlinear fabric load-deflection curve can be followed in the analysis.

Solution. The stiffness of the system is determined by adding the stiffness contributions from the isoparametric, interface and fabric elements. Using the applied loads and system stiffness, the linear equations of the system are solved using an efficient, narrow banded equation solver (6). The stiffness of the system is varied after each load increment and iteration. This requires extra computer time to form the stiffness matrix for each load increment and iteration, but reduces considerably the number of iterations for convergence in a nonlinear analysis. By varying the system stiffness in this manner, a closer representation of the constitutive equations is obtained when slip, tension, and yield occur.

MODEL TESTS

A laboratory program was developed to experimentally examine under conditions of repeated loading the structural response of aggregate-soil (AS) and aggregate-fabric-soil (AFS) systems. An important secondary purpose of the test program was to aid in the validation of the GAPPS7 program. Haul road type systems consisting of soft subgrade soil, fabric and crushed stone were modeled in two circular test tanks. Measurements made during the repeated loading included permanent deformations throughout the section (including surface rutting), vertical and horizontal stresses and a limited number of fabric deflection strain) measurements.

Test Pit Construction and Loading. An extensive test program was performed in a 0.9 m dia. (3 ft.) pit to establish relationships between rut depth, aggregate thickness, and subgrade strength. A large 2.5 m (8 ft.) dia. pit was used for a limited number of tests. This size test facility permitted using an aggregate layer thickness and loading representative of conventional haul roads. One purpose for this latter series of experiments was to provide a scale factor for the results obtained from the smaller scale tests.

A soft clay subgrade (CL by the Unified Soil Classification System) was constructed having a 21 to 62 kN/m^2 (3 to 9 psi) undrained strength in the small pit and about 28 kN/m^2 (4 psi) in the large pit. The clay was thoroughly blended with the desired amount of water and then placed in uncompacted lifts of about 50 mm (2 in.) thickness in the small pit and 100 mm (4 in.) in the large pit, and then compacted. The moisture content, which varied from about 19.5 to 23%, was adjusted to achieve the desired subgrade strength. A subgrade thickness of 760 mm (30 in.) was used in all tests in the large pit; a 380 mm (15 in.) thick subgrade was used in the small pit.

For tests using fabric, a circular piece larger than the test pit was cut and placed on the subgrade with the excess material being turned up at the pit wall. The fabric used in this study was a nonwoven, heat bonded polypropylene manufactured by DuPont and sold under the trade name of Typar®* spunbonded polypropylene, Style 3401.

A crushed, dense-graded granite aggregate having a 25.4 mm (1 in.) maximum size was then placed in loose lifts of 50 to 75 mm (2 to 3 in.) and thoroughly compacted. For the 2.5 m (8 ft.) pit, a 370 mm (14.6 in.) thick aggregate layer was used while for the 0.9 m (3 ft.) pit, layer thicknesses of 150, 190 and 230 mm (6, 7.5, and 9 in.) were used. The repetitive load was applied to the surface of the crushed stone through a 300 mm (12 in.) dia. rigid plate for the large pit; a 150 mm (6 in.) dia. rigid plate was used for the small pit. In both cases 20 repetitions of a haversin shaped load pulse was applied per minute to the top of the rigid loading plate using a pneumatic or hybrid pneumatic-oil loading system. The dynamic load pulse had a duration of 0.2 sec. and a peak pressure of 482 kN/m^2 (70 psi).

Typical measured rutting as a function of the number of load applications obtained from the 0.9 m (3 ft.) dia. test pit is illustrated in Fig. 4. The surface deformations (rutting) shown in this figure was measured using dial gages. This figure shows that the use of the Typar® fabric has an important beneficial effect upon reducing rutting.

Pressure and Plastic Strain Measurements. Small pressure cells utilizing a special diaphram wire resistance strain gage were used to measure the pressures within the soft clay subgrade. Details of cell construction, calibration and placement are given elsewhere (7). The pressure cells were offset 75 mm (3 in.) in the small pit from the load centerline, and placed at depths of 38, 89, 150, 216 and 279 mm (1.5, 3.5, 6, 8.5, 11 in.) below the top of the subgrade. Pressure cell readings were taken during application of the repeated loading. Final cell locations and orientations were carefully determined after completion of each test. The vertical pressures measured in the large pit are summarized in Table 1 (including location) and were obtained for surface rut

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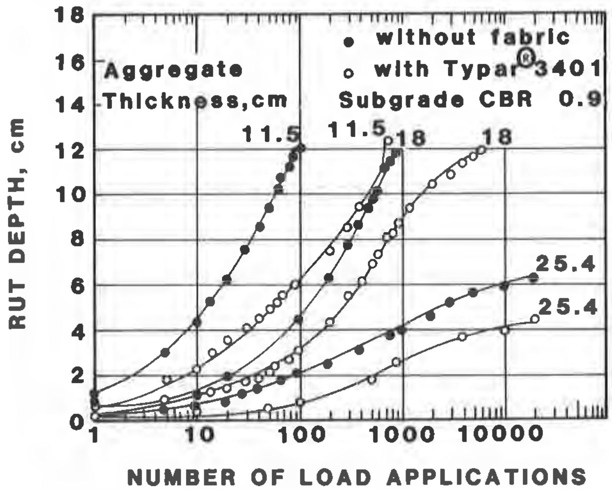

Fig. 4 Rut Depth as a Function of Applications With and Without Fabric: 0.9 m Test Pit.

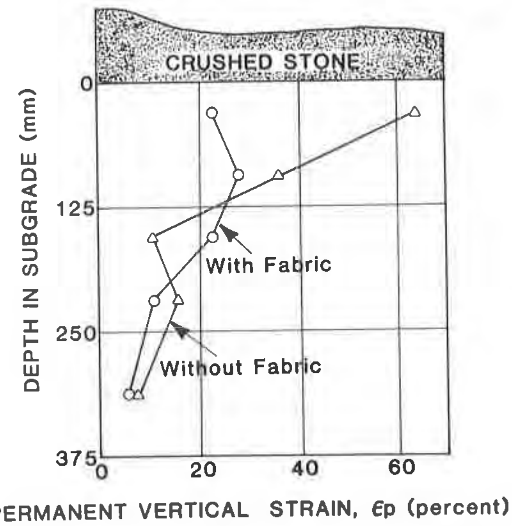

Fig. 5 Permanent Vertical Compressive Subgrade Strain Observed as a Function of Depth in the Subgrade: 0.9 m Pit Tests.

depths of about 70 to 100 mm (3 to 4 in.).

Vertical plastic strains within the soft clay subgrade were determined for selected tests in the 0.9 m (3 ft.) pit using 19 Bison sensors (7). Vertical stacks of strain sensors were placed at 16 mm (4 in.) intervals going from the centerline out to 63 mm (16 in.). The typical variation of plastic strain in the subgrade on the centerline of the load is shown in Fig. 5 for a similar system with and without fabric. A significant reduction in plastic strain was found to occur immediately below the fabric along the centerline.

DISCUSSION

Stress Distribution. The GAPIN Program indicates as shown in Fig. 6 a reduction of 8 to 14% in vertical stress along the centerline for systems reinforced with Style 3401 Typar® compared with nonreinforced systems. Stress distributions measured in both test pits also show similar important stress reductions in fabric reinforced systems.

Figure 7 shows the measured variation of vertical stress along the centerline as a function of depth for pressure cells 3, 7 and 9 in the large pit. The systems reinforced with fabric showed approximately a 20% reduction in stress in the soft clay along the centerline compared to the AS systems. At the crushed stone-clay interface the vertical stress in the fabric reinforced systems was slightly less than Boussinesq stress distribution theory for a homogeneous elastic continuum. With increasing depth the measured stresses were greater than Boussinesq in this test series. The rigidity of the bottom of the tank was at least partly the cause of the increase in the measured stresses. Pressure cell 5, offset 460 mm (18 in.) from the load centerline and 150 mm (6 in.) below the top of the subgrade, showed greater vertical stresses for the AFS system than for the AS system (Table 1). Subgrade profile measurements made after the test showed this cell was directly below the approximate location of maximum subgrade heave. The GAPPS7 program also indicates a slight increase in pressure in the heave zone, although not as great as measured. The program also indicates the stress redistribution is caused by (1) plastic flow of soil beneath the originally more highly stressed center area resulting in softening of the clay and (2) the downward membrane effect due to in-plane fabric stress.

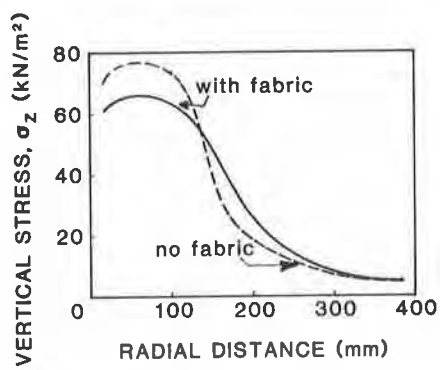

Fig. 6 Vertical Stress on a Horizontal Plane Computed by GAPPS7 (Mesh Shown in Fig. 1).

Table 1. Stresses Measured by Pressure Cells in 2.5 m Pit Tests.

Cell No.	Initial Position		Stress Normal Pressure Cell (kN/m²)	
	Depth (mm)	Offset (mm)	With Style 3401 Typar®	Without Fabric
9	406.	0	82.7	107
3	508.	0	75.8	107
4	762.	0	-	-
7	1070	0	37.9	44.8
8	508.	152.	56.5	85.6
6	508.	305.	48.2	51.7
5	508.	457.	11.0	5.5
2*	406.	457.	23.4	34.5
1*	813.	762.	23.4	23.4

*Radial stress - all other stresses are vertical (25.4 mm = 1 in.; 6.89 kN/m² = 1 psi).

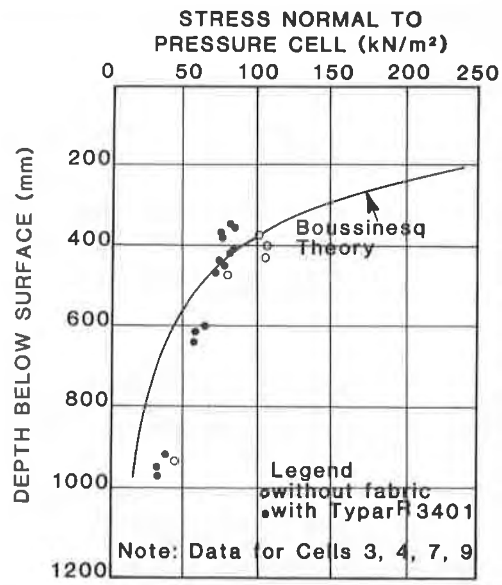

Fig. 7 Measured Stress Under Load Centerline in Subgrade of 2.5 m Pit Test.

Pressure cell measurements made in the small pit indicate: (1) prior to substantial rutting, the measured stresses in both AFS and AS systems averaged about 90.5% of that predicted by Boussinesq theory; less than 2% difference was observed between the two systems; (2) after substantial surface rutting (7 to 10 cm or 3 to 4 in.), the measured stresses for the AFS systems averaged about 83% of Boussinesq values compared with 88% of Boussinesq stresses in the AS system. In each case the changes of cell location and orientation were considered in the comparisons. The GAPIN program gave, for the fabric reinforced systems and moderate rut depths, vertical centerline stresses corresponding to about 80% of Boussinesq stresses in the subgrade.

Permanent Deformation. Beneath the repeated loading the model tests show the aggregate pushes downward into the subgrade causing the clay to flow laterally outward and upward. This lateral flow results in a radially offset bulge (heave) on the surface. As the thickness of the aggregate increases, the depth of subgrade rutting and height of heave both decrease, while the width of the rutted area increases. Use of Style 3401 Typar® was found to have a similar effect on rut development as increasing the aggregate layer thickness.

AFS systems, reinforced with this fabric, exhibited up to three times more vertical plastic strain in the aggregate at failure than did nonreinforced systems having similar surface rutting. This trend was most pronounced in the weaker systems. The number of repetitions to cause the same magnitude of rutting in the AFS systems, however, was much greater than in the AS systems. In the AFS systems the greatest amount of plastic vertical strain occurred in the aggregate layer, while in the AS systems the greatest plastic strain occurred in the clay.

Fig. 8 shows the experimental relationship between the amount of densification (volume change) and shear (distortion) occurring in the soft clay subgrade for both AFS and AS systems. As the initial stress ratio increases, the importance of shear distortion rapidly increases in both type systems, accounting for about 50% of the subgrade rutting at the stress ratio of 3 and about 75%

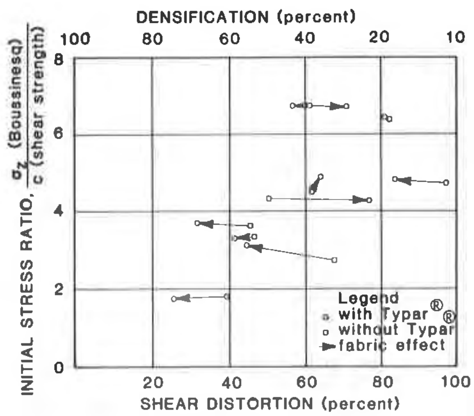

Fig. 8 Amount of Shear Distortion and Densification in Subgrade as a Function of Initial Stress Ratio.

at a stress ratio of 5. The stress ratio is defined as the vertical Boussinesq stress divided by the subgrade shear strength. These findings are quite reasonable since densification in the soft clay occurs gradually under the application of a large number of load repetitions as the water is slowly squeezed from the clay. Therefore, either AFS or AS systems subjected to a high initial stress ratio undergo a shear distortion type failure before densification can become the dominating mechanism.

Use of the fabric causes a substantial reduction in permanent strain (and hence rutting) in the soft clay subgrade for a depth equal to about one loading diameter B beneath the interface (Fig. 5). This reduction in strain decreases almost linearly with depth. Further, the maximum vertical plastic strain in systems containing fabric occurred at a depth of about 0.6B beneath the interface. In systems without fabric, the maximum strain occurred at or very near the interface and generally decreased with increasing depth. Due to application of a heavy wheel loading, the soft clay just below the crushed stone is stressed to a high percent of its shear strength. As a result, the relatively small reduction in stress which occurs when the fabric is present causes an important reduction in the accumulation of permanent strain in the zone just below the fabric as shown in the model tests.

Fabric-Interface. The variation of strain in Style 3401 Typar® fabric with stone thickness and rut depth obtained using the GAPPS7 program is shown in Fig. 9. The GAPPS7 program indicated excessive rutting would develop before 700 repetitions for an aggregate thickness of 330 mm (13 in.). As this system approached an excessive rutting condition, the calculated maximum fabric strain was 3% at the centerline and 2% at a radial distance from the centerline of 1.33B, where B is the diameter of the load. The fabric strain measured in the small pit for a rut depth of 76 mm (3 in.) and radial offset distance of 1.33B was 1.9%; this value compares quite favorably with the theoretical value of 2%. These results indicate that the Typar® fabric is stressed to about the elastic limit which corresponds to 3.2 N/mm (18 lb./in.). This stress level is in a safe working range well below the ultimate value of about 7.4 N/mm (42 lb./in.).

Using measured interface friction and adhesion properties, the GAPPS7 program indicates slip does not occur for AFS systems which are adequately designed having a subgrade shear strength as low as 22.7 kN/m^2 (3.3 psi); lower strengths have not been investigated. As large rut

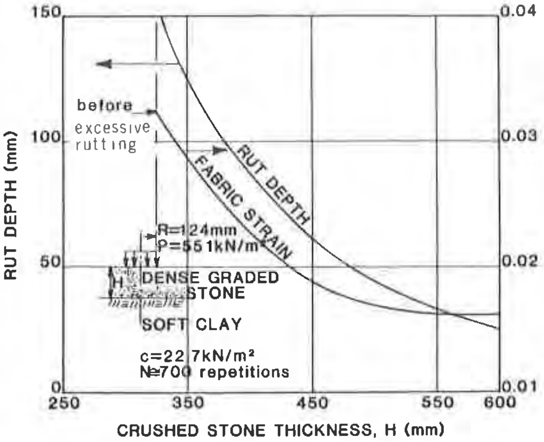

Fig. 9 Variation of Rut Depth and Fabric Strain as a Function of Crushed Stone Thickness.

depths develop, however, slip between the clay and fabric can occur at least at some locations.

General. The test results show for a given system with or without fabric, a threshold thickness of crushed stone exists which separates a stable system having tolerable levels of rutting from an unstable system. This threshold stone thickness is less for AFS systems than for AS systems (Fig. 4). For overstressed systems having crushed stone thicknesses less than the critical value, rapid accumulation of large rutting in the subgrade is primarily due to shear distortion (flow) of the subgrade laterally from beneath the load.

These results clearly indicate that Style 3401 Typar® is effective in changing the state of stress and strain in the subgrade. As rutting increases, vertical and radial stress under the loaded area are reduced while the vertical compressive stress outside the loaded area (in the region of subgrade heave) is increased. The reduced stresses observed are most likely due to (1) upward induced stress (membrane effect) and possibly (2) greater aggregate modulus (as a result of confinement/reinforcement provided by the fabric). This alteration in state of stress is a primary reason that the fabric reinforced system exhibits greater rutting resistance.

The substantial alteration of plastic strain distribution in the subgrade is very interesting and obviously has a major impact on system rutting. The reduction of vertical plastic strain in the subgrade for systems containing fabric apparently is due at least in part, to the reduced vertical compressive stresses. However, the reduced vertical stresses do not explain why the vertical plastic strain in the soil adjacent to the fabric is less than that a few centimeters down into the subgrade. This anomaly is most likely caused by (1) the increased confinement offered to the soil adjacent to the fabric and (2) reduced shear stresses at the surface of the subgrade.

CONCLUSIONS

Both the model tests and the GAPPS7 program show the presence of fabric causes a definite beneficial alteration of the stress and plastic strain distribution in a fabric reinforced haul road type system. The laboratory model tests and the GAPPS7 computer program offer a sound mechanistic approach for studying the behavior of fabric reinforced systems and developing useful design relationships. The GAPPS7 program was recently developed and application of this powerful tool is just being implemented.

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Effect of Fabric Properties on the Performance and Design of Aggregate-Fabric-Soil Systems

Influence des textiles sur les performances et le dimensionnement de systèmes: agrégats-textile-sol

The effects of fabric properties on performance and design of aggregate-fabric-soil (AFS) systems are discussed and quantified where possible. Primary emphasis is placed on examining the effect of mechanical properties on performance and design using data obtained from the pertinent literature and a study conducted at Georgia Tech. In general, it was found that the fabric modulus was the single most important fabric property governing the behavior of the AFS system. The use of a geotextile in an aggregate soil system leads to improved system performance (e.g. longer service, reduced rutting) or alternately, a 25-40 percent reduction in the amount of required aggregate.

Quantitatively, the amount of this performance improvement (aggregate reduction) resulting from the use of a particular fabric correlates well with the modulus (resistance to stretch) of the fabric used. High modulus fabrics result in less rutting or better system performance than those with lower modulus.

Les influences des propriétés textiles sur la performance et sur l'étude des systèmes agrégat-tissu-sol (AFS) sont discutés, et, dans la mesure du possible, quantifiés. On souligne l'examen de l'influence des propriétés mécaniques sur le fonctionnement et sur l'étude, en employant les données obtenues de la bibliographie et d'une étude faite à Georgia Tech. On a conclu, en générale, que le module du tissu est la propriété la plus importante pour le fonctionnement du système AFS. L'emploi d'un géotextile dans un système agrégat-sol mène à un fonctionnement amélioré (e.g. fonctionnement prolongé), moins d'ornières, ou, comme alternative, une diminution de 25 à 40 pourcent de la quantité requise d'agrégat. L'amélioration qu'apporte l'emploi du tissu spécifique au fonctionnement du système (diminution d'agrégat) présente une bonne correlation avec le module d'élasticité (résistance à la traction) du tissu employé. L'emploi du tissu à module élevé, comparé à celui d'un module moins élevé, donne une réduction d' ornières et un fonctionnement meilleur du système.

INTRODUCTION

The use of geotextiles or fabrics in high deformation, low volume road construction has become increasingly popular over the last two decades. In this application the fabric is used in conjunction with a locally available aggregate such as crushed stone, quarry "shot rock", sand, sea shells, etc. to develop a structural support layer.

The benefit offered by the fabric is attributed to reinforcement and separation and is most often measured in terms of improved system performance or, alternately, in terms of reduced aggregate thickness requirements (1). Figure 1 shows the effect of fabric on aggregate road performance. For low strength support conditions where fabrics appear most beneficial, a reduction in aggregate thickness in the range of 25 to 40% can be made normally when fabric is installed between the aggregate and soil (1).

Selection of fabrics and establishment of use specifications for particular field applications is often difficult for the potential fabric user due to a general lack of knowledge relative to the impact of fabric properties on potential performance. Since many existing design methods for aggregate roads are (a) fabric specific, (b) unclear as to basic assumptions, (c) empirical and (d) unable to predict performance, the impact of various fabric types (and hence properties) on performance and economics of the fabric reinforced aggregate road is not readily apparent.

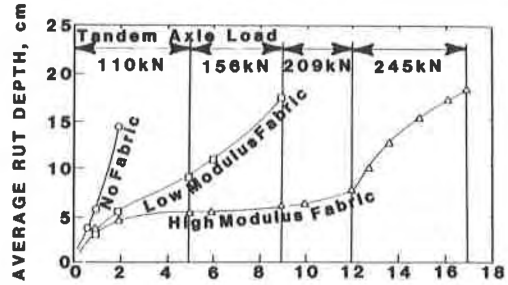

Fig. 1 Rut Depth as a Function of Vehicle Passes (From Ref. 5).

PURPOSE AND SCOPE OF PAPER

The primary purpose of this paper is to quantify where possible, based on current literature and the results of a recent study, the effect of fabric properties on the performance and design of fabric reinforced aggregate roads or aggregate-fabric-soil (AFS) systems. Sources of data and information used to develop this paper include pertinent literature and the results of a study being conducted in the School of Civil Engineering at the Georgia Institute of Technology, Atlanta, Georgia, U.S.A.

GEOTEXTILES

General

For the purpose of this paper, the term geotextile or fabric will be used interchangeably and will adhere to the definition established by ASTM which says a "geotextile is any permeable textile used in conjunction with goetechnical materials as an integral part of a manmade project, structure, or system".

The geotextile market has increased dramatically in recent years as a result of new uses and new manufacturers. Available commercially is a wide range of man-made, synthetic fabrics ranging in type (woven and nonwoven), fiber composition (mainly polyester and polypropylene), basis weight and inherent properties. Undoubtedly more fabrics will become available in the future. Space limitations do not permit a detailed description and/or identification of all the geotextiles available commercially.

Fabric Properties

The degree of benefit offered by a fabric to the AFS system for its service life depends to a large extent on the inherent properties of the fabric used. Other factors such as subgrade strength, loading environment, and aggregate properties also have an important influence on the behavior and performance (rutting resistance) of the AFS system (1).

Specific properties of significance relative to the optimum use of fabrics in aggregate surfaced roads are numerous, although the exact contribution of each is largely unknown. Bell, et al. (2) have listed and discussed in detail a large number of fabric properties of apparent significance in the broad sense of geotechnical applications which include the following: Mechanical Properties -- strength, elongation, modulus, creep, stress relaxation, fatigue, tear resistance, cutting and abrasion resistance, friction; Hydraulic Properties -- permeability, filtering ability, clogging and blinding resistance, capillary siphoning; Durability Properties -- thermal, biological, chemical, and ultraviolet light stability.

The previous list of fabric properties is formidable. In a qualitative sense all may appear significant. However, the minimum, maximum or optimum level of each and the combined or interactive effect of these properties are yet to be fully understood and quantified. Even test methods to evaluate specific properties have not yet been universally accepted. As an example, mechanical properties of fabrics are often determined from mechanical tests on the fabric in isolation. When a fabric is placed in the AFS system, the fabric may behave in a substantially different manner because of the presence of aggregate and soil. Holtz (3) and McGown, et. al (21) report that the modulus of fabrics in soil may be 2-3 times the value in isolation. Complicating the situation even more is the fact that most fabrics are anisotropic, i.e. they have properties which depend upon orientation.

Bell, et al. (2) suggest that mechanical properties of fabrics may be the most significant for ground stabilization applications. Hydraulic properties probably have secondary importance. Dissipation of pore pressure created due to loading and settlement can be accommodated by most geotextiles. Furthermore, for typical AFS system applications, most geotextiles have adequate durability (e.g., thermal, biological, chemical, and ultraviolet stability).

BENEFIT MECHANISMS

Mechanisms by which a fabric improves the structural performance of the AFS system under repetitive vehicular loading have been discussed by several investigators and include basically two categories, reinforcement and separation.

Specific mechanisms that have been identified are:

A. Restraint Effect

Two types of restraint effect may occur in the AFS systems. The first, often referred to as subgrade restraint, is related to the reverse curvature of the fabric that develops outside the wheel path and the resultant induced downward pressure or apparent "surcharge" applied to the soil, Figure 2. Such an effect increases the bearing capacity or resistance to shear flow of the soil from the wheel path. A second type of restraint, called aggregate restraint, occurs when the aggregate particles at the soil-aggregate interface tend to move from under the loaded area but are restrained or confined due to the presence of the fabric (4). Modulus, strength, friction, creep (stress relaxation) and abrasion or puncture resistance of the fabric may be very important to this mechanism.

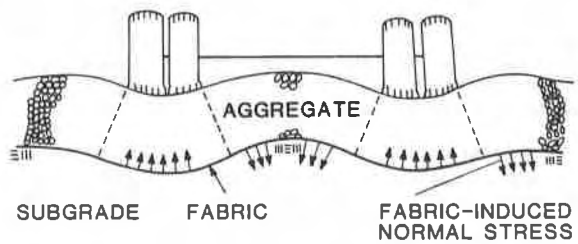

Fig. 2 Schematic of Aggregate-Fabric-Soil System.

B. Membrane Effect

As the roadway undergoes large deformation, Figure 2, the fabric is stretched and develops in-plane tensile stress, the magnitude of which depends on fabric strain and fabric modulus. A stress perpendicular to the plane of the fabric is induced, the magnitude of which at any point equals the in-plane stress divided by the radius of curvature of the fabric at that point. The net effect is a change in the magnitude of stress imposed on the subgrade (a reduction under the wheel load and an increase outside of the wheel path) and an increased confinement of the aggregate.

In order to develop fabric-induced stress, substantial vertical deformations, plus proper geometry, fabric anchorage and proper fabric mechanical properties are generally required. Fabric properties of modulus, strength, creep (stress relaxation), elongation-to-break and friction are important to this mechanism.

C. Friction and Boundary Layer Effect

Friction developed along the interface between aggregate-fabric and friction/adhesion at the fabrics-oil interface create a "boundary-layer" or composite material of aggregate and soil immediately adjacent to the fabric. The composite material created should possess more favorable properties of ductility and tensile strength. Fabrics capable of developing high friction/adhesion appear to be desirable.

D. Local Reinforcement

Concentrated stresses due to imposed vehicular loading can cause a punching or local bearing capacity failure at the points of contact between the aggregate and subgrade. Use of fabric between the aggregate and soft soil will serve to distribute the load, reduce localized stresses and in general provide increased resistance to vertical displacement. Mechanical properties of modulus, strength, and puncture resistance appear important for this mechanism.

E. Separation

In the separation function, the fabric serves to prevent the fine-grained subgrade soil from pumping and intermixing with the coarse-grained aggregate material and thereby reducing its shear strength and stability. Depending on aggregate gradation, 10 to 20 percent additonal plastic fines can cause a substantial reduction in shear resistance (5,6).

Bell, et al. (2) have discussed extensively the function of separation provided by fabric. Basically two phenomena have been identified which must be mitigated if the separation function is maintained; these are subgrade pumping and subgrade intrusion. Pumping requires relatively high stress at the subgrade-fabric interface, free water, pumpable subgrade and a granular material open enough to allow entry of the fine material (if the fabric were not at the interface). In regard to the pumping phenomenon, Bell, et al. (2) conclude that "the theories of pumping and fabric influences (on pumping) are not well developed. They (theories) do not even show clearly the fabric properties which are important to prevent pumping". Bell, et al. (2) also state "There are however, numerous installations of fabrics..which indicate that many of the fabrics on the market today do effectively prevent pumping of subgrades".

With respect to the intrusion phenomena, the fabric serves to physically prevent the intermixing of the granular and subgrade material. Bell, et al. (2) state that in order to prevent intrusion, the fabric must not be punctured by the aggregate and must not fail by localized rupture. Furthermore, they state that fabrics will tend to prevent intrusion or pumping of the subgrade and that important fabric properties (although not quantified) include pore characteristics, friction, strength, puncture resistance and abrasion resistance.

From the previous discussion, it can be assumed that as long as a geotextile remains intact, few problems will be encountered relative to pumping and intrusion.

EFFECT OF FABRIC PROPERTIES ON SYSTEM PERFORMANCE

In the previous section, various mechanisms responsible for the benefits accruing from the use of fabric have been discussed. Possible mechanical properties necessary for the benefit mechanism have been suggested. However, quantification of properties was not presented. In this section, the influence of fabric properties on AFS system performance will be discussed.

An examination of the literature to determine documented evidence of the influence of fabric properties on performance does not reveal many sources where specific comparisons and/or quantification have been presented. Following is a general summary of the significant literature.

WES Study

The results of a full-scale traffic test conducted by the Waterways Experiment Station, Vicksburg, Mississippi, have been published (7). Two test sections, each containing a fabric and one test section without fabric were constructed. The subgrade was placed to have a CBR of about 1 in the upper 25 cm (10 inches) and a CBR ranging from 1.5 to 2.3 in the next 35 cm (14 inches). A crushed limestone layer, 35 cm (14 inches) thick was placed above this subgrade with the respective fabric in each of the two test sections. The two fabrics used were Bidim* C-38 spunbonded polyester and T-16 (a neoprene-coated, one ply, woven, nylon). The fabric properties were (7):

	"Bidim" C-38	T-16
Elongation @ Failure:	58%	31%
Breaking Strength:	49 kN/m (280 lb/in)	76 kN/m (435 lb/in)
Modulus: (calculated by authors)	67 kN/m (380 lb/in)	300 kN/m (1720 lb/in)

The performance of these three test sections subjected to traffic by a tandem axle, dual wheel, military dump truck is depicted in Figure 1.

The T-16 fabric had a much higher modulus than the C-38 and thus, the influence of the higher modulus fabric is evident. For 900 vehicle passes the T-16 fabric section had about 5 cm (2 inches) of rutting while the "Bidim" C-38 section had about 18 cm (7 inches) of rutting. The section without fabric sustained only 200 vehicle passes to 15 cm (6 inches) of rutting.

Kinney and Barenberg

Kinney and Barenberg (8) have published performance data from small-scale repeated load tests on AFS systems containing two fabrics designated M-1 and W-2. They reported a modulus for these fabrics of 53 kN/m (300 lb/in) and 193 kN/m (1100 lb/in), respectively (8) and concluded that the higher modulus fabric improved performance as a result of greater confinement in the aggregate and resultant greater "load spreading ability" of the aggregate. Barenberg (9) has developed a design method for these two fabrics. Using this method, it can be shown that an ~ 10% reduction in aggregate thickness can be made if the high modulus fabric is used in lieu of the low modulus one.

Giroud and Noiray

Giroud and Noiray (15) have developed a design method which requires fabric modulus and failure elongation as design inputs. For the design conditions of CBR=0.5, rut depth = 30 cm (12 inches) and N=1000, this design method allows a reduction in aggregate thickness ranging from 25 to 40 percent for fabric modulus values ranging from 10 to 200 kN/m (60 to 1200 lb/in) (see Figure 3). Again it is seen that high modulus fabrics reduce the required amount of aggregate thickness.

Georgia Tech Study

A major study concerning the use of geotextiles in AFS systems is being conducted in the School of Civil Engineering at the Georgia Institute of Technology, Atlanta, Georgia. A number of papers based on this study have already been placed in the technical literature (1,11,12,13,14).

In one phase of this study, a scale model test apparatus was used to evaluate relative performance characteristics of various AFS systems and to evaluate the relative significance of important parameters on the system performance. Details of the testing method have been reported elsewhere (13) but the following is a brief summary of the equipment and test method:

1. 0.9 m (3 ft) diameter test pit with 38 cm (15 inches) thick, soft silty clay subgrade and dense-graded aggregate with layer thicknesses ranging from 13 to 33 cm (5 to 13 inches).
2. Subgrade soil prepared and placed to have a vane

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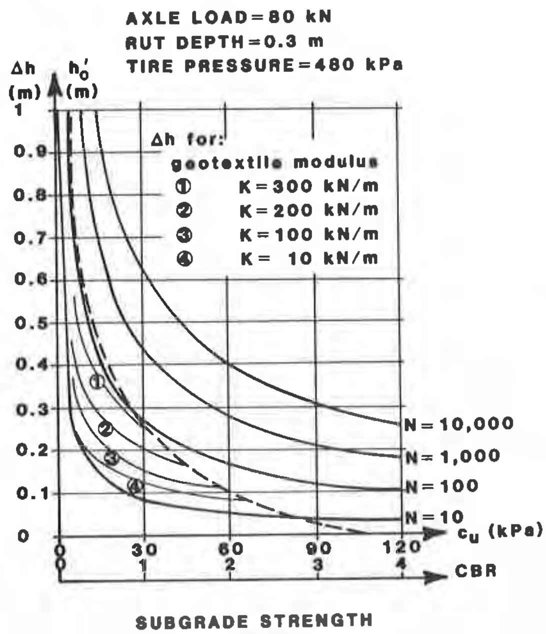

Fig. 3 Aggregate Thickness h_o' (Case without fabric) and Aggregate Thickness Reduction, Δh (Case with fabric) as a Function of Subgrade Strength (Redrawn from Ref. 15).

shear strength ≅ 28 kPa (4 psi) and CBR ≅ 0.9.

3. Fabric placed between soil and aggregate.

4. Repeated loading applied on 15 cm (6 inch) diameter plate with contact pressure = 480 kPa (70 psi), repetition rate = 20 per minute, and pulse duration = 0.2 sec.

5. During loading, vertical movement of loading plate is monitored.

In order to develop insight as to the manner in which a variety of fabric properties influence AFS system performance, two test series were conducted as part of the study. In one series of scale model pit tests, different types of commercially available nonwoven fabrics, e.g. Typar* spunbonded polypropylene fabric, Terram** construction membrane, Supac*** nonwoven polypropylene fabric, "Bidim", and 3 diagnostic membranes were tested under appoximately the same conditions (e.g., aggregate thickness, subgrade strength, and loading). In a second series, different basis weights of "Typar" were similarly tested. Table 1 summarizes pertinent characteristics of the fabrics and membranes and general test conditions for Test Series I. In all cases, the primary measure of performance was surface rutting of the AFS system. Table 2 summarizes the performance results from Test Series I.

The results were analyzed in a number of ways. In Figure 4, the general effect of initial fabric modulus on the initial rate of rut formation is shown. Figure 5 depicts the influence of initial fabric modulus on the number of repetitive loads required to cause either 5 or 10 cm (2 or 4 inches) of rutting in the AFS system. From these figures, it is obvious that increased fabric modulus relates quite significantly to increased rutting resistance.

*Registered Trademark of E.I. du Pont de Nemours
**Registered Trademark of ICI Fibers
***Registered Trademark of Phillips Fibers Corporation

Table 1. Fabric and Test Condition Summary--Test Series I.

Test	Fabric or Membrane	E_o (a) kN/m	Aggregate Thickness, cm	Subgrade Shear Strength, kN/m^2
I-1	"Typar" 3401	137	18.4	36
I-2	"Typar" 3251	100	18.5	30
I-3	"Terram" 1000	81	18.2	29
I-4	"Bidim" C-22	18	17.8	31
I-5	"Bidim" C-34	28	19.3	33
I-6	"Supac" 5P	30	18.7	33
I-7	Kevlar*(Aramid woven fabric)	1130	18.3	33
I-8	Dental Dam (sheet rubber)	0.5	17.3	31
I-9	Teflon*(sheet)	121	17.6	32
I-10	None	0	17.5	28

Footnotes:

(a) E_o = initial tangent fabric modulus (wide width tensile test).

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Table 2. Selected Performance Results from Test Series I AFS System Tests.

Test Designation	Number of Load Applications to Given Rut Depth 2.5 cm	5 cm	7.5 cm	10 cm	12.5 cm	Initial Rate of Rut Formation(a), cm/cy
I-1	37	72	120	220	385	0.068
I-2	26	51	76	116	170	0.096
I-3	22	38	55	82	130	0.114
I-4	9	14	26	48	90	0.278
I-5	27	50	62	88	140	0.093
I-6	47	66	77	95	126	0.053
I-7	57	205	630	8000	-	0.044
I-8	18	22	24	28	32	0.139
I-9	45	63	89	115	120	0.056
I-10	11	29	52	72	102	0.227

Footnote:

(a) Initial rate of rut formation = $\frac{2.5 \text{ cm}}{\text{N @ 2.5 cm rut}}$

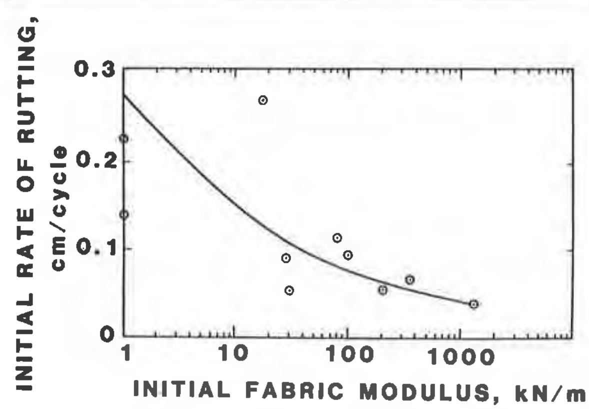

Fig. 4 Effect of Fabric Modulus on Initial Rate of Rutting (0.9 m Test Pit).

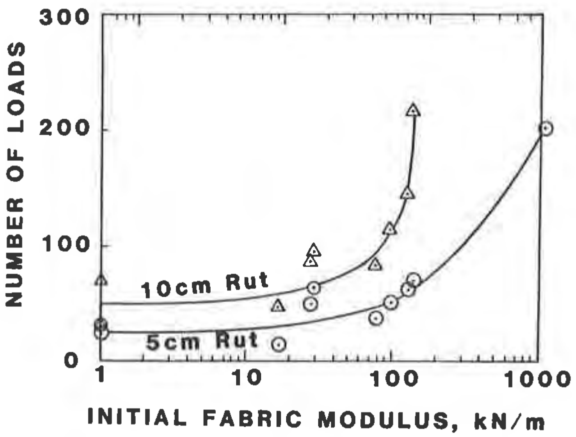

Fig. 5 Effect of Fabric Modulus on Number of Load Applications Required to Cause 5 and 10 cm Rut (0.9 m Test Pit).

The second test series was conducted wherein only various basis weight fabrics ranging from 70 to 270 g/m^2 (2 to 8 oz/yd^2) of "Typar" spunbonded polypropylene were included in the AFS systems. Figure 6 depicts the data relating initial modulus, E_0 and modulus at 10 percent elongation E_{10} to the number of load applications for 7.5 cm (3 inches) of rutting. Again the significant effect of fabric modulus on AFS performance and rutting resistance is quite obvious.

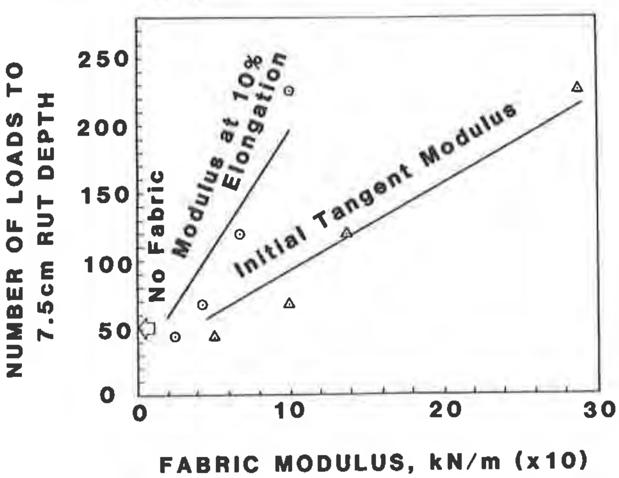

Fig. 6 Effect of Fabric Modulus of Number of Load Applications to Cause 7.5 cm Rut Depth (Basis Weight Series).

Other Studies

Kinney (22) has concluded, based on experimental and theoretical analyses, that the "ideal fabric from the standpoint of structural reinforcement will have a high ratio between tension and strain (i.e., high modulus) and a low tendency to creep to lower loads at constant strain" (i.e., low stress relaxation). However, exceedingly high tensile stress in the fabric can only be developed if adequate anchorage accompanies and for typical AFS systems, anchorage is limited by friction/adhesion, the amount of surcharge (aggregate thickness) and the length of embedded fabric outside the loaded area.

Raad (23), in his theoretical analysis of the effect of prestressing the fabric on behavior also indicates that high tension in the fabric is beneficial but that the in-plane stress is limited by frictional capabilities. He found that fabric prestressing reduced surface deflection and maximum shear stress in the subgrade.

EFFECT OF FABRIC PROPERTIES ON SYSTEM DESIGN

Based on the previous discussions, it is apparent that fabric modulus has an important effect on AFS system performance and as such should be considered in design. The authors are aware of 8 design methods generally available at this time (5,10,15,16,17,18,19,20). Of these eight, six are fabric-specific.

The more general non-fabric-specific design method developed by Giroud and Noiray (15) considers directly the effect of fabric modulus and failure elongation on design of AFS systems. A typical thickness design chart for this method is shown in Figure 3. Using this method, Figure 7 was developed and shows the general influence of fabric modulus on aggregate thickness requirements.

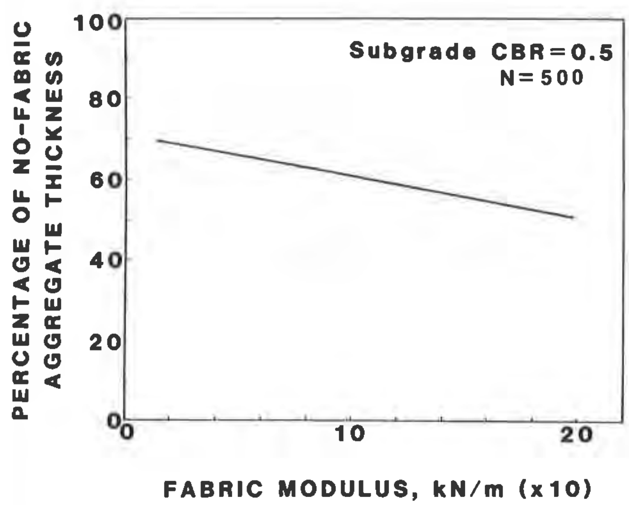

Fig. 7 Effect of Fabric Modulus on Design Thickness of Aggregate (From Ref. 15).

The method by the United States Forest Service (5) does not take into consideration any fabric properties except to indicate that "Preliminary information from trial use projects show that the lightweight nonwoven fabrics [135 g/m^2 (4 oz/yd^2)] performed as well as the heavier [270 to 540 g/m^2 (8 to 16 oz/yd^2)] once they were installed". The USFS method does suggest that a grab strength of $\geq$ 530 N (120 lbs) and an elongation $\geq$ 50% at failure are fabric requirements.

Using the fabric-specific design methods, it is virtually impossible to determine quantitative effects of fabric properties on thickness design due to differences in basic design assumptions (performance, number of loads, etc.).

DISCUSSION

Numerous mechanical properties of the fabric are important to the performance and design of AFS systems. The previous discussion has shown the tremendous importance of fabric modulus. Many other mechanical properties may be important, but their specific contribution, if any, has not yet been quantified. Maintenance of high levels of in-plane fabric stress are important to the performance of AFS systems and, thus, any fabric liabilities such as rupture, stress relaxation, or slippage are undesirable.

SUMMARY AND CONCLUSIONS

The effects of fabric properties on performance and design of aggregate-fabric-soil (AFS) systems have been discussed and quantified where possible. Primary emphasis has been placed on examining the effect of mechanical properties on performance and design. Data obtained from the pertinent literature and a study conducted at Georgia Tech were used. In general, it was found that the fabric modulus was the single most important fabric property governing the behavior of the AFS system.

Specific conclusions apparent from this paper are:

1. The use of a geotextile in an aggregate soil system can lead to improved system performance (e.g. longer service, reduced rutting) or alternately, a 25-40 percent reduction in the amount of required aggregate.
2. Quantitatively, the amount of performance improvement (aggregate reduction) resulting from the use of a particular fabric appears to correlate well with the modulus (resistance to stretch) of the fabric used. High modulus fabrics result in less rutting or better system performance than those with lower modulus.
3. Conceptually, it seems certain that other mechanical properties of the fabric are important for overall long-term system performance, although their specific effects have not yet been quantified. In general, a fabric with a good overall balance of properties is probably desired.

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Soil-Fabric Interaction—An Analytical Model

Un modèle analytique d'interaction entre un sol et une membrane géotextile

A new theoretical model for soil-membrane (geotextile) interaction is presented. A qualitative interpretation of the mechanical role of the membrane is proposed using a probabilistic concept for the vertical stress diffusion in a particulate media (Harr, 1977). A generalization is also offered for multilayered geotextile-reinforced systems.

An iterative numerical procedure is used to obtain solutions. Results are presented for a uniformly distributed load, and typical values of soil and geotextile properties. The efficiency of the geotextile increases as the subgrade underlying the membrane becomes more compressible, as long as the geotextile is sufficiently strong and possesses sufficient frictional resistance.

Un nouveau modèle est présenté pour analyser l'interaction entre un sol et une membrane géotextile. Il est proposé une interprétation qualitative du rôle mécanique de la membrane, qui utilise un concept probabilistique pour la diffusion des contraintes verticales dans un milieu particulaire (Harr, 1977). Une généralization est e'galement efferte pour des systèmes renforcés par plusieurs membranes géotextiles.

Une methode numérique itérative est utilisée pour obtenir ces solutions. Des résultats sont présentés pour une charge distribuée uniformément et des valeurs courantes des propriétés du sol et du géotextile. L'efficacité du géotextile accroit avec la compressibilité du sol sous-jacent à la membrane, a condition que le géotextile soit suffisamment resistant et possède une resistance à la friction suffisante.

INTRODUCTION

In order to describe theoretically the mechanical behavior of a layered soil system reinforced by a geotextile, the entire range of physical properties of the soil, geotextile, and the load must be considered. Long before modern reinforced earth concepts were born, the problem of a soft elastic material reinforced by closely spaced flexible but inextensible sheets was solved by Westergaard (1). The solution was later extended by Harrison and Gerrard (2), but very few other closed-form analytical solutions have been formulated. Most of the recent related work applied numerical methods wherein the soil-reinforcement system was represented by an equivalent homogeneous or a discrete finite element model, e.g., Herrmann (3), Hausmann and Vagneron (4), and McGown, et al., (5). Recently, a practical design procedure for unpaved roads reinforced with geotextiles was proposed by Giroud and Noiray (6).

In spite of this progress, it is felt that a better understanding of the problem can be reached by taking into account such factors as, e.g., the extreme heterogeneity of the subgrades on which the geotextiles are typically used (Leflaive, 7) and the particulate nature of the soils. The objective of the present study was to consider this latter factor for the two-dimensional, plane strain case.

MATHEMATICAL DEVELOPMENT

The conventional approach to assess the transmission of boundary loadings through soils computes stresses using the linear theory of elasticity, the "Boussinesq solution". Such solutions require the existence of a continuum. Harr (8), taking cognizance of the particulate nature of real soils, introduced the concept of "distribution of stress at a point". Given a unit vertical load acting normal to the surface of the soil (Fig. 1a) the vertical normal stress $S_z(x, z)$ at point (x, z) was found to be a Poisson variate, defined by the porosity of the soil, with an expected value

$$\overline{S}_z(x,z) = \frac{1}{z\sqrt{2\pi\nu}} \exp\left[-\frac{x^2}{2\nu z^2}\right] \tag{1}$$

ν is called the "coefficient of lateral stress"; conceptionally, it is related to the coefficient of lateral earth pressure. $\overline{S}_z(x, z)$ is the solution of the diffusion equation:

$$\frac{\partial \overline{S}_z}{\partial z} = D\frac{\partial^2 \overline{S}_z}{\partial x^2} \quad \text{with} \quad D = \frac{\nu_z}{2} \tag{2}$$

and it is seen to be a normal variate with an equivalent mean value of zero and a standard deviation of $z\sqrt{\nu}$.

For a unit pressure uniformly distributed over a strip of width 2a (Fig. 1b) Harr obtained

$$\overline{S}_z(x,z) = \psi\left(\frac{x+a}{z\sqrt{\nu}}\right) - \psi\left(\frac{x-a}{z\sqrt{\nu}}\right) \tag{3}$$

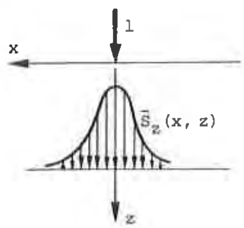

Fig. 1 a

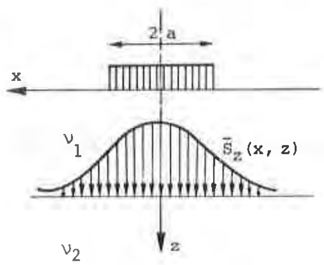

Fig. 1 b

where:

$$\psi\left(\frac{x\pm a}{z\sqrt{\nu}}\right) = \int_0^{\frac{x\pm a}{z\sqrt{\nu}}} \frac{1}{\sqrt{2\pi}} \exp\left[-\frac{t^2}{2}\right] dt$$

The foregoing results were extended to a multi-layered system (Fig. 2). Defining the equivalent thickness $\tilde{h}_1$, of the first layer as:

$$\tilde{h}_1 = h_1\sqrt{\frac{\nu_1}{\nu_2}}$$

The expected vertical stress at any point (x, z) within the second layer is

$$\bar{S}_{2_z}(x,z_2) = \frac{1}{(\tilde{h}_1+z_2)\sqrt{2\pi\nu_2}} \exp\left[-\frac{x^2}{2\nu_2(\tilde{h}_1+z_2)^2}\right] \quad (4)$$

For the n^{th} layer,

$$S_{n_z}(x,z_n) = \frac{1}{\tilde{h}_{n-1}+z_n}\sqrt{\frac{1}{2\pi\nu_n}} \exp\left[-\frac{x^2}{2\nu_n(\tilde{h}_{n-1}+z_n)^2}\right] \quad (5)$$

where:

$$\tilde{h}_{n-1} = h_1\sqrt{\frac{\nu_1}{\nu_n}} + h_2\sqrt{\frac{\nu_2}{\nu_n}} + \dots + h_{n-1}\sqrt{\frac{\nu_{n-1}}{\nu_n}}$$

It should be noted that the above were obtained without the necessity of specifying interface boundary conditions; further, the medium is a "no-tension" material.

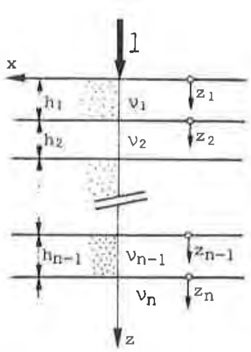

Fig. 2

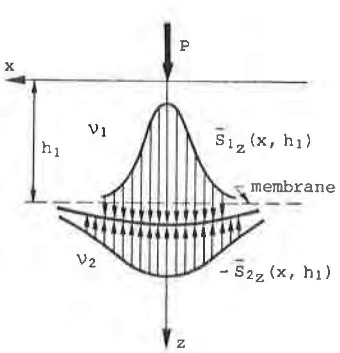

Fig. 3

Consider that a geotextile membrane (of infinite extent) exists at the interface of two layers, Fig. 3. The layers are characterized by their coefficients ν_1 and ν_2. A line load of intensity P acts normal to the surface, and it induces a tensile force in the membrane. The membrane is assumed to be of negligible thickness and offers no bending resistance. Assuming small deflections and slopes, vertical equilibrium requires:

$$\int_{-\infty}^{+\infty} \bar{S}_{1_z}(x,h_1)\, dx = \int_{-\infty}^{+\infty} \bar{S}_{2_z}(x,h_1)\, dx = P$$

Noting that the membrane is, in effect, equivalent to a very thin layer, say of thickness between $h_1 - \varepsilon$ and $h_2 - \varepsilon$, where ε is very small, Eqn. (1) specifies at depth of $z = h_1$, at the top of the membrane,

$$\frac{\bar{S}_{1_z}(x,h_1)}{P} = \frac{1}{s_1\sqrt{2\pi}} \exp\left[-\frac{x^2}{2s_1^2}\right] \quad (6)$$

where $s_1 = h_1\sqrt{\nu_1}$ is equivalent to the standard deviation, with respect to x, in the first layer. At the top of the second layer, a similar argument produces

$$\frac{\bar{S}_{2_z}(x,h_1)}{P} = \frac{1}{s_2\sqrt{2\pi}} \exp\left[-\frac{x^2}{2s_2^2}\right] \quad (7)$$

where $s_2 = \approx{h}_1 \cdot \sqrt{\nu_2}$ is the equivalent to the standard deviation, with respect to x, in the layer below the membrane, at depth $z = h_1$.

We define the ratio of the equivalent standard deviations as

$$k = \frac{s_1}{s_2} = \frac{\bar{S}_{2_z}(0,h_1)}{\bar{S}_{1_z}(0,h_1)} = \frac{\bar{S}_{2_z}(x,h_1)_{max}}{\bar{S}_{1_z}(x,h_1)_{max}} \leq 1$$

where $\tilde{h}_1$ is the equivalent thickness of the system without the membrane, $\tilde{h}_1 = h_1\sqrt{\nu_1/\nu_2}$. In concept, $(1 - k) \cdot 100(\%)$ is the effective reduction of the thickness of layer 1 (the upper layer) due to the membrane for a given state of stress $\bar{S}_z(x, h_1)$. Fig. 4 presents a plot of the developed relationships.

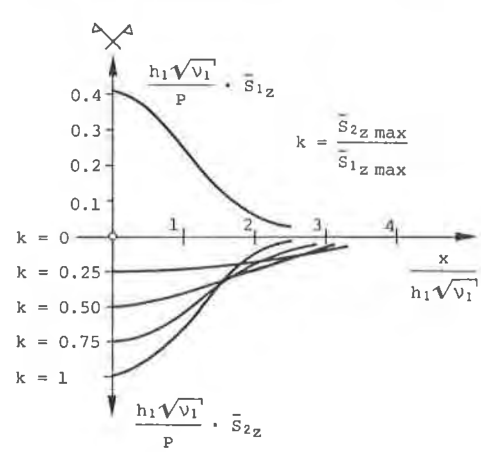

Fig. 4

Numerical values of the ratio "k" are yet to be determined. Its measure is seen to depend upon the deformability of the geotextile and of the underlying soil. Following Rosner and Harr (9) and Nieuwenhuis (10), the underlying soil (layer 2) is taken to offer a reaction to loading proportional to its deflection (Winkler model); that is,

$$\bar{S}_{2_z}(x,h_1) = K_s \cdot w(x) \tag{8}$$

where w(x) is the deflection of the membrane and K_s is the coefficient of subgrade reaction (Fig. 5).

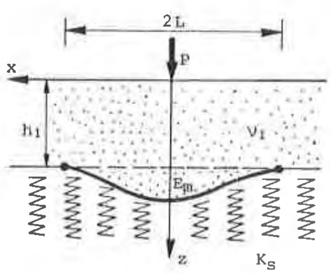

Fig. 5

Consider now the equilibrium conditions of the membrane as shown in Fig. 6. The respective tangential stresses $\tau_1(x)$ and $\tau_2(x)$ act to oppose the direction of the displacement of the membrane. Equilibrium specifies that

$$T_H(x)\,\frac{d^2w}{dx^2} + \tau(x)\,\frac{dw}{dx} + K_s \cdot w = \bar{S}_{1_z}(x,h_1) \tag{9}$$

where $\tau(x) = \tau_1(x) + \tau_2(x)$

and $T_H(x) = T_o - \int_0^x \tau(x)\,dx$

The deformation condition is

$$\int_0^L \sqrt{1 + \left(\frac{dw}{dx}\right)^2}\,dx - L = \frac{1}{E_m}\int_0^L T_H(x)\sqrt{1 + \left(\frac{dw}{dx}\right)^2}\,dx \tag{10}$$

where E_m is the elongation modulus $[kN \cdot m^{-1}]$ of the membrane of width 2L, T_o is the tension in the membrane.

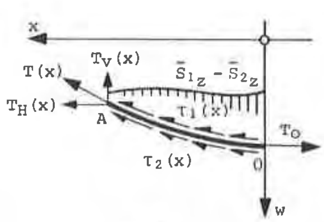

Fig. 6

Model wall tests by Holtz and Broms (11) and pull-out tests performed with woven polyester fabrics (Holtz, 12) have shown that full frictional resistance of the geotextile is achieved with only small displacements. Accordingly, a rigid-perfectly plastic friction law was adopted to represent the behavior at the soil-fabric interface. Thus, a Mohr-Coulomb yield criterion is invoked for the frictional stresses τ_1 and τ_2 (Fig. 7).

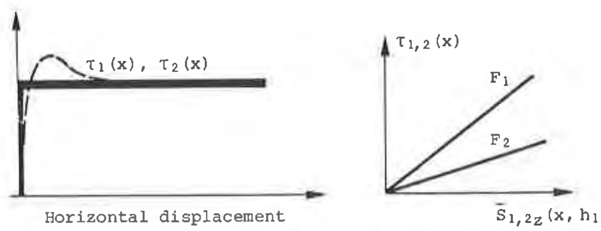

Fig. 7

That is, the frictional resistance is in proportion to the resultant normal (vertical) stress at the point in question.

If $\varepsilon(x) \neq 0$ (some strain exists)

$$\begin{aligned} \tau_1(x) &= F_1\left[\bar{S}_{1_z}(x,h_1) + \gamma \cdot h_1\right] \\ \tau_2(x) &= F_2\left[\bar{S}_{2_z}(x,h_1) + \gamma \cdot h_1\right] = F_2\left[K_s \cdot w(x) + \gamma \cdot h_1\right] \end{aligned} \tag{11}$$

Consequently, $T_H(x)$ and $\tau(x)$ in Eqn. (9) are dependent on w(x). F_1 and F_2 are corresponding coefficients of friction.

The finite difference form of Eqn. (9) is:

$$T_{H_i} w_{i-1} + (K_s h^2 - 2T_{H_i} + h\tau_i)w_i + (T_{H_i} - h\tau_i)w_{i+1} = h^2 (S_{1_z})_i \tag{12}$$

In matrix notation this can be written as

$$[B] \cdot w = \bar{S} \tag{13}$$

where [B] is a tridiagonal matrix.

A numerical solution for the system was obtained using the following iterative algorithm with "N" nodal points (h = spacing) per half-length "L" selected along the membrane (Fig. 8). It is assumed that F_1, F_2, L, P, ν_1, K_s, γ and h_1 are given.

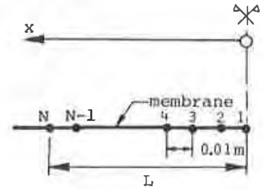

Fig. 8

1. At each nodal point, calculate $\bar{S}_{1z}(x,h_1)$ from Eqn. (6) and $\tau_1(x)$ from the first of Eqns. (11).

2. Form the $[B]$ matirx, Eqn. (13), with $\tau_2(x) = 0$, T_{H_i} = constant, and solve the linear system for the w's.

3. Check if the deformation condition, Eqn. (10), is satisfied. If not, change the value of T_{H_i} and repeat step 2.

4. Having satisfied step 3, calculate the resulting values of $\tau_2(x)$.

5. Form the new matrix $[B]$ using the new values of T_{H_i} and $\tau(x)$ and return to step 2.

6. The solution is achieved when the deflection at x = 0, w(x = 0) converges to a constant value. When this is realized $T_H \rightarrow T_o$, which can then be compared to the maximum tensile strength (T_{max}) of the geotextile membrane.

DISCUSSION OF RESULTS

The above procedure was programmed and some results were obtained for typical values of soil and geotextile properties to illustrate the relative influence of the various parameters. These results are shown in Figs. 9a and 9b and Fig. 10. Solutions were obtained for both a frictionless membrane (case I, shown dotted) and one that displays friction (case II, shown solid). The frictionless case has no relevancy to common geotextiles. However, it was obtained in the early stages of the study to aid in the development of the numerical algorithm. On the other hand, the frictional case is of considerable practical importance. In these plots the curves marked (1 - k) represent an "improvement factor", $1 - k = 1 - (w_{max}/w_m)$, where w_{max} is the maximum deflection with the membrane and w_m is the maximum deflection without the membrane. It is noted that the efficiency of the membrane increases, as measured by 1 - k, as the underlying soil becomes more compressible (smaller values of K_s). For heavy loads, the factor (1 - k) can exceed 50%; however, as shown, the geotextile must also be able to sustain large tensile forces (T_m). At the present time, it appears that only heavy woven geotextiles and geogrids can meet such requirements for the cases shown.

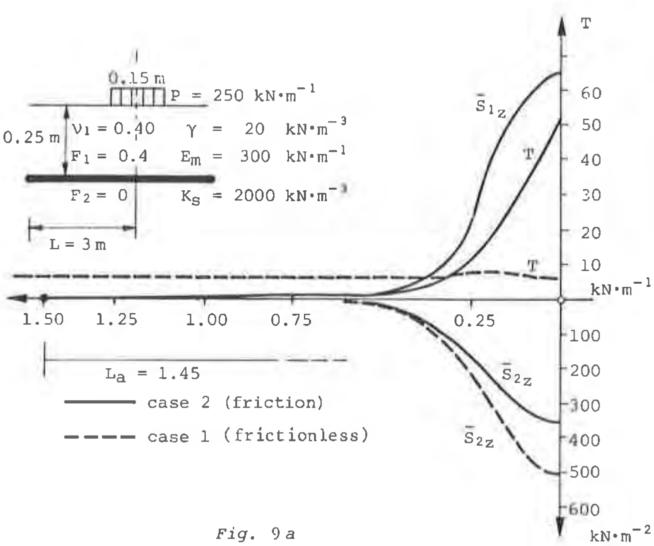

Fig. 9a

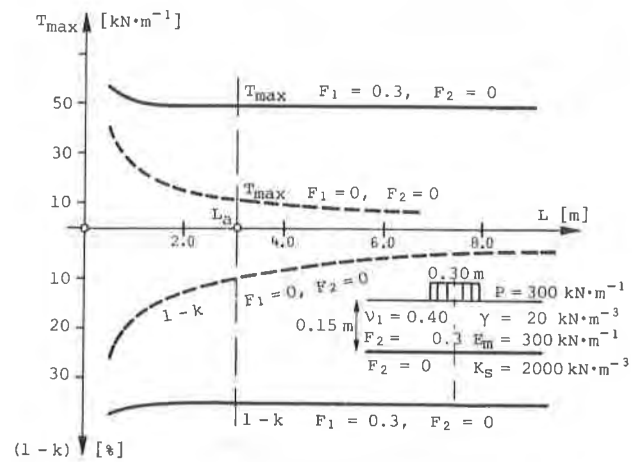

Fig. 9b

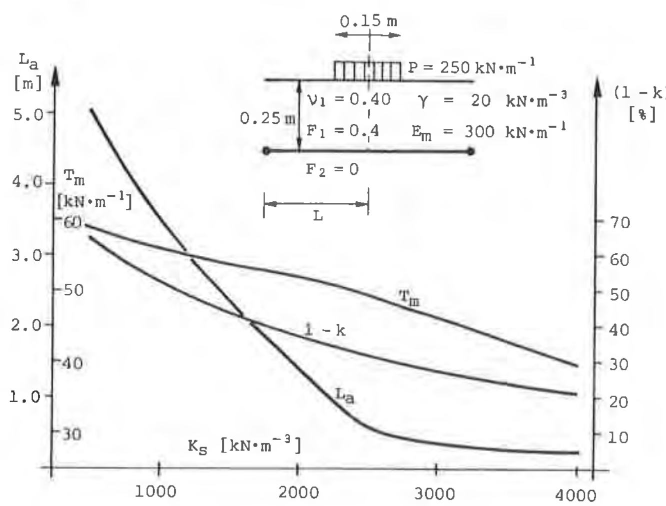

Fig. 10

The required length of fabric (L_a) is seen to decrease rapidly as the friction increases and the underlying soil becomes stiffer.

Finally, it should be noted that the developed model does not consider failure induced by the lack of adequate bearing capacity. However, it does indicate two important possible modes of distress, (1) that of the tensile force T_m exceeding the tensile strength, and (2) the slipping ("pull out") of the fabric if its length L is less than the required value of L_a.

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Calculation Method for a Fabric Reinforced Road

Programme de calcul pour un chemin renforcé par géotextiles

A calculation method has been developed by which the increase of the bearing capacity of a road by geotextile reinforcement can be determined. Different from the existing methods (Nieuwenhuis (1), Bakker (2), Ludwig (3), Barenberg (4), Giroud and Noiray (5), the method presented here fulfills the equilibrium conditions of the geotextile and the subgrade simultaneously. The geotextile is described by a membrane type differential equation: the subgrade equations include a bilinear elastoplastic soil model, in which an elastic analysis based upon a modulus of subgrade reaction is coupled with a rigid plastic analysis based upon a Brinch Hansen type characteristics method. The calculation method is programmed for a pocket calculator. This enables determination of the aggregate height and the required geotextile strength as a function of the properties of aggregate and subgrade, traffic conditions and rutting.

Une méthode de calcul a été développée pour déterminer l'augmentation de la capacité portante d'une route, en utilisant un géotextile. Autrement que les méthodes existantes (Nieuwenhuis (1), Bakker (2), Ludwig (3), Barenberg (4), Giroud et Noiray (5), la présente méthode assure simultanément les conditions d'équilibration du géotextile et du sol. Le géotextile est représenté par une équation différentielle type membrane: les équations du sol comportent un modèle de sol élasto-plastique bilinéaire, dans lequel une analyse élastique, basée sur un module de réaction du sol est couplée à une analyse plastique rigide, basée sur une méthode caractéristique type Brinch Hansen. La méthode de calcul a été programmée pour un ordinateur de poche et permet la détermination de la hauteur du remblai et de la résistance requise du géotextil en fonction des propriétés du remblai, du sol, des conditions de circulation et des ornières.

Introduction.

The main functions of geotextiles in high deformation, low volume road construction are assumed to be separation, filtration and reinforcement. Separation is defined as the prevention of aggregate penetration into the subgrade, while filtration is defined as the prevention of subgrade migration into the aggregate. Reinforcement is defined as the action of the geotextile that causes a change in the stress state in the road profile. This paper deals only with this function.
Two types of reinforcement are distinguished: Lateral restraint and membrane action.
Lateral restraint is the restraint of lateral movement of the aggregate over the geotextile. Consequently, the aggregate shows a stiffer behaviour and spreads the traffic load over a wider area. Deformations of the road and stresses and strains in the geotextile are small. The stress-strain properties of the geotextile are rather irrelevant. Lateral restraint is an interaction between the aggregate and the geotextile. It is assumed to be predominant in roads on rather good subgrade.
The second type of reinforcement is predominant in roads on a very weak subgrade. During rutting the fabric is strained like a membrane. The traffic load on the subgrade is spread over a larger area by the vertical components of the stresses induced in the geotextile. The membrane type reinforcement is an interaction between subgrade and geotextile. It is more effective if the geotextile has a high modulus of elasticity.
Separation and filtration are well accepted functions of the geotextile. The reinforcement function however used to be more doubtful and less defined. In many test sections reported in literature - e.g. Wilmers (7), Grossman (8), Heijnen and Lubking (9) - it seems to be of minor importance. Other authors - e.g. Blumer (10), Webster and Watkins (6), Webster and Alford (11), Barenberg (12), Le Flaive (13) - find a certain reinforcement which is more distinct as the geotextile is stiffer and the subgrade weaker. Kinney and Barenberg (14), derived similar results from laboratory tests. Potter and Currer (15) performed tests in which the membrane reinforcement was eliminated for the greater part by a special test set-up. They found differences between the test section and a reference section, which may be attributed to lateral restraint effects. Haliburton (16) gives a description of lateral restraint and makes a first step to a physical description.
A mathematical description has not been given yet.
The mathematical description of the membrane type reinforcement was first attempted by Nieuwenhuis (1). Nieuwenhuis solved the equilibrium equations of the geotextile (the

membrane equation), but did not give an adequate description of the behaviour of the subgrade. Bakker (2) and Ludwig (3) have a better description of the behaviour of the subgrade, which they assumed to be in a rigid plastic state. However, they did not fulfill the equilibrium conditions of the geotextile. They assumed a rather arbitrary shape of the deformed geotextile, which they gave a prescribed type of deformation.
Giroud and Noiray (5) used essentially the same method, but made a rather good assumption of the type of deformation of the geotextile, as will be shown in the following sections. All authors of mathematical models conclude on a theoretical basis that the application of high modulus geotextiles with sufficient strength may lead to considerable savings or improvement in road construction. The same conclusion was drawn by Barenberg (12), who determined design graphs for low and high modulus geotextiles on a more empirical base. In 1975 Barenberg (11) made already design curves for low modulus geotextiles. They were based on the empirical assumption that a soil-geotextile-aggregate system will only show large deformations at the point of general shear, while roads without a geotextile will start to show large deformations at the point of local shear. Therefore a higher bearing capacity factor (6.2 instead of 3.14) may be taken into account. Steward (17) improved this theory slightly in 1977.
Most of the "reinforcing" effects of this approach must be attributed to lateral restraint and separation. Accurate interpretation of the results of Webster and Watkins (6) confirms this last statement.
It is likely that the negative findings of many authors with regard to the reinforcement function of a fabric can be explained by the fact that not real stiff and strong fabrics were used in their test sections.
Because of a distinct feeling that considerable savings were possible by the use of high modulus geotextiles, Nicolon B.V. decided to introduce a range of high modulus wovens, the so called Geolon® wovens. At the same time it was felt necessary to have a sophisticated and safe design method, since these wovens were meant to be essentail structural elements.
The Delft Soil Mechanics Laboratory was committed to develop such a design method and to determine specifications for the Geolon® range.

Calculation method.

The calculation method is mainly set up for low volume roads for which the construction phase is determinative. Figure 1 shows a cross section over such a road reinforced with a geotextile and surcharged by an axle load.
The calculation is actually made for the situation of a truck driving in the centre of the road. However, the solution with minor adeptions is succesfully applied to excentrically loaded roads too.
The calculation is kept two dimensional, assuming a long queue of trucks. The geotextile is considered to behave like a membrane loaded by the axle load and the subgrade reactions.
During the elastic phase of the subgrade the membrane equation is given by the equations (1).

$$S_0 \frac{d^2w}{dx^2} - kw = -q \qquad S = S_0\sqrt{\left(\frac{dw}{dx}\right)^2 + 1} \qquad (1)$$

where,

S : tensile force in the geotextile
S_0 : horizontal component of S
w : vertical displacement
x : horizontal distance from the truck axis
k : subgrade reaction coefficient
q : membrane loading

Equation (1) is solved for a constant load q_0 distributed by the aggregate; $q_0 = F/2(na+2eH)$. The result is the following:

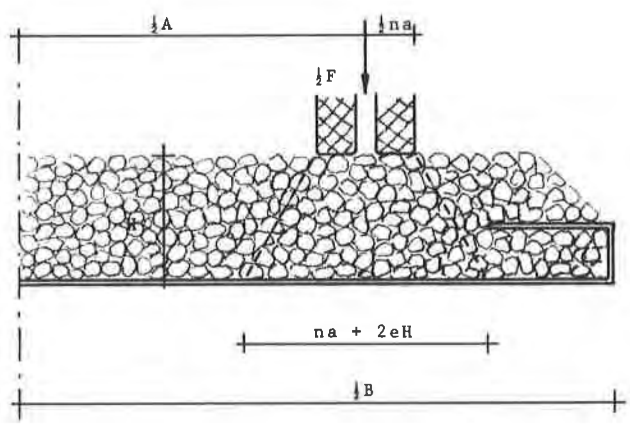
$$\frac{kw}{q_0} = 2\,\mathrm{ch}(\beta-\alpha)\ \mathrm{ch}\xi\ \mathrm{sh}\eta\ /\ \mathrm{sh}\beta$$

$$\frac{kw}{q_0} = 2\,\mathrm{ch}(\beta-\alpha)\ \mathrm{ch}\xi\ \mathrm{sh}\eta\ /\ \mathrm{sh}\beta + 1 - \mathrm{ch}(\xi-\alpha+\eta)$$

$$\frac{kw}{q_0} = 2\,\mathrm{ch}\alpha\ \mathrm{ch}(\xi-\beta)\ \mathrm{sh}\eta\ /\ \mathrm{sh}\beta \qquad (2)$$

with, $\alpha = \frac{1}{2}A\sqrt{k/S_0}$ $\quad \eta = \frac{1}{2}(na+2eH)\sqrt{k/S_0}$
$\beta = \frac{1}{2}B\sqrt{k/S_0}$ $\quad \xi = x\sqrt{k/S_0}$

The first equation holds in $0 < \xi < \alpha - \eta$; the



B : width of the road
F : axle load
A : track
a : tire width
n : number of tires
H : thickness aggregate layer
e : load distribution factor aggregate

Fig. 1 : Cross section over reinforced road

second in $\alpha-\eta<\xi<\alpha+\eta$; the third in $\alpha+\eta<\xi<\beta$.
From solution (2) it is clear that the value of the parameter k/S_o has a strong influence. Its value is relatively large even for the strongest available geotextiles and the softest soils. In that case the displacements outside the area $\alpha-\eta<\xi<\alpha+\eta$ are negligible and therefore the stress state of the subsoil is hardly influenced by the geotextile.
This yields the conclusion that geotextiles will hardly reinforce a subgrade that is in elastic stress state.

If one consideres the soil stresses inside the area $\alpha-\eta<\xi<\alpha+\eta$ it appears that these stresses already reach the plastic state for modest loadings. Therefore a plastic state has to be considered. The membrane equation is then simplified to,

$$S_o \frac{d^2w}{dx^2} = -q \qquad S = S_o\sqrt{\left(\frac{dw}{dx}\right)^2 + 1} \tag{3}$$

where q is the combination of loading and soil stress. The soil failure stress is based on Brinch Hansen's (18) bearing capacity formula,

$$q_l = N_c \left(c + \frac{3}{4} B\tilde{\gamma}\,tg^2\varphi\right)$$

where,

q_l : plastic soil stress
c : cohesion
φ : angle of internal friction
$\tilde{\gamma}$: effective unit weight of subsoil
N_c : bearing capacity factor

The size b of the plastic area is determined by the condition of equilibrium,

$$b = F/2(q_l - \gamma H)$$

with γ being the unit weight of aggregate.
For the location of the plastic area of the soil stresses there are three possibilities. They are given in figure 2. In case a) the area is situated centrically under the axle load. This is the normal situation.
In case b) a normal situation should cause overlapping of the plastic area's at the truck axis. To avoid this the plastic area is placed against the truck axis. In case c) extension outside the road is not feasible. Here the plastic area is placed against the side of the road. Equation (3) is solved for these three cases. However, the solution still contains the unknown parameter S_o. Its value has to be determined by the compatibility condition of the geotextile. Its length is changed by strain due to tensile stresses. On the other hand the vertical displacements cause the geotextile to stretch. If there is no slip and this requires considerable attention of the side anchoring of the geotextile, then strain and increase of length are compatible. This condition is mainly influenced by the Young's modulus E of the geotextile.
The following solution is determined:

$$\Delta s = \frac{1}{4}\,\frac{b - na - 2eH \pm d}{b - na - 2eH}\;\frac{1}{\lambda}$$

$$\lambda^3 = \frac{2E}{3q_l B}\;\frac{(b - na - 2eH)^2 + 3d^2}{b^2} \tag{4}$$

$$S_m = \tfrac{1}{2} q_l b \sqrt{\left\{\frac{b - na - 2eH \quad d}{b}\right\}^2 + \lambda^2}$$

where,

Δs : rut depth
E : Young's modulus
S_m : maximum tensile stress
d : location parameter of plastic area

d=0	b<A	b<B-A
d=B-A	b>A	b<B-A
d=B-A-b	b<A	b>B-A

The solution supplies for $d\neq 0$ two values of Δs and S_m. This is due to the fact that the settlements at both sides of the wheel for $d\neq 0$ are not equal. This is noticed in the test cases.

The result shows the rut depth as function of all the introduced parameters. On the other hand, one may assume a tolerable rut depth and then determine the minimum required Young's modulus and strength of the geotextile. A calculation programme is made on a pocket calculator to determine this.

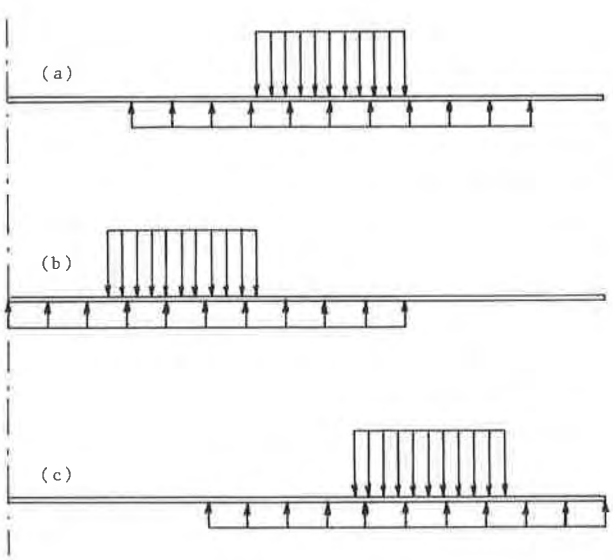


Fig. 2 : Location of the plastic area

Calculation programme.

The calculation method described above has been programmed on a pocket calculator. In this interactive programme the following input is asked for:

1. width of the road (B)
2. tire width (a)
3. track (A)
4. number of wheels
5. number of trucks
6. axle load
7. load distribution factor of the aggregate

8. unit weight of the aggregate
9. effective unit weight of the subgrade
10. undrained shear strength
11. angle of internal friction

Output consists of all possible combination of the height of the aggregate (H) and Young's modulus of the geotextile (E), which are given for a certain rut depth. As a standard rut depth three times the elastic rutting is chosen, but other values for the total rut depth can be put in optionally. Apart from E and H which are the main design parameters, information is given about the width of the plastic zone b and the maximum stress in the geotextile S. The latter value should be checked against the tensile strength of the geotextile. An example of the input and output of the programme has been given in fig. 3.

Design charts.

A great number of calculations has been compressed into design charts (see fig. 3 and 4). In one type of design charts the relative aggregate saving H/H_o is given versus the undrained shear strength of the soil (Cu) H_o is the weight of aggregate that is required without the use of a geotextile. This reduction is based on the additional load spreading of the geotextile. So with a lower aggregate height the same stress will be applied on the subgrade as with the original aggregate height H_o. At low Cu values this saving parameter H/H_o is going to be undetermined, as the road cannot be constructed without a geotextile. A second type of design chart gives the additional shear strength of the subgrade (Cu) versus the original shear strength (Cu). The additional shear strength is the apparent improvement of the subgrade due to the use

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   FABRIC REINFORCED
   ROAD CONSTRUCTION

WIDTH ROAD    4,00 m (B)
WIDTH TIRE    0,20 m (a)
TRACK         1,70 m (A)
NUMB WHEELS   2      (n)

NUMB TRUCKS   1
AXLE LOAD   100 KN   (F)
LOAD DISTR    0,50   (e)

HEIGHT AGGREG : H [m]
FABR YOUNG MOD: E [KN/m]
FABR MAX STR    S [KN/m]

condition       s <= b/4
RUT DEPTH       s [m]
elast (el)    plast (pl)
WIDTH PL ZONE   b [m] 2*

=========================

VOL W AGGREG 19,0 KN/mt3
VOL W EFF     7,0 KN/mt3
UNDR SH STRG 15,0 KN/mt2
ANG INT FRIC  0,0 DEGREE

      s(el) = 0,046 m
      s(pl) = 0,138 m
      s     = 0,184 m

H     b     E      S

0,26  1,51  2083  125
0,28  1,45  1753  109
0,30  1,40  1475   95
0,32  1,35  1241   83
0,34  1,31  1041   72
0,36  1,27   871   62
0,38  1,23   725   53
0,40  1,20   599   45
0,42  1,17   498   38
0,44  1,14   396   31
0,46  1,11   313   26
0,48  1,08   241   20
0,50  1,06   177   15
0,52  1,03   122   11
0,54  1,01    72    6
0,56  WEDGE WITHIN  ROAD
0,56  0,99    29    3
0,58  ELAST DISPLMT ONLY
1,00  ELAST DISPLMT ONLY
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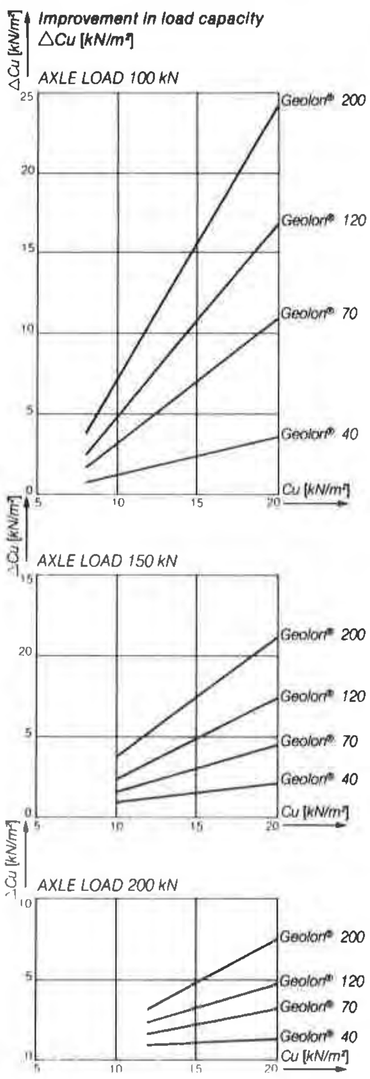

Fig. 3

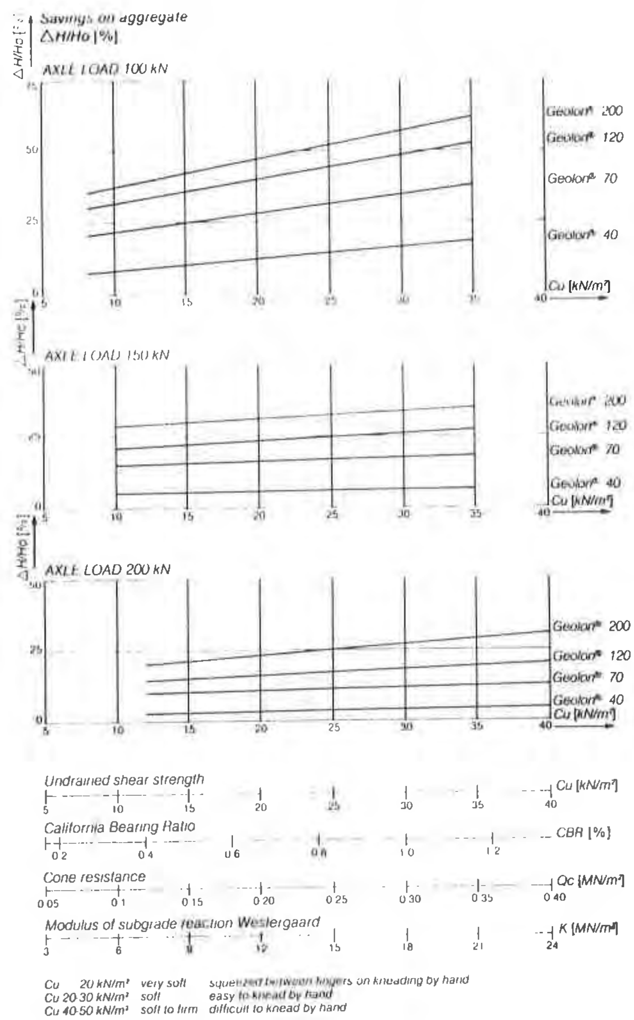

Fig. 4

of a geotextile. In this way the influence of a geotextile may be included in the usual design methods.
Both types of design charts have been given for 3 axle loads (100 kN, 150 kN and 200 kN respectively) and the four strongest wovens from the Geolon® range.

Comparison of GEOL-savings with full scale tests of Webster & Watkins.

Webster & Watkins have done full scale tests on an unpaved road with a.o. three different sections. One section with a woven membrane with E = 200 kN/m, one section with a non woven with E = 70 kN/m and one control section without a geotextile. These sections are respectively mentioned item 7,6 and 5.
The road sections were loaded by a truck with 80 kN axle load and graphs were made of the number of coverages versus rut depth.
The following data are listed for rut depths of 0,28, 0,15 and 0,075 m:

S.1 - The GEOL-savings based on calculations with the really occured axle loads.
S.2 - The GEOL-savings based on calculations with the equivalent standard axle of 80 kN.
S.3 - The savings as determined by Webster and Watkins in relation to a design formula.
S.4 - The savings as determined by Webster and Watkins in relation to a design formula, but now diminished by the performance savings of the section without geotextile in relation to the same design formula.

These field tests are not representative for GEOL, which is based on the construction phase of a road and not for a number of coverages exceeding 20.000. However, an attempt comparison has been made in table 1 and 2.

Evaluation.

1. The savings calculated with GEOL, based on stress reduction on the subgrade alone, are for most of the cases on the safe side, compared with the performance savings of Webster and Watkins.
2. The trend that a greater rut depth corresponds with a greater saving based on stress reduction due to membrane type support only, exists in item 6 of Webster and Watkins and not in item 7.

Field test Study Centre for Road Construction (SCW).

The Study Centre for Road Construction has conducted a field test on an unsurfaced road in the construction phase using geotextiles. Elongations have been measured with the use of electric strain gauges. From the preliminary results it can be seen that the maximum strain in the geotextile coincides reasonably well with the maximum strain calculated with GEOL. In this field test a non woven is used under an aggregate layer of 50 cm on a subgrade with Cu is about 15 kN/m2. The number of passes is about 250, the axle loads 110 kN.

Further research on field tests.

The next step will be a field test using a Geolon 200 woven fabric in two sections with aggregate thickness savings of 10 and 20 cm with factors of safety 2 and 1. The original design thickness in this location is 50 cm which will be represented by a section without a geotextile.

Number of coverages.

One of the most interesting parameters is the number of coverages for which the physical in situ conditions meet the output of the calculation programme. These physical conditions are aggregate thickness, rut depth and load characteristics. For the woven membrane section of Webster and Watkins the number of coverages of the 80 kN axle is about 10.000 and for the non woven section about 600.
For the non woven sections of the field tests of the Dutch Study Centre for Road Construction the number of coverages of an 80 kN axle is also about 600. This seems to indicate that the number of coverages needed to meet the in situ conditions is related to the elastic modulus of the fabric.

Evaluation.

These numbers of coverages show that the calculation programme is at least valid during the construction phase of the road.

Conclusions.

1. The here presented calculation method seems to be a good and safe design method for low volume roads which are loaded most heavily during construction.
2. The use of high modulus geotextiles may lead to considerable savings or improvement in low volume road construction.
3. The absolute saving of aggregate increases with the value of the modulus of elasticity of the geotextile. This increase faints off at higher values of E. The use of geotextiles with moduli over 1500 kN/m makes little sense considering savings on aggregate due to membrane type reinforcement.
4. The absolute saving of aggregate decreases with increasing strength Cu of the subsoil. The use of a geotextile on subgrades with a Cu value exceeding 30 kN/m2 has little effect as a structural reinforcement membrane.
5. The modulus of elasticity of the geotextile is the most determining factor in aggregate saving. However, it should be checked that the geotextile has sufficient strength.
6. Using a geotextile as a structural reinforcement, proper design calculations are absolutely necessary to avoid mishaps.

Table 1

	S.1			S.2		
	rut depth					
	0,28	0,15	0,075	0,28	0,15	0,075
	Savings in % of the design thickness					
item 6	30	9	4	30	9	1
item 7	37	11	3	56	17	3

Table 2

	S.3			S.4		
	rut depth					
	0,28	0,15	0,075	0,28	0,15	0,075
	Savings in % of the design thickness					
item 6	27	20	18	21	12	10
item 7	48	49	52	42	41	44

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Cyclic Loading of a Two Layer Soil System Reinforced by Geotextile

Chargement cyclic d'un bicouche renforcé par géotextile

For a best understanding of the behaviour of a geotextile material set under the subbase of a temporary road on a soft subgrade, comparative experiments related to quasi static and cyclic loading on laboratory models are investigated in the present work. It is shown that a geotextile material is very performing when the cyclic loading is higher than the bearing capacity of the unreinforced soil : the study also outlines the importance of the frequency (rate of the truck) and of the loading-level.

A rheological simulation of the geotextile behaviour is then proposed for a repeated traction loading and the more complex case where the geotextile is considered as an anchored membrane under a cyclic loading.

Dans le but de mieux comprendre le fonctionnement d'un géotextile placé sous la couche de forme d'une chaussée provisoire sur sol mou, les auteurs présentent une série d'expériences comparatives de chargements quasi-statique et cyclique sur modèle réduit : on montre ainsi qu'un géotextile est très performant lorsque la charge cyclique appliquée est supérieure à la force portante du sol non renforcé ; on montre aussi l'influence de la fréquence (vitesse des véhicules) et de la charge appliquée.

Parallèlement les auteurs proposent un modèle mécanique simulant le comportement du géotextile soumis isolément à un effort de traction répété et le comportement plus complexe du géotextile en place agissant cycliquement comme une membrane ancrée.

L'analyse présentée ci-dessous est le prolongement logique de l'étude effectuée à l'Université de Grenoble (1) sur le poinçonnement statique d'un sol bicouche constitué d'une couche de forme en gravette ou sable fondée sur une couche molle d'argile et renforcée par un géotextile à l'interface.

Avant d'aborder le problème cyclique, nous montrerons en quasi-statique la validité du modèle mécanique proposé, en l'appliquant à des études expérimentales.

Dans une seconde partie, nous présentons la démarche suivie afin de modéliser le passage d'une charge roulante sur une chaussée provisoire, et nous exposons les premiers résultats expérimentaux obtenus, le programme d'essais n'en étant que dans sa phase initiale.

La troisième partie est une première tentative pour appliquer le modèle mécanique utilisé en quasi-statique dans le domaine cyclique. Nous ferons appel pour cela aux résultats obtenus par Rigo - Perfetti (2) sur le comportement à la fatigue des géotextiles sollicités en traction

I. CHARGEMENT QUASI-STATIQUE

1. Essais sur modèle réduit :

Nous utiliserons trois essais appartenant au programme de recherche présenté de façon détaillée en (1) : on enfonce une plaque de largeur B=0,15 m à vitesse constante (0,04 mm/s) dans une cuve d'essai à déformation plane. Le bicouche est constitué d'une couche d'épaisseur H=0,10 m de sable (ϕ=34°) ou gravette (ϕ=48°). Le géotextile est un non-tissé thermolié Terram, TM230 (μ=230 g/m^2) ou aiguilleté Bidim, BD550 (μ=550 g/m^2). Leur comportement en traction est défini à partir de l'essai de traction "bande large"(réf.1:fig. 4) : K=80 kN/ml pour le TM230, K = 35kN/ml et K*=75 kN/ml avec T*= 3,5 kN/ml pour le BD 550 suivant le loi en traction représentée à la figure 4.

Le gain de portance expérimentale $\Delta p = p_G - p_O$, est égal à la différence de pression moyenne pour le poinçonnement avec géotextile (p_G) et sans (p_O), à même orniérage de surface r (fig. 1 à 3).

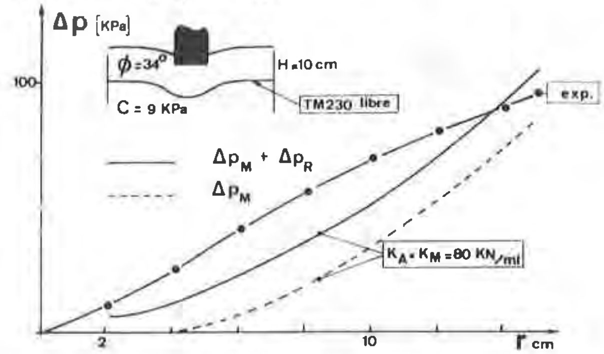


figure 1 : Essai de poinçonnement quasi-statique : gain de portance expérimental et théorique

2. Mécanisme : (fig. 5)

Le gain de portance est attribué à un effet membrane et à un effet répartiteur.

* effet membrane : entre A et I la membrane s'infléchit sous l'effet d'une pression uniforme Δq. L'ouverture θ est déterminée à partir de

$$e(r)/B^* = tg\ \theta/2 \qquad (1)$$

e(r) et B^* sont mesurés expérimentalement, B^* étant pratiquement indépendant de r.

La membrane AI est ancrée suivant AG : la tension T dans la membrane égale à la tension T_A d'ancrage cause un glissement u_A au point A :

. Pour un comportement élastique du géotextile (K, K^*) :

membrane : $T \leq T^*$ $\quad T = K\left(\frac{\theta}{\sin\theta} - 1 - \frac{u_A}{B^*}\right)$ (2)

$T \geq T^*$ $\quad T = K^*\left(\left(\frac{\theta}{\sin\theta} - 1 - \frac{u_A}{B^*}\right) + \left(\frac{1}{K^*} - \frac{1}{K}\right)T^*\right)$ (2')

ancrage : $T \leq T^*$ $\quad T = K.(du/dx)$ (3)

$T \geq T^*$ $\quad T = K^*\left(\frac{du}{dx} + \left(\frac{1}{K^*} - \frac{1}{K}\right)T^*\right)$ (3')

. Loi de frottement en ancrage :

$u \leq u_p$ $\quad \tau = \alpha.u$ (4)

$u \geq u_p$ $\quad \tau = \tau_p = \left(\frac{\mathcal{C} + \sigma_N \operatorname{tg}\varphi}{2}\right)$ avec $\sigma_N = \gamma H$ (4')

$\mathcal{C}$, adhérence textile-sol de fondation cohérent et φ frottement textile-sol pulvérulent de surface.

En intégrant le long de l'ancrage et en vérifiant les conditions aux limites (T membrane = T_A et u_A membrane = u_A ancrage), on obtient la tension T d'équilibre en fonction de r ou θ. On en déduit :

$$\Delta p_M \,.\, B = 2\,T \sin\theta \qquad (5)$$

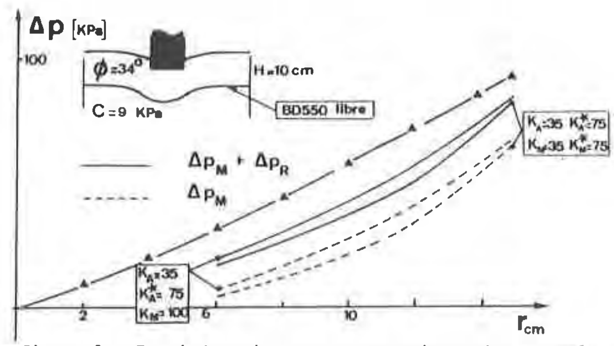


figure 2 : Essai de poinçonnement quasi-statique : gain de portance expérimentale et théorique

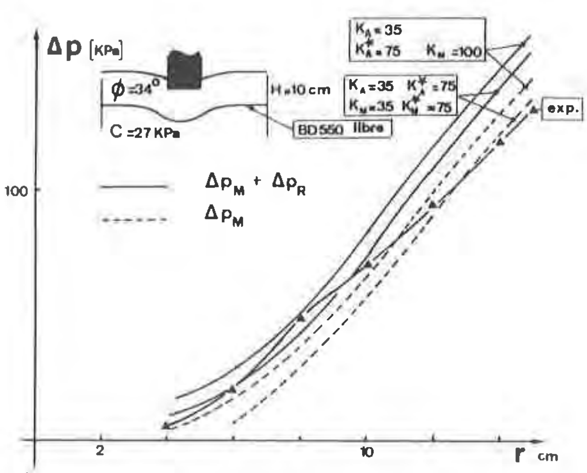


figure 3 : Essai de poinçonnement quasi-statique : gain de portance expérimental et théorique

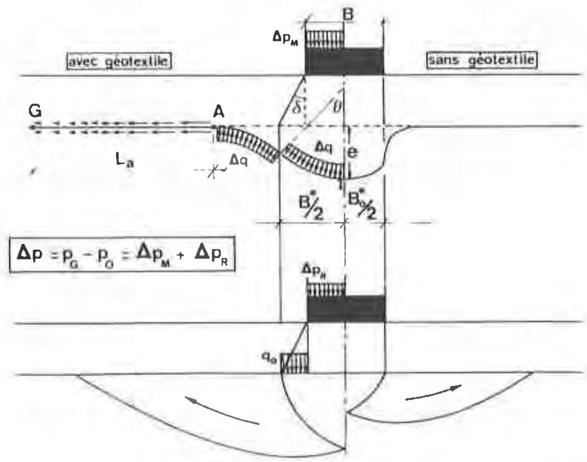


figure 5 : Renforcement par géotextile, mécanisme proposé

* effet repartiteur : La zone en écoulement plastique est visiblement plus importante en présence de géotextile (fig. 6). Considérons que ceci est équivalent à une augmentation de la largeur fictive du poinçon au niveau de l'interface : soit B_0^* sans géotextile et B^* avec géotextile. On supposera indépendante de B^* la pression moyenne $q_0(r)$ de poinçonnement de la couche d'argile :

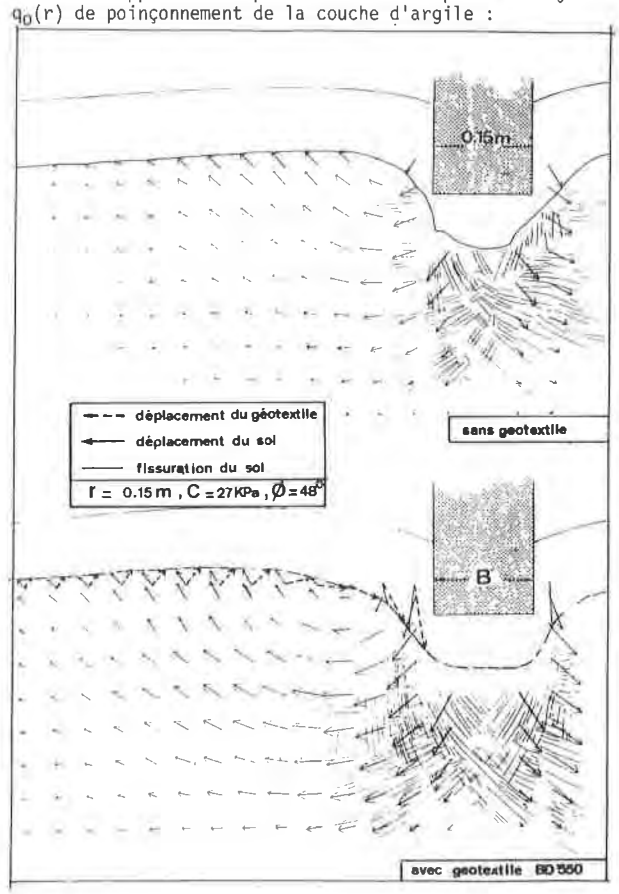


figure 6 : Effet repartiteur d'un géotextile

$$\Delta p_R \;.\; B = q_0 \;(B^* - B_0^*) = p_0 \;.\; B\left(\frac{B^*}{B_0^*} - 1\right) \qquad (6)$$

(p_0B) est la force portante sans géotextile, B^* et B_0^* sont mesurées expérimentalement (distance entre les 2 points d'inflexion principaux de l'interface).

3. Validité du mécanisme : (fig. 1 à 3)
Ce mécanisme permet d'obtenir une bonne estimation du gain de portance : une incertitude réside cependant sur les valeurs des modules K, K^*. Des essais de traction effectués par Mc Gown (3) ont montré que, particulièrement dans le cas des non-tissés aiguilletés, le module de traction augmente avec la compression σ_N normale à la nappe. Ceci nous a amené à considérer dans le cas du BD550 (fig. 2 et 3) un fort module pour la partie en membrane très comprimée : K_M = 100 kN/ml. On constate une faible sensibilité du gain de portance théorique au module en membrane. Par contre (fig. 4) une augmentation du module en ancrage K_A a une influence nettement supérieure sur Δp. Ceci nous amène à une remarque relative aux modèles réduits : dans le cas où l'augmentation importante du module avec la pression de confinement se confirmerait, les non-tissés aiguilletés se trouvent défavorisés en modèle réduit puisque les contraintes normales sont insuffisantes pour mobiliser un module équivalent à celui qu'ils mobiliseront dans un ouvrage réel.

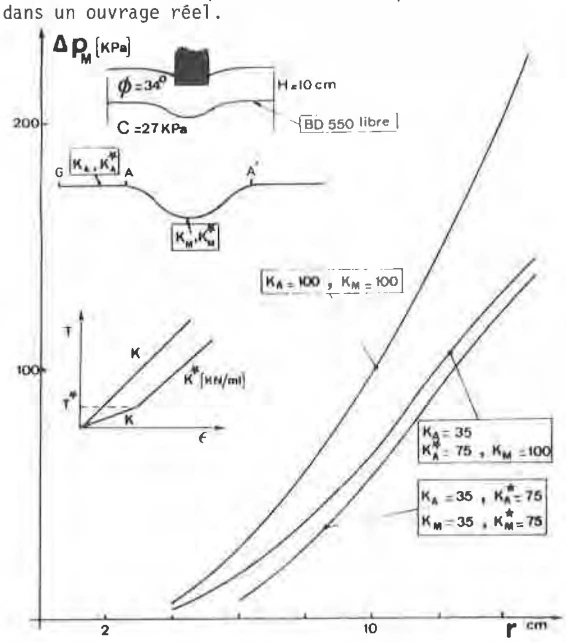


figure 4 : Influence des modules du géotextile sur le gain de portance

II. SIMULATION D'UNE CHARGE ROULANTE SUR CHAUSSEE :

Nous utiliserons la même cuve d'essai que pour les essais quasi-statiques, mais le bâti de chargement est modifié, le verin mécanique est remplacé par un verin hydraulique asservi : il s'agit maintenant de simuler le passage répété de véhicules. L'échelle choisie est $1/L^*$ = 1/3.

1. Structure de chaussée :
Nous reprendrons les conditions vues en (1, §VI) : pour simuler le cas réel d'une couche de forme (H_r=0,45 m) graveleuse (ϕ=48°) sur sol mou (I_{CBR}=1, C_{ur}=27 kPa), nous choisirons une couche de gravette (ϕ=48° = ϕ_r/ϕ^* avec ϕ^* = 1) placée sur une couche d'argile (C_u = 9 kPa = $C_{ur}/C_u^{(*)}$ = 27 kPa/3).

Quant au géotextile, il fallait trouver 2 nappes de caractéristiques de frottement semblables (ancrage) et dont les modules vérifieraient ($K_r/K = K^{(*)2}$ = 9). Ceci n'ayant pu être réalisé, nous avons choisi pour le modèle le Bidim de μ minimale (150 g/m²) : son module K (unique) est égal à 18 kN/ml, ce qui correspond à K_r = 162 kN/ml, géotextile intermédiaire entre le BD550 et le BDrf (réf. 1, fig. 4).

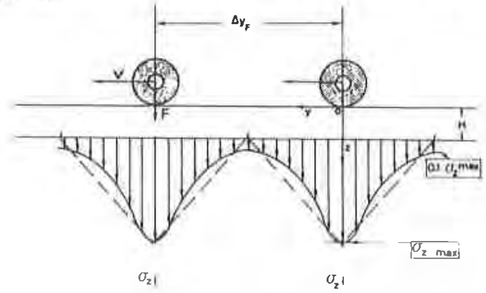


figure 7 : Sollicitation due à une charge roulante

2. Sollicitations :
Soit (fig. 7) Oy la direction du trafic. Notre modèle de cuve d'essai impose ε_y = 0 et la diffusion de la charge ne peut se faire que dans le plan (xOz) perpendiculaire au sens de circulation.

La condition (ε_y = 0) de déformation plane peut être considérée comme vérifiée dans la réalité : l'écoulement plastique du sol se produit préférentiellement dans le plan (xOz) sauf en cas de chargement incliné (freinage, accélération, circulation à sens unique). Dans ces conditions la déflexion du textile est nulle suivant Oy et l'effet membrane peut être considéré dans la plan xOz uniquement.

* Pression maximale appliquée p_m : La charge roulante unitaire F correspond classiquement à la double roue d'un essieu chargé à 130 kN. La surface de contact d'une double roue est égale à : (B_r = 0,45 m) x (l_r = 0,27 m). La pression appliquée $p_{mr} = F/(B_r.l_r)$ = 535 kPa.

La pression correspondante pour le modèle $p_m = p_{mr}/L^{(*)}$ = 180 kPa sur une largeur $B = B_r\ /L(*)$ = 0,15 m.

Cependant en déformation plane, nous avons fait remarquer que la charge se répartit uniquement dans le plan xOz de chargement, alors que dans le cas réel la diffusion se fait suivant toutes les directions : notre modèle de chaussée soumis à une pression maximale p_m sera donc relativement plus sollicité que la chaussée réelle soumise à p_{mr}.

Une autre démarche consisterait à traiter le problème axisymétrique du chargement d'une plaque circulaire en cuve cylindrique, mais dansce cas les inconvénients apparaissent plus importants : le champ de déformation est axisymétrique et en particulier le géotextile fonctionne en membrane suivant toutes les directions, ce qui n'est pas conforme à la réalité et majore l'effet membrane.

Par contre, nous prendrons en compte un second effet de la diffusion de la charge suivant l'axe Oy de circulation :

* Mode de chargement et fréquence : La charge roulante F génère des contraintes σ_z en avant et en arrière de xOz (fig. 7). Un calcul élastique correspondant au chargement uniforme (p_{mr}) suivant une surface ($B_r.l_r$) d'un bicouche (E gravette/E argile fixé) donne σ_z f(x,y,z): dans le plan de symétrie yOz, au niveau de l'interface gravette-argile (z=H), on obtient σ_z<0,10.($\sigma_z$max) à une distance y = 4.5 H suivant Oy. On limitera donc l'influence de F à y= ± 4.5 H et par ailleurs on assimilera la répartition des σ_z à une répartition triangulaire entre y = -4.5 H et y = +4.5H, pour z = H.

Si maintenant au lieu de se déplacer sur Oy à charge fixe, on déplace la charge à une vitesse v_r et que l'on observe l'évolution des σ_z (z=H, y fixé), la contrainte σ_z variera triangulairement en fonction du temps. Le temps d'un cycle charge-décharge sera égal à :

$$\beta_r = \frac{9H_r}{v_r} \quad H_r=0.45\text{ m} \quad v_r=15\text{ km/h} \quad \beta_r \simeq 1\text{ s}$$

Le modèle en déformation plane permettra de simuler les effets d'une charge roulante, à condition d'admettre la proportionnalité entre la pression p appliquée et la contrainte σ_z en un point donné : il suffit alors d'appliquer un chargement triangulaire, de pression maximale p_m correspondant à la pression exercée par une double roue et sur une période $\beta = \beta_r/\beta^* = \beta_r$ fonction de la vitesse v_r de circulation. Nous avons choisi d'exercer des chargements triangulaires sans temps de repos (fréquence $f = 1/\beta$: f = 1Hz pour H_r = 0.45 m et v_r = 15 km/h). Ce cas correspond à 2 essieux chargés à 130 kN distants de $\Delta y_F = 9\ H_r$ = 4,05 m c'est le cas réel le plus critique.

Cependant notre matériel expérimental nous permet d'exercer des chargements de type triangulaire, pour une large gamme en f et p_m (5).

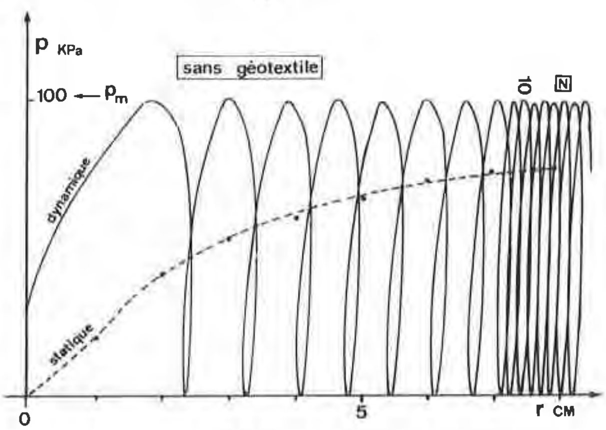


figure 8 : Poinçonnement d'un bicouche sans géotextile

3. Essais de chargement cyclique :

La structure de base étudiée est telle que :
B_r=0,45 m ϕ=48° H_r=0,45 m C_{ur}=27 kPa BD entre 550 et rf

Le modèle réduit semblable présente les caractéristiques

B =0,15 m ϕ=48° H =0,15 m C_u = 9 kPa BD150

Quoique le chargement le plus réaliste puisse être défini par (p_{mr} = 540 kPa f_r= 1Hz (v_r= 15 km/h) simulé par (p_m = 180 kPa, f = 1Hz), nous avons jugé important de faire apparaître l'influence de certains paramètres, le mode de chargement (quasi-statique ou cyclique), la fréquence f ainsi que la pression maximale p_m.

* Comportement général : sans géotextile, cette chaussée (fig. 8) apparaît nettement sous-dimensionnée pour supporter p_m = 180 kPa puisque sous poinçonnement quasi-statique, p = 80 kPa pour r = 0,08 m (échelle 1/3) qui constituera l'ornierage max. toleré. Cependant cette chaussée supporte N = 15 cycles à p_m = 100 kPa, f = 1Hz, avant d'atteindre le même ornierage : l'observation des cycles pf(r) de la fig. 8 prouve que le poinçonnement se poursuit en décharge tant que p est supérieur à α% de la pression portante $p_o(r)$. Le temps pendant lequel la pression cyclique p est supérieure à $\alpha.p_o(r)$ sera donc, avec la valeur de p_m, un paramètre important dont dépendra l'orniérage (fig. 9). Le poinçonnement cyclique du bicouche renforcé (fig. 10) est plus complexe dans la mesure où le géotextile se rigidifie sous cycles et la comparaison au cas statique est donc plus aléatoire.

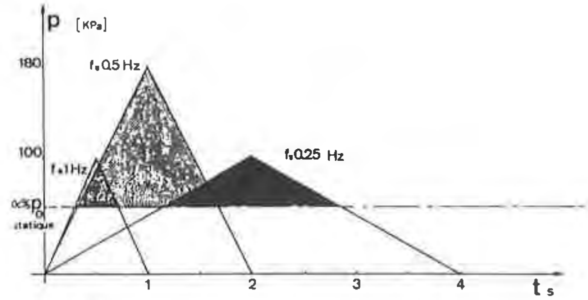


figure 9 : Temps de dépassement de la charge limite statique en fonction de f et p_m

* Influence de différents paramètres : Nous avons, dans le tableau ci-dessous donné le nombre N de cycles atteint pour différents cas de figures et orniérage r=0,08 m

- influence du géotextile :

p_m = 100 kPa	f = 1 Hz	sans géot.	N = 15
p_m = 100 kPa	f = 1 Hz	BD150	N = 3000

- influence de la charge

p_m = 100 kPa	f = 0,5 Hz	BD150	N = 1200
p_m = 180 kPa	f = 0,5 Hz	BD150	N = 4

- influence de la fréquence :

p_m = 100 kPa	f = 1 Hz	BD150	N = 3000
p_m = 100 kPa	f = 0,25 Hz	BD150	N = 800
p = 100 kPa	quasi-statique	BD150	r = 0,065 m

L'influence du géotextile est tout à fait notable (N = 15 ⟶ N = 3000), et quoique ce ne soit pas comparable, beaucoup plus évidente qu'en poinçonnement quasi-statique. Le gain important sur le trafic s'explique bien sûr partiellement par l'effet d'échelle (BD150 correspond à un non-tissé aiguilleté très épais) mais il se justifie surtout par le sous-dimensionnement de la couche de forme : le géotextile est dans ce cas très performant.

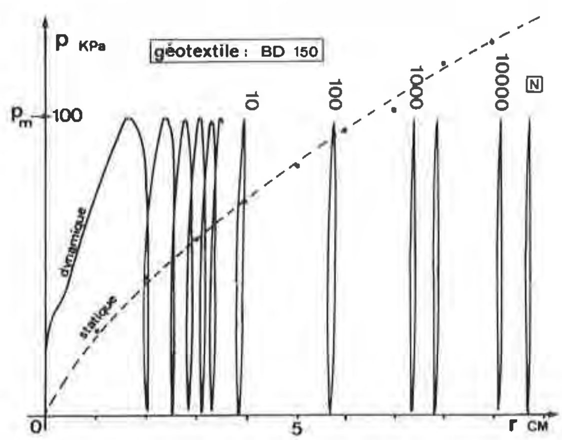


figure 10 : Poinçonnement d'un bicouche avec géotextile

L'influence de la charge (p_m.B) est notable, du moins au-dessus de la capacité portante du bicouche (fig. 11). Le choix de la valeur de p_m en similitude doit donc être réaliste, si l'on veut obtenir des résultats significatifs. L'influence de la fréquence f, qui traduit la vitesse de circulation n'est pas négligeable (fig. 12) : la destruction de la chaussée ira en s'accentuant puisque l'orniérage croissant produira un ralentissement des véhicules.

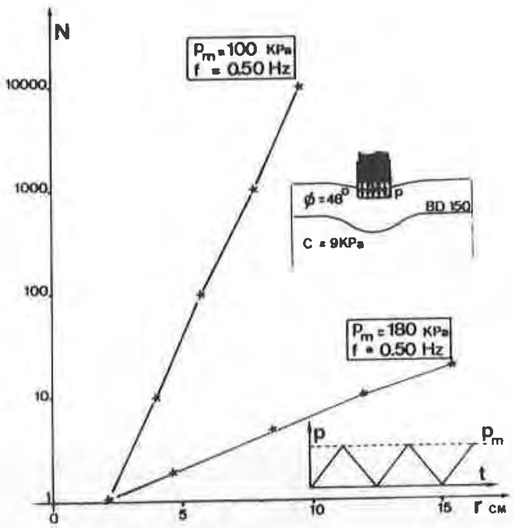


figure 11 : Influence de la valeur de la charge (cyclique)

Ces premiers essais ont déjà donné des résultats intéressants. Rappelons qu'ils s'appliquent au cas peu étudié en calcul de chaussée d'une couche de forme sous-dimensionnée. Dans ce cas le géotextile se montre très utile à la stabilité du bicouche.

Nous allons maintenant exposer une première approche théorique du comportement d'un géotextile dans un bicouche chargé cycliquement. Mais auparavant nous définirons le comportement à la fatigue d'un non-tissé en adaptant à notre problème le modèle rhéologique proposé par Rigo - Perfetti (2, 4).

III. RHEOLOGIE D'UN GEOTEXTILE SOUS TRACTION CYCLIQUE :

Le modèle rhéologique proposé est constitué par :

- un ressort (E_b) associé à une crémaillère, qui bloque le retour élastique : cet élément traduit l'arrangement initial irréversible du matériau,
- un ressort (E_a) correspondant à l'élasticité rémanente du géotextile,
- un ressort (E_c) associé à un dash-pot (viscosité η_c) qui rend compte de l'influence du temps sur le comportement du matériau.

On exerce sur un échantillon de géotextile une traction $T = Tmax/2.(1-\cos 2\pi t/\beta)$ de période $\beta = 2\pi/\omega$: (7)

$t = i.\beta \quad T = o \;;\; t = i\beta + (\beta/2) \quad T = Tmax$

Déformation due à (a + b) :

$0<t<\beta/2 \quad \varepsilon_{a+b} = T/E_a + T/E_b$

$t>\beta/2 \quad \varepsilon_{a+b} = T/E_a + Tmax/E_b$

Déformation due à (c) :

$T = E_c.\varepsilon_c + \eta_c\, d\varepsilon_c/dt$ soit

$$d\varepsilon_c/dt + (E_c/\eta_c)\varepsilon_c = Tmax/2\eta_c \cdot (1-\cos 2\pi \frac{t}{\beta})$$

Une fois résolue cette equation différentielle, on obtient la relation entre la déformation totale $\varepsilon_T = \varepsilon_{a+b} + \varepsilon_c$ à partir de t = o) et le temps t.

$0 < t < \beta/2 \quad \varepsilon_T = T/E_a + T/E_b + \varepsilon_c$

$\beta/2 \quad \varepsilon_T = T/E_a + Tmax/E_b + \varepsilon_c =$ (8)

$$\frac{Tmax}{2E_a}(1-\cos 2\pi\frac{t}{\beta}) + \frac{Tmax}{E_b} + \frac{Tmax}{2E_c}(1-\cos 2\pi\frac{t}{\beta} \cdot \frac{E_c}{E_c+(\omega\eta_c)^2/E_c})$$
$$+ \frac{Tmax}{2E_c} \cdot e^{-E_c t/\eta_c} \cdot (-1+\frac{E_c}{E_c+(\omega\eta_c)^2/E_c}) + \frac{\omega\eta_c \cdot Tmax}{2(E_c^2+(\omega\eta_c)^2)} \cdot \sin 2\pi\frac{t}{\beta}$$

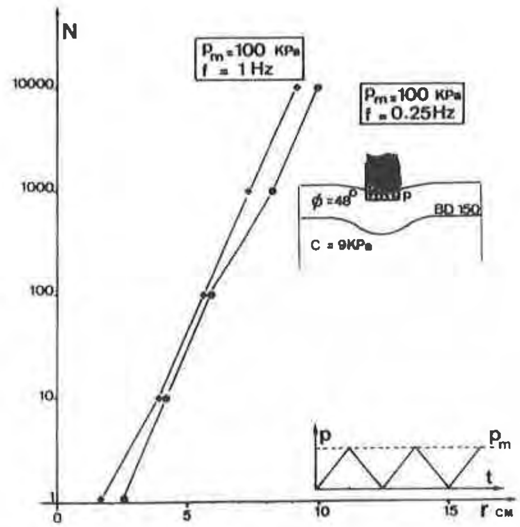


figure 12 : Influence de la valeur de la fréquence

Soit $t = t_j$ le temps correspondant à la fin du $j^{ème}$ chargement et $t = t'_j$ le temps correspondant à la fin du $j^{ème}$ déchargement

$t_j = (j-1)\beta + \beta/2 \qquad t'_j = j.\beta$

Pour une éprouvette de longueur initiale L'_o, on obtient ainsi la longueur L_j au temps t_j et L'_j au temps t'_j :

$$L_j = L'_o\,(1 + \varepsilon_T(t_j)) \qquad (9)$$

$$L'_j = L'_o\,(1 + \varepsilon_T(t'_j)) \qquad (9')$$

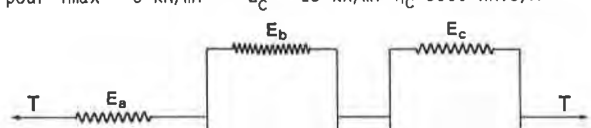
On définit 2 modules Kj et K'j apparents, de chargement et déchargement, que nous utiliserons par la suite.

$$\frac{Tmax}{K_j} = \frac{L_j - L'_{j-1}}{L'_{j-1}} = \frac{\varepsilon_T(t_j) - \varepsilon_T(t'_{j-1})}{1+\varepsilon_T(t'_{j-1})} \qquad (10)$$

$$\frac{Tmax}{K'_j} = \frac{L_j - L'_j}{L'_j} = \frac{\varepsilon_T(t_j) - \varepsilon_T(t'_j)}{1+\varepsilon_T(t'_j)} \qquad (10')$$

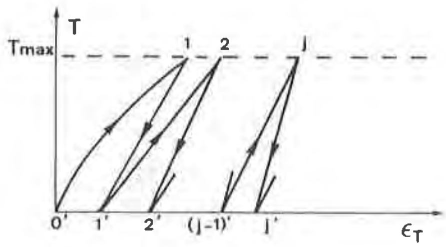
Des essais cycliques effectués à l'Université de Liège sur BD150 ont permis d'obtenir :

E_a = 320 kN/ml $\qquad E_b$ = 22 kN/ml
pour Tmax = 4 kN/ml $\qquad E_c$ = 7 kN/ml $\quad \eta_c$=2400 kN.s/m
pour Tmax = 6 kN/ml $\qquad E_c$ = 18 kN/ml $\quad \eta_c$=8000 kN.s/M



A partir de (8) et (10), (10') nous en avons déduit l'évolution des modules apparents K_j de charge et K'_j de décharge.

Tmax	N=j	1	2	10	100	1000	10 000
4	K_j	20.8	366	368	391	463	469
4	K'_j	391	391.2	393	411	465	469
6	K_j	20.8	402.3	403.6	414	454	460
6	K'_j	411	411	412	421	455	460



IV EFFET MEMBRANE SOUS CHARGEMENT CYCLIQUE :

Nous avons montré au chap. I la validité de l'hypothèse faite sur le mode de fonctionnement du géotextile lors du poinçonnement quasi-statique. Nous étendons ce schéma au cas cyclique : la charge max. est constante (p_m.B) mais quelle est la part de la charge reprise par le géotextile et celle reprise par le sol de fondation cohérent ? :

Nous avons montré expérimentalement (fig. 8) que la déformation obtenue à chaque cycle ne devenait importante que lorsque la charge appliquée dépassait $\alpha\%.p_oB$ (capacité portante statique). Dans le cas du bicouche renforcé (fig. 10) la déflexion de la couche de fondation obtenue à chaque cycle est beaucoup plus faible, ce qui induit que la part de l'effort de poinçonnement reprise par cette couche ne dépasse pas $\alpha\%.q_oB^*$ (mais on néglige le poids de la couche de forme susjacente). α est pris voisin de 100 et $q_oB^* = p_oB$ peut être considéré comme indépendant de r lorsque l'écoulement plastique du sol de fondation est déclenché. Nous considérerons donc que le géotextile est soumis à chaque cycle à une charge maximale constante avec r : $Q_m = (p_m - p_o).B$

La charge minimale est égale au poids de la couche de forme d'épaisseur H, que nous négligeons (Q' = 0).

Le schéma de fonctionnement sera le suivant :

L'_{j-1} longueur en membrane en fin de (j-1)' décharges

L_j longueur en membrane en fin de (j) charges

L'_j longueur en membrane en fin de j' décharges.

La position d'équilibre de la membrane est caractérisée par la valeur de l'ouverture θ :

$$L_j = B^* .(\theta_j/\sin\theta_j) \quad L'_j = B^* .(\theta'_j/\sin\theta'_j) \qquad (11)$$

avec $B^* = L'_o$ et $\theta = 0$

En reprenant la notion de module apparent (10) : fin du chargement (j) :

$$\text{(équilibre)}\ T_j = Q_m/(2.\sin\theta_j) \qquad (12)$$

$$\text{(élasticité)}\ T_j/K_j = ((L_j-u_A^j)-L'_{j-1})/L'_{j-1} \qquad (13)$$

En effet comme pour le quasi-statique, la tension dans la membrane génère un glissement u_A^j d'ancrage ($u_A^j > 0$).

Le déchargement (j') produit un retour élastique (K'_j supérieur à K_j) avec décollement de la membrane (fig. 13). L'ancrage, dont la tension T_A^j s'annule subit lui aussi un retour élastique ($u_A^{j'} <0$).

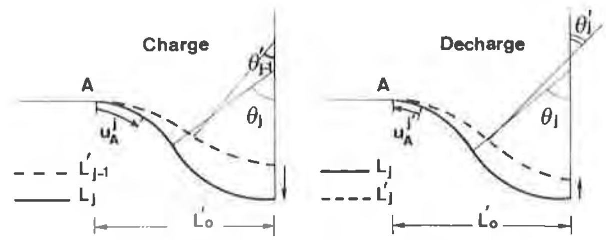
En considérant le déchargement comme un pseudo-chartement inverse :

$$T_j/K'_j = ((L_j + u_A^{j'}) - L'_j)/L'_j \qquad (13')$$

En procédant cycle par cycle à partir de j'= 0, on peut déterminer la déflexion (à partir du calcul de θ_{j+1}) en fonction du nombre de cycles. L'équation générale est de la forme

$$A(\theta_j, u_A^j, K_j).\frac{\theta_{j+1}}{\sin\theta_{j+1}} + B(K_j+1).\frac{1}{\sin\theta_{j+1}} = C(\theta_j, u_A^{j'}, K'_j) \qquad (14)$$

Les glissements u_A sont déterminés à partir de la loi d'ancrage et les modules apparents K et K' à partir d'essais de traction (chap. III).



La tension Tmax des essais de traction cyclique effectués sur BD150 ne correspondant pas aux tensions en service pour nos essais de poinçonnement, et K_j, K'_j étant dépendants de Tmax, il ne nous a pas été possible d'effectuer une application numérique. Cependant le modèle présenté apporte une première contribution à la formulation de la fatigue des chaussées provisoires renforcées.

figure 13 : Retour élastique d'un géotextile en membrane

V CONCLUSION :

Cette étude de chargement cyclique nous a permis de faire apparaître les points importants régissant le fonctionnement d'une chaussée soumise à de grandes déformations : valeur de la charge roulante, vitesse de circulation, rigidité et dimensionnement de la couche de forme, et choix du géotextile.

Fort de ces renseignements, nous avons entrepris, en collaboration avec le Laboratoire Régional des Ponts et Chaussées de Nancy, une étude grandeur nature de piste sur sol mou, qui contribuera à l'orientation de nos nouvelles recherches sur le sujet.

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Small Scale Load Tests on a Soil-Geotextile-Aggregate System

Une eprevus de fardeau de petite magnitude sur une systéme des agrégats de sol géotextile

Small scale laboratory tests were performed to study the effect of a geotextile on a soil-geotextile-aggregate system. The analysis technique lead to qualitative conclusions regarding performance parameters that may be used (with due caution) in developing quantitative data for field applications. The tests demonstrate that a geotextile disc slightly larger than the loading plate has a very significant reinforcing effect on the system; whereas, a disc identical in size to the loading plate does not appear to have any effect. The tests also indicate that the effect of a limited expanse of geotextile diminishes as the thickness of the aggregate increases.

Les expériences de laboratoire, simple, était preformés pour étudier l'effet du geotextile d'une systéme agrégat de sol-geotextile. Les analyses ont produit les conclusions qualitatif au subjet des representations de parameter qu'ils ont utilizés pour obtenir les informations qualitatif. Pour les applications due champ des experiments ont demontrés que la disque geotextile, qui est un peu grandeur que l'assiett chargement, a un effet renforcement de signification au systême. Tandis qu'une disque avec une taille identique que l'assiette chargement n'apparaîte pas avoir un effet. Les expériences indiquent, aussi, que l'effet du expanse limité de geotextile diminue puisque l'épaisse des agrégats ont augmentés.

1. INTRODUCTION

Several theoretical and experimental studies indicate that the inclusion of a geotextile changes the stresses within soil-geotextile-aggregate systems, thus improving their performance. Several researchers have presented theoretical approaches to calculating the supporting effect of the geotextile. These approaches consider only the vertical component of the tensile stresses in the geotextile as it is stretched over the curved surface caused by rutting. Kinney (1979) extended these concepts to include the energy absorption characteristics of the geotextile and the effects of the shear stresses developed on the subgrade by the geotextile. All of these studies indicate that for the beneficial effects to be substantial the tensile stresses in the geotextile must be significant and the profile must have undergone deformation under the load.

Field and laboratory evidence shows that the presence of the geotextile may have a stabilizing effect on the system, even at small deformations before any significant tension could possibly develop in the geotextile. There is also indirect but supportive evidence; for example, geotextiles are being used more frequently in railroad applications where the geotextile strains should remain very low. A number of publications point out that the geotextile changes the stresses within the railroad roadbed; however, the available data is inconclusive.

The mechanics by which the geotextile could change the stress distributions within a system prior to developing signficant tension in the geotextile has not been adequately explained. The stress-strain characteristics of both subgrade and aggregate are stress dependent. Minor changes in horizontal stress distributions therefore cause changes in the stress-strain characteristics, which in turn cause changes in the stress distributions and so on. Hence, fairly minor externally caused changes in the horizontal stress distribution can have a significant effect on the system performance. The stress-strain characteristics of granular materials are particularly susceptible to changes in stress when the minor principal stress is near zero. During loading the tendency is for the bottom of the aggregate to go into tensile strain in the horizontal direction and the stress in that direction to drop to near zero. If the geotextile were to restrain the bottom of the aggregate in the horizontal direction even slightly, it could cause significant changes in the entire system performance.

The purpose of the tests was to study the effects of a geotextile on a soil-geotextile-aggregate system in which no tension was developed in the geotextile outside of the area of direct influence of the load.

A small scale test apparatus consisting of a container, a cyclic dynamic loading system and a measuring system was designed for the study. A matrix of tests were performed. In each test a soft clay was placed in the bottom and up to 50mm of aggregate was placed on top. Some of the tests were performed with a geotextile placed between the two.

2. TEST APPARATUS AND MATERIALS

The small scale load tests consisted of placing clay, a geotextile and an aggregate in a cylindrical mold and loading the surface with a circular plate. The mold was 152mm in diameter and 106mm deep.

Several sizes of porous stones were used for loading plates. They ranged in size between 25mm and 80mm in diameter. The load was applied sinusoidally by a pneumatic piston with a 1.0 second period. The load ranged from a low value of 6.7 Newtons to a high value varying up to 32.7 Newtons. The apparatus is shown on Figure 1.

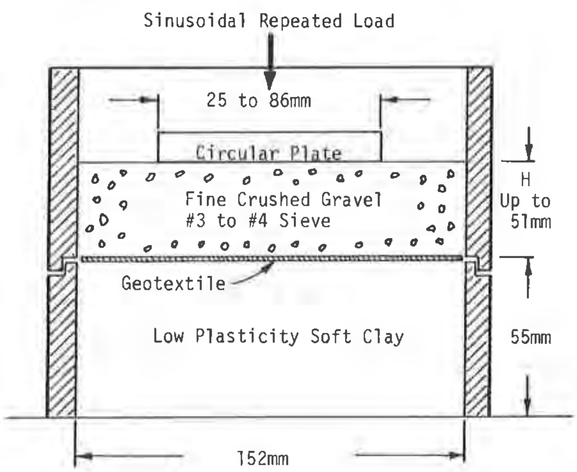

Fig. 1 Test Apparatus

The load was controlled by air pressure and recorded using a load cell. The permanent displacement of the loading piston was measured with a dial gauge.

A naturally occurring silty clay with a liquid limit of 36 and a plasticity index of 14 was used throughout all testing. The clay was mixed at slightly below the liquid limit and smeared into the mold. Water content and vane shear tests were made after every test.

The aggregate was a uniform crushed stone with all material passing the #3 sieve and retained on the #4 sieve. The aggregate was placed in the cell and compacted lightly by hand using a wooden rod. The aggregate ranged in thickness up to 50mm.

A 227gm per square meter heat-bonded, nonwoven polypropolene fabric was used throughout with the heat-bonded side down.

3. TEST RESULTS

Fifteen test series with a total of 61 tests were performed with varying conditions of geotextile placement, loading plate diameter, maximum cyclic load and aggregate thickness. The conclusions drawn are based on the results of all the tests, but only that data required to demonstrate particular points are shown herein.

3.1 Self Stabilizing Effect of Soil-Aggregate-Systems

Without exception the displacement per load cycle decreased with increasing number of load cycles. This is as expected and conforms with other experience to date. Although this response has been frequently observed, it is not well understood. A detailed discussion of the response is beyond the scope of this report; however, mention of its significance to geotextile reinforcement is relevant.

In systems which exhibit the self-stabilizing effect, any factors which retard rut development tend to accelerate the self-stabilizing effect. It should be noted that not all systems demonstrate this effect. Systems with highly sensitive soils or systems which exhibit pumping or liquefaction become softer with increasing number of load cycles.

3.2 General Effect of the Geotextile on the System Performance

The general effect of the geotextile on the test results can be seen by making three basic comparisons. Comparisons between systems with and without a geotextile and no aggregate (Figure 2, tests e and c) show that the geotextile increases the stability of the system slightly even when there is no aggregate present. This is probably due primarily to several factors which are significant to the test but which may not be significant in the field, such as the relative thickness of the geotextile and its ability to carry some bending moment. Both these properties tend to cause the load to be spread out on the clay, reducing displacement. In addition, the geotextile probably adheres to the clay more than the porous stone, thereby reducing lateral spreading of the clay and reducing displacement. These effects must be considered throughout the analysis of the test data.

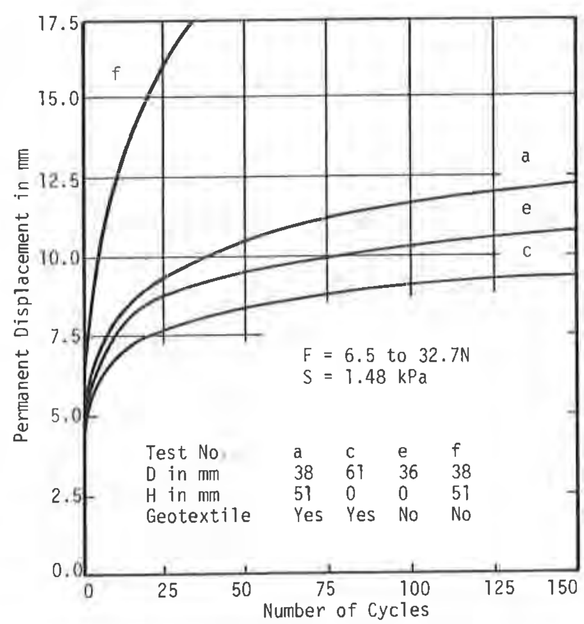

Fig. 2 Test Series No. 10

Comparisons between systems with and without a geotextile and 25mm of aggregate (Figure 2, Tests a and f) show that with 25mm of aggregate the geotextile has a very substantial stabilization effect. The accuracy of the data at low deformations was not adequate to determine at what point the stabilization effect became significant; however, it appears to have been at a displacement of about 5mm. This is a displacement to loading plate diameter ratio of about 0.13 which translates to a rut depth on the order of 25 to 100mm in typical field installations.

Comparisons between systems with and without a geotextile and 50mm of aggregate (Figure 3, Tests c and d) show that when 50mm of aggregate are present the geotextile has very little influence on the system response. Conceptually it seems reasonable that a geotextile will have a minimal effect on the system response if placed under a large thickness of aggregate. There are serious questions about the data from these tests at large thicknesses of aggregate because of the boundary conditions, hence care should be exercised in making this comparison.

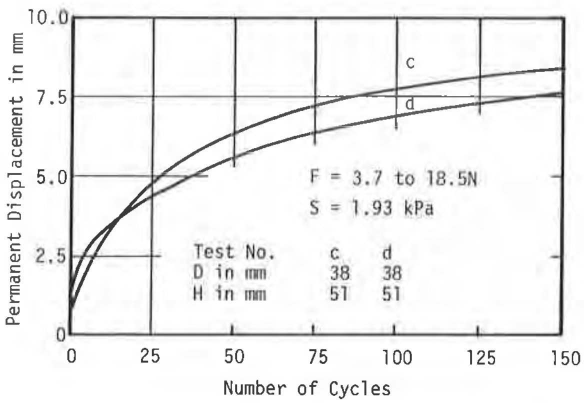

Fig. 3 Test Series No. 12

3.3 Effect of the Lateral Extent of the Geotextile in the Tests

Five tests were performed, as shown on Figure 4, to help evaluate the effect of the lateral extent of the geotextile under the loaded area. All aspects of the tests were held constant except for the lateral extent of the geotextile at the interface.

No geotextile was used in Test e. Tests d and c contained geotextile discs 38mm and 89mm in diameter respectively. Tests b and a both contained a 152mm diameter disc of geotextile, but in Test b the geotextile was cut at a diameter of 89mm around the center. An aggregate thickness of 25mm and a plate diameter of 38mm were used throughout.

Three significant conclusions are apparent from comparing these test results. The first comes from comparing tests without the geotextile and with the 38mm diameter disc of geotextile (Tests c and d, respectively). The two test's results are almost identical, indicating that the relatively small disc of geotextile did not change the system response significantly. This leads to the conclusion that the geotextile must have some minimum extent to be of any benefit. This conclusion is not axiomatic. The center of the profile has the highest normal stresses, the largest radial tensile strains and the most potential for intermixing of aggregate and clay. It would therefore seem reasonable to assume that even a small area of geotextile under the center of the load would improve the performance, but it did not in the tests.

The second conclusion is apparent from comparing tests with 89mm and 152mm discs of geotextile (Tests c and a, respectively). It appears from these tests that the larger disc of geotextile is slightly more beneficial than the smaller. There are several obvious explanations for this effect:

* The load distribution may spread out through the granular material slightly past the extent of the 89mm disc of geotextile.
* There may be significant tensile stresses developed in the geotextile from outside the direct influence of the loading plate, resulting in a drum effect.
* The bending stiffness of the geotextile may effect the small scale test results.

The third conclusion is apparent from comparing the results of Tests c, b and a. The load displacement history for Test b with the cut 152mm diameter disc lies between Test c with the 89mm diameter and Test a with the 152mm diameter disc. It appears from these tests that cutting the geotextile decreases its effectiveness, and the geotextile outside the cut is still somewhat effective. Therefore, the radial tension in the geotextile at this radius in these tests must be important. The origin of the tensile stresses is not apparent. It could be due to direct load induced, outward directed shear stresses from the aggregate, or due to outward directed shear stress on the geotextile from the clay outside the loaded area as the geotextile is stretched over the longer rutted surface.

3.4 Effect of Geotextile on Load Distribution

The first two sections have discussed qualitatively the effects of a geotextile on the test results. This section is devoted to a quantitative analysis of the data and a discussion of how this might be interpreted in terms of field response. The method of analysis used on the test data is unique and would be applicable to many types of future testing.

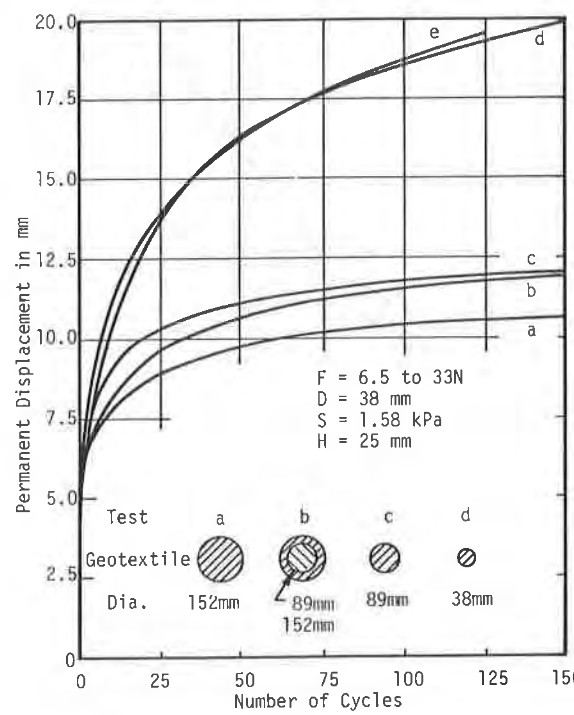

Fig. 4 Test Series No. 3

3.4.1 Method of Analysis

Basically the method of analysis is simple. Loads of various magnitudes were applied to various sizes of loading plates placed directly on the clay. The responses of these tests were normalized, giving the general response of the clay to various loading conditions. The assumption is made that the behavior of the clay is the dominant factor in the response of the system to the load applied on top of the aggregate. The response of the load on the surface of the aggregate was then compared to the generalized response for the load on the clay alone, resulting in an effective loading condition on the clay. The effective loading condition for systems with and without a geotextile were then compared.

A total of 15 tests were performed with various loads on various sizes of loading plates placed directly on the clay. The relationship between the vertical displacement occurring between 10 and 50 cycles and the ratio of the peak stress divided by the clay shear strength is shown on Figure 5. The relationship can be represented fairly accurately with a single line with no apparent skewness caused by peak load or plate size. Therefore, a reasonable estimate of the ratio of the peak stress to clay shear strength can be made if the change in deflection between 10 and 50 cycles is known. It should be noted that edge effects are included in the analysis within the limits of the tests performed.

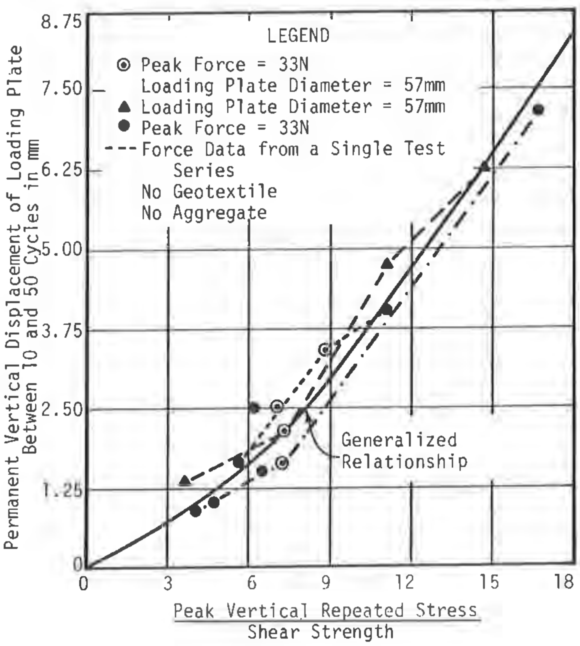

Fig. 5 Normalized Effect of Stress Level and Loading Plate Diameter

Figure 5 leads to the "effective area" concept used throughout the remainder of the analysis. The effective area is defined as the area of a circular plate placed directly on the clay which would result in the same displacement under the same total peak force as the actual load on the actual loading plate placed on the surface of the aggregate. Definitions for "effective stress" and "effective radius" follow similar logic.

In making these definitions, it was necessary to make some basic assumptions about the behavior of the system. The assumptions are made that the stress is transmitted through the aggregate and geotextile in a fashion that makes the stress on the clay appear as though it were being applied by a rigid circular plate. In other words, the stress is applied to the subgrade over a circular area that is depressed uniformly. The stress is in fact spread out over the clay surface, the deformed surface is dish shaped and the maximum displacement of the clay surface is less than that of the loading plate. The assumption is therefore not good, and the absolute value of any answer obtained would be suspect. However, the shape of the stress distribution and deformed surface should be similar between tests with and without a geotextile; hence, the assumption should lead to reasonably accurate comparisons between these two conditions. On larger scale tests, steps could be taken to correct for the inaccuracies in the assumption.

3.4.2 Test Results

Following the concept presented above, the effective parameters were developed for each test. The effective radius versus aggregate depth is shown on Figure 6. Each test in the comparison had a loading plate diameter of 38mm, therefore, the relationships normalized by loading plate radius are identical.

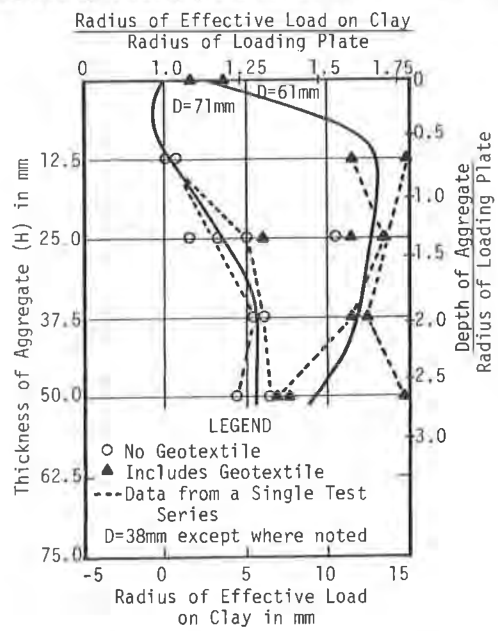

Fig. 6 Radius of Effective Load on Surface of Clay

The effective radius in systems with a geotextile was significantly greater than the effective radius in equivalent systems without a geotextile for most aggregate thicknesses used. This means that the presence of the geotextile causes the load to be distributed more widely than with the aggregate alone. Previously, it was noted that the systems with a geotextile were only slightly influenced by the drum effect; hence, it appears reasonable that the geotextile either acts as a tension member allowing the aggregate-geotextile combination to carry tension and bending, and/or the geotextile creates a confining pressure in the aggregate, changing its load distribution properties.

The effect of the geotextile appears to become less significant as the thickness of the aggregate increases. As described earlier, this would also be expected in field situations and was noted in larger scale laboratory tests by Kinney (1979). The results of this test may, however, overestimate the effect. The test results show some stabilization effect with no aggregate. This is probably due to the flexural stiffness and thickness of the geotextile in the small scale tests. This effect could be significant at all depths in the test but should be insignificant at all depths in the field. In addition, the test is small and as the thickness of the aggregate increases the size of the effective area increases and the boundary conditions become more significant. The boundary conditions are included in the normalizing procedures; however, the actual stress distribution probably extends outside the radius of the largest loading plate used. Therefore, the effects of the boundary conditions are probably underestimated for tests with a large effective radius.

It is interesting to note that in tests without a geotextile the effective radius is the same for no aggregate and for 12mm of aggregate. This is probably caused by a combination of two factors. The effective area concept may underestimate the influence of the aggregate, and/or 12mm of aggregate may not effectively distribute the stress from the loading plate outside the loaded area. The effective area may be underestimated for several reasons. At small aggregate thicknesses the deformed shape of the interface is more pronounced, which causes lateral spreading and a reduction in aggregate thickness. Since the aggregate thickness is decreased, the displacement of the clay is less than that measured, which results in an underestimate of the effective area. It is also possible that the effect of the lateral spreading allowed by the aggregate, which was restrained by the loading plate placed directly on the clay, is enough to negate any effect of the load distribution through the aggregate. In either case the response of the system to load was not improved by the addition of 12mm of aggregate and a proportional response should be anticipated in the field.

4. CONCLUSIONS

The test results and analyses presented herein provide valuable insight into the behavior of geotextile reinforced unsurfaced roads. The following conclusions were reached regarding the test results:

* Geotextiles act as structural reinforcement to the aggregate, causing it or the combination to distribute the load over the subgrade. This is true even if the geotextile does not extend outside the area of direct influence of the applied load.
* The effect of the geotextile diminishes with greater thicknesses of aggregate.
* Significant deformation appears to be required for the geotextile to act as a reinforcement, and additional deformation accentuates the beneficial effects.

These concepts can be extended to the field conditions if caution is used. The numerical data cannot be extrapolated directly to the field because of the differences in scale, the boundary conditions in the test, and the inaccuracies in the analysis techniques.

The testing technique and method of analysis appear to be valuable research tools and the work should be continued, but on a large scale. Full scale tests are desirable but expensive and time consuming. Tests performed in a tank about 1.22m square with .6m of clay, up to .3m of a pea gravel sized crushed stone aggregate, and loading plates on the order of 76 to 33mm in diameter would provide very valuable information. This scale is large enough to be meaningful for extrapolation and yet manageable from a cost and time of testing standpoint. The square configuration is suggested to allow studying the effect of the three dimensional wheel load placed in a two dimensional rut.

ACKNOWLEDGEMENTS

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Bearing Capacity of a Sand-Soft Subgrade System with Geotextile

Capacité portante d'un bicouche, sable sur sol mou, renforcé par géotextile

When considering a two layer system (a cohesionless soil subbase of thickness H and a clay subgrade) reinforced by geotextile, we obtained an increase of bearing capacity of the system under punching.

This subject is investigated here and can be applied to temporary roads on soft subgrades.

Numerous data obtained on models (quasi-static punching) enable us to perform an analysis of the influence of geotextile modulus, taking into account the setting conditions (free or fixed extremities of the fabric) and to outline the importance of anchorage design. Also other positions for the geotextile, particularly those close to the middle of the subbase, are considered.

The experiments suggest an interpretation of the fabric behaviour : membrane effect, under the axis of the load and lateral sliding of the anchorage with compatibility conditions of stresses and strains for the two zones

Finally, the conditions of similarity for the model can be analysed.

L'étude présentée traite du gain de portance obtenu, lors du poinçonnement d'un sol bicouche (couche de forme pulvérulente d'épaisseur H sur couche de fondation argileuse) par la pose d'un géotextile à l'interface des 2 couches. Cette étude pourra s'appliquer en particulier aux chaussées provisoires sur sol mou.

Un important programme d'essais sur modèles réduits (poinçonnement quasi-statique) nous permet d'analyser le rôle du module du géotextile, suivant les conditions de mise en oeuvre (extrémités du géotextile fixes ou libres) et de mettre en valeur l'importance du dimensionnement de l'ancrage du géotextile. Enfin nous envisageons d'autres positionnements du géotextile, en particulier à mi-épaisseur de la couche de forme.

Ceci nous a permis de proposer un mécanisme traduisant le comportement du géotextile : fonctionnement en membrane, dans l'axe de la charge et ancrage glissant latéralement, les contraintes et déformations dans les deux zones devant être compatibles.

Ceci nous permet d'analyser les principales conditions pour que le modèle réduit respecte la similitude.

I. OBJECTIFS

Cette étude de poinçonnement sur sol bicouche (sol pulvérulent sur sol cohérent) renforcé par un géotextile placé à l'interface avait plusieurs objectifs :

Le premier était de traiter le problème des chaussées provisoires sur sol mou, l'utilisation d'un géotextile s'étant montrée dans ce cas très intéressante. Notre modèle réduit est censé représenter une chaussée provisoire à l'échelle 1/3. La plaque simule la double roue d'un camion, et l'orniérage toléré étant dans ce cas important, l'essai de poinçonnement, effectué à vitesse d'enfoncement constante (0,04 mm/s) est poursuivi jusqu'à un enfoncement égal à la largeur de la plaque (r = B). La couche de forme est une couche de sable ou de gravette d'épaisseur H et la couche molle de fondation est une couche d'argile d'épaisseur D = 0,50 m (fig. 1). Ces essais de poinçonnement quasi-statique ont été complétés (1) par des essais de chargement répété simulant le passage de véhicules.

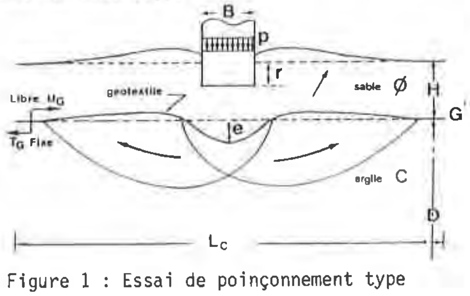


Figure 1 : Essai de poinçonnement type

Le second objectif, plus général, était d'étudier le mode de comportement d'une inclusion textile dans un massif de sol à la rupture, afin d'optimiser le choix du géotextile et son positionnement dans l'ouvrage en terre. A ce titre les renseignements obtenus ici ont été utilisés pour un programme de calcul de stabilité de remblais sur sol mou renforcés à leur base.

II. MODELE EXPERIMENTAL : (2)

La cuve d'essai (fig. 1) est parallèlipépédique (largeur L_C = 2.3 m, hauteur h_C = 0.75 m, épaisseur l_C = 0.30 m). La paroi frontale avant est en altuglass renforcé, ce qui permet de photographier la structure déformée du bicouche en cours d'expérience. Cette paroi se translate latéralement pour permettre le montage et le démontage du massif d'argile (fig. 2).

La plaque est rectangulaire, B = 0.15 m de largeur égale à l'épaisseur 0.3 m de la cuve, afin d'obtenir un poinçonnement en déformation plane. Pour éliminer les effets de bords (frottement du sol en écoulement le long des parois frontales), les parois sont lubrifiées et la plaque est constituée d'une plaque centrale (0.15 m x 0.15 m) comprise entre deux plaques de garde (0.075 m x 0.15 m) et l'effort de poinçonnement est mesuré indépendamment sur la plaque centrale et les plaques latérales.

L'argile, moyennement plastique (w_P = 20 %, w_L = 53 %) est utilisée à une cohésion non drainée C_u = 9 ou 27 kPa. Le sol pulvérulent à granulométrie étroite est un sable inférieur à 1 mm (ϕ = 34°) ou une gravette concassée, entre 5 et 10 mm, (ϕ = 48°).

Les géotextile sont soit des tissés de laminettes (t205 et t110), soit des non tissés aiguilletés polyester Bidim classiques (BD150, BD210, BD550) ou renforcé par

Figure 2 : Modèle expérimental

des multifilaments (BDrf), soit un non tissé thermolié polyester-polypropylene Terram (TM230) (notation : TM230, Terram de μ = 230 g/m²).

Le géotextile placé à l'interface sol pulvérulent-sol cohérent fait toute la longueur de la cuve (GG' = 2.20 m). Il est, soit libre à ses extrémités G et G' (et on mesure le déplacement horizontal u_G), soit fixé (et on mesure l'effort T_G). Plus quelques cas particuliers. nappe courte (GG' = 1 m), bi-nappe (l'une des nappes est placée à mi-hauteur (H/2) de la couche de forme), ou conteneur (bi-nappe dont les deux nappes cousues entre elles aux extrémités confinent le sol à l'intérieur) (fig. 3).

Pendant l'essai de poinçonnement, on mesure l'enfoncement r de la plaque et l'effort de poinçonnement (p.B) par ml, p étant la pression uniforme équivalente, on photographie le massif déformé, ce qui permet une mesure de e, la deflexion et B* la largeur fictive à l'interface sable-argile.

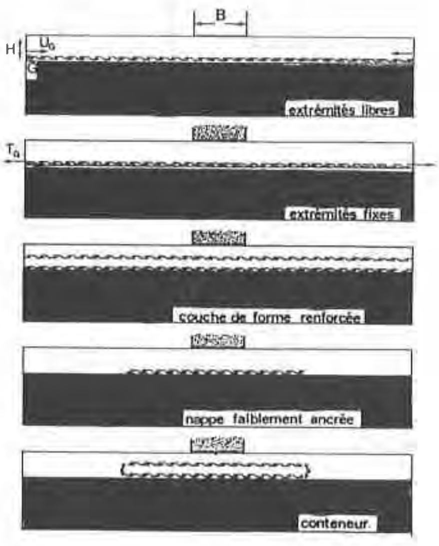


Figure 3 : Différents positionnements du géotextile

III. PARAMETRES MECANIQUES D'UN GEOTEXTILE

1. Traction en déformation plane

Dans la plupart des applications, le géotextile est sollicité en traction mais les déformations latérales sont empêchées.

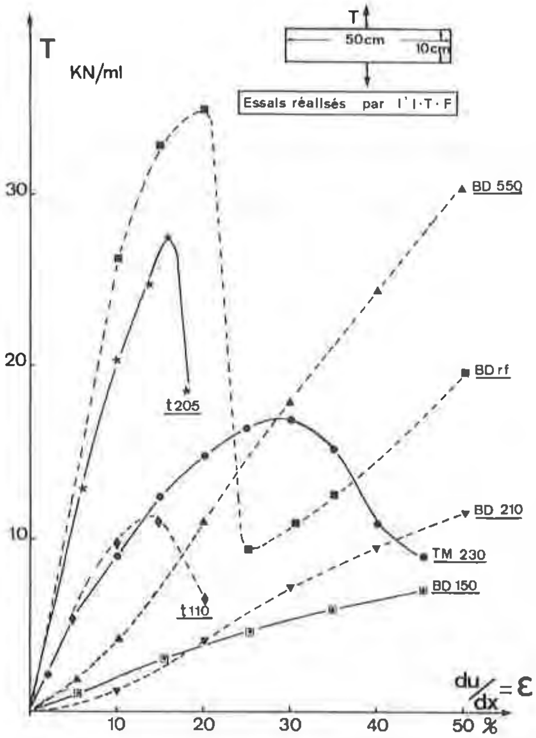


Figure 4 : Essais de traction sur géotextile

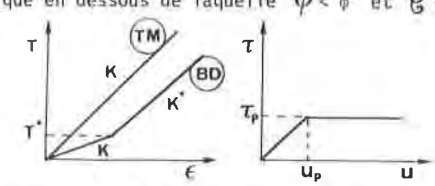
L'essai sur éprouvette (0.50 m sur 0.10 m de hauteur) correspond assez bien à ce critère. Il nous a permis de définir un module moyen de déformation plane : T = K.du/dx pour chaque géotextile (fig. 4). Pour certains géotextiles, il peut être nécessaire de définir deux modules, un module initial K pour T < T* et un module K* pour T > T* :

$$T = K^* \, du/dx + (1 - K^*/K)T^* \qquad (1)$$

2. Interaction sol-géotextile :

Nous disposons d'une grande boite de cisaillement (section 0.40 m x 0.25 m) permettant de mesurer l'angle de frottement φ pour un sol pulvérulent (ϕ) au contact ou l'adhérence $\mathcal{C}$ pour une argile (C_u).(3).

Nous présentons ici quelques résultats particuliers obtenus pour des contraintes normales σ_N très faibles (fig. 5), correspondant aux conditions propres aux modèles réduits : alors que les géotextiles présentent généralement une bonne capacité de liaison avec les sols ($\varphi = \phi$ et $\mathcal{C} = C_u$), lorsque σ_N devient trop faible pour appliquer correctement le géotextile sur le sol au contact, φ et $\mathcal{C}$ mobilisés se trouvent minorés : pour chaque système sol-géotextile, il existera une contrainte σ_N critique en dessous de laquelle $\varphi < \phi$ et $\mathcal{C} < C_u$.



3. Application : comportement en ancrage

Nous présentons (fig. 6) les essais d'ancrage effectués sur des nappes BD550 et TM230 de longueur initiale GA = L_a = 1.54 m. Soit T_A l'effort en tête et u_A le déplacement correspondant.

La nappe est enterrée dans un massif de sable ($H = 0.31$ m $\longrightarrow \sigma_N = 4.8$ kPa). A partir des lois de traction et de frottement définies ci-dessus, nous proposons une formulation théorique du comportement en ancrage :

TM230 est supposé posséder un module de traction K unique quel que soit T tandis que pour BD 550, on considèrera deux modules, le réarrangement initial des fibres de l'aiguilleté obligeant à considérer un module initial faible :

TM230 $K = 80$ kN/ml
(fig. 4) BD550 $K = 35$ kN/ml pour $T < T^* = 3,5$ kN/ml
et $K = 75$ kN/ml pour $T > T$

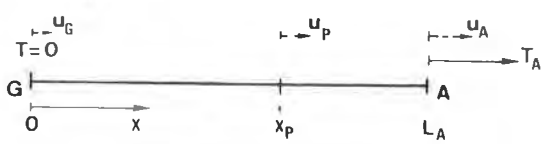
Le comportement en frottement sur sable est supposé elasto-plastique : $\tau = \alpha . u$ pour $u < u_p$ et $\tau = \tau_p = \sigma_N . tg\varphi$ pour $u > u_p$, u étant le déplacement relatif sol-geotextile.

TM230 $\varphi = 36°$ $u_p = 15.5$ mm
BD550 $\varphi = 35°$ $u_p = 31$ mm

En appliquant ces lois de comportement d'un bout à l'autre de l'ancrage géotextile, on détermine deux zones ($x < x_p$, $\tau < \tau_p$ et $x > x_p$, $\tau = \tau_p$). Dans le cas du module K unique, on écrit : $dT = -\tau dx$ et $T = K.du/dx$ $\forall x$ et on obtient la tension T_A et le déplacement u_A en tête d'ancrage, pour un même x_p : (figure 6)

$$T_A = 2\alpha K.u_p \frac{e^{\sqrt{\frac{2\alpha}{K}}x_p} - e^{-\sqrt{\frac{2\alpha}{K}}x_p}}{e^{\sqrt{\frac{2\alpha}{K}}x_p} + e^{-\sqrt{\frac{2\alpha}{K}}x_p}} + 2\tau_p(L_a - x_p) \quad (2)$$

$$u_A = u_p - \frac{\tau_p . L_a^2}{K} + \frac{\tau_p . x_p}{K}(2L_a - x_p) + \frac{T_A}{K}(L_a - x_p) \quad (3)$$



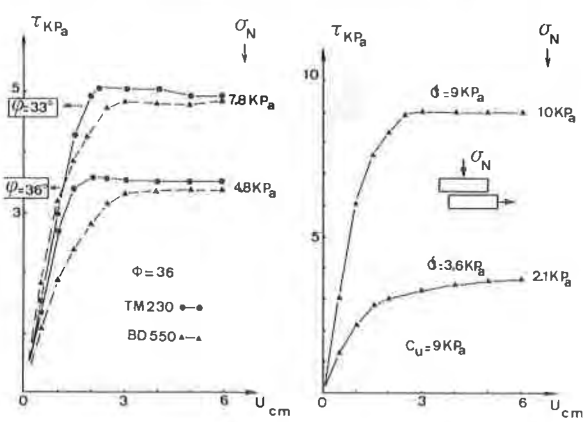


Figure 5 : Essais de frottement et d'adhérence sous faible contrainte normale σ_N

IV. MECANISME DE RENFORCEMENT PAR GEOTEXTILE ;

Les essais de poinçonnement (60 au total) ont été systématiquement effectués, pour les mêmes sols du bicouche, avec géotextiles ($p_G = f(r)$) et sans ($p_O = g(r)$ afin d'obtenir le gain de portance $\Delta p = p_G - p_O = h(r)$. Ce gain s'est toujours révélé positif. L'action du géotextile présente en fait plusieurs aspects :

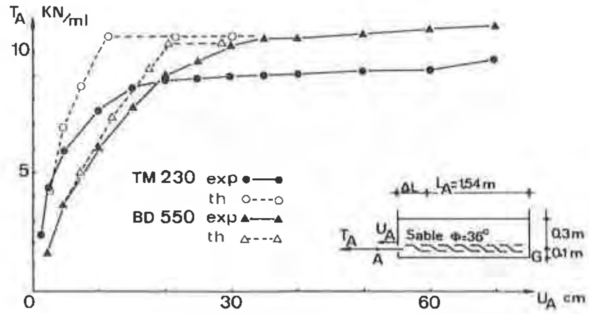


Figure 6 : Essais d'ancrage sur géotextile enterré dans un massif de sable

1. Effet anticontaminant :
Nous avons dans nos essais neutralisé cet effet en plaçant un film plastique sans résistance entre couches de forme et de fondation, dans le cas sans géotextile.

2. Effet membrane :
Le géotextile placé à l'interface s'oppose au poinçonnement de la couche d'argile, en se mettant en traction dans la zone sous la plaque (entre A et A' fig. 7). Ce schéma supposé pour la déformée de l'interface, à double arc de cercle, est en accord avec les relevés (fig. 8) Les effets "membrane" latéraux dus aux remontées d'argile latéralement à la plaque, pris en compte dans une théorie précédente (5), se sont révélés négligeables. Sur la "largeur fictive" B^*, le géotextile résiste à une pression uniforme Δq, en prenant la forme d'un arc d'ouverture 2θ. L'équilibre vertical du géotextile impose la mobilisation en appui d'une pression Δq suivant les arcs CA et C'A', de même rayon et ouverture.

gain de portance $\Delta p_M . B = \Delta q . B^*$ (4)

La distance horizontale B^* des points d'inflexion C et C' est pratiquement constante en cours d'enfoncement e (fig. 8b).

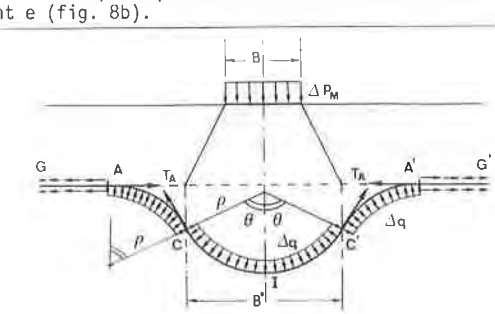


Figure 7 : Effet membrane du géotextile

$e = B^*.(1-\cos\theta)/\sin\theta$ (5)

rayon $\rho = B^*/(2\sin\theta)$ (6)

La tension dans la membrane AA' est constante : $T_A = T_C = T_I = T$

L'équilibre vertical impose : $\Delta p_M . B = 2T_A . \sin\theta$ (7)

Pour une membrane élastique linéaire :
$T = K.(du/dx)$
* membrane fixée en A et A' :
La tension dans la membrane étant uniforme, la déformation l'est aussi :

$\varepsilon = du/dx = (\theta - \sin\theta)/\sin\theta$ (8)

d'où $T = T_A = K.(\theta - \sin\theta)/\sin\theta$ (9)

$\Delta p_M . B = 2K.(\theta - \sin\theta)$ (10)

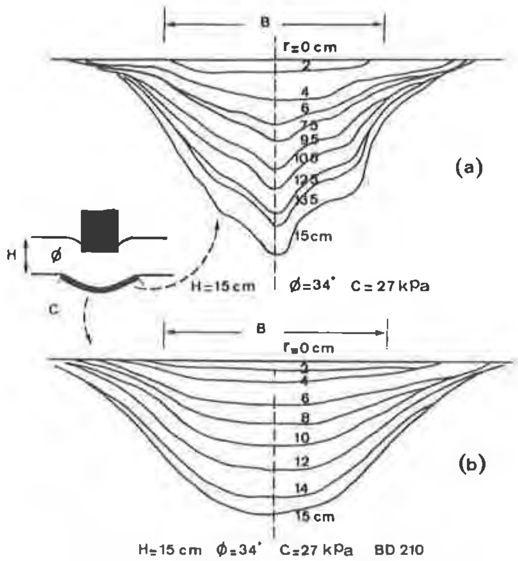


Figure 8 : Déformée de l'interface sable-argile en présence (b) ou non (a) de géotextile

La détermination expérimentale de B^* et e permet (5) d'évaluer θ, ainsi que T et Δp_M (9 et 10) proportionnels au module K du géotextile (abaque ci-contre).

Cependant ce calcul surestime l'effet "membrane" car A et A'ne sont pas fixes : T_A est repris par la longueur AG de géotextile servant d'ancrage à la membrane.

* membrane ancrée en A et A' :

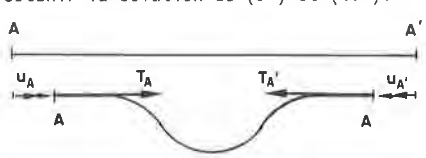
La reprise par GA d'une tension T_A nécessite un déplacement u_A (§ III.3) qui minore ε, T et Δp_M.

$$\varepsilon = (\theta - \sin\theta - u_A.\sin\theta/B^*)/\sin\theta \qquad (8')$$

$$T = T_A = K\,(\theta - \sin\theta - u_A.\sin\theta/B^*)/\sin\theta \qquad (9')$$

$$\Delta p_M.B = 2K\,(\theta - \sin\theta - u_A.\sin\theta/B^*) \qquad (10')$$

La compatibilité entre les relations (2) + (3) et (9') liant T_A et u_A soit en ancrage, soit en membrane, permet d'obtenir la solution de (9') et (10').



3. Effet repartiteur :

Le poinçonnement de la couche d'argile se produit suivant une largeur $B^* \neq B$ généralement. Cette largeur est définie comme la distance entre les 2 points d'inflexion principaux de l'interface du bicouche. Cependant sans géotextile $B_0^* = B$ avec le sable comme couche de forme (fig. 8a) et B_0^* légèrement supérieur à B pour la gravette, ces couches étant faiblement rigides.

Le géotextile augmente systématiquement B^* (fig. 8 et 9), le volume d'argile en écoulement plastique est aussi plus important. D'où un gain de portance :

$$\Delta p_R.B = q_0(B^* - B_0^*) \qquad (11)$$

si la pression portante q_0 de l'argile est supposée non modifiée par la présence du géotextile.

Cette augmentation $(B^* - B_0^*)$ peut se justifier partiellement par la compression horizontale de la couche pulvérulente due à l'ancrage géotextile (suivant AG et A'G').

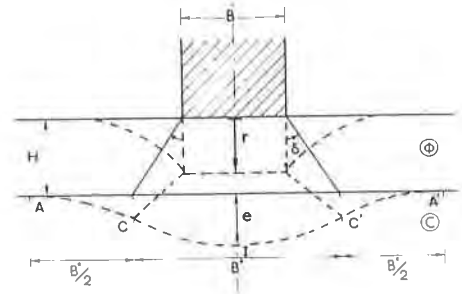
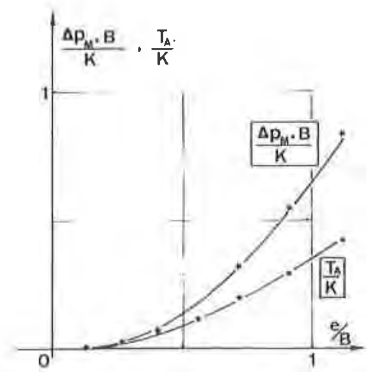


Figure 9 : Largeur fictive en présence de géotextile



V. BILAN DES ESSAIS EXPERIMENTAUX :

Le gain de portance ($\Delta p.B = (p_G - p_0).B$) expérimental est toujours positif pour r/B > 0,15 et il croit avec r

Figure 10 : Pour mêmes couches de forme et géotextile, le gain relatif de portance $\Delta p/p_0$ croit lorsque la qualité du sol de fondation décroit.

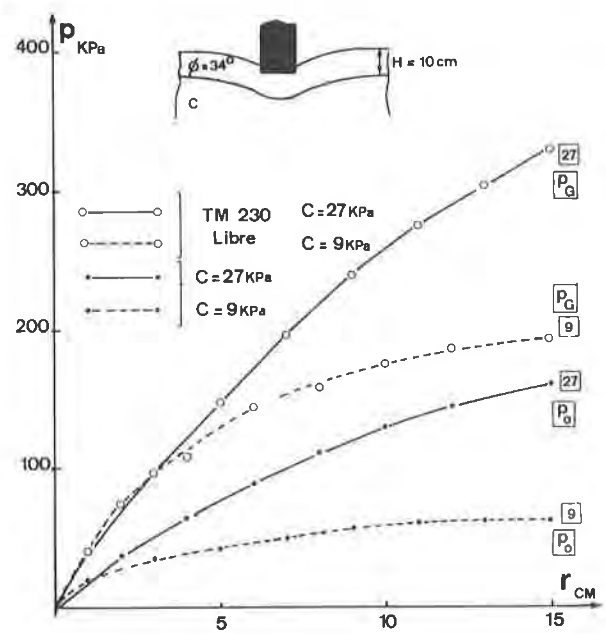


Fig. 10 : Influence de la qualité de la couche de fondation

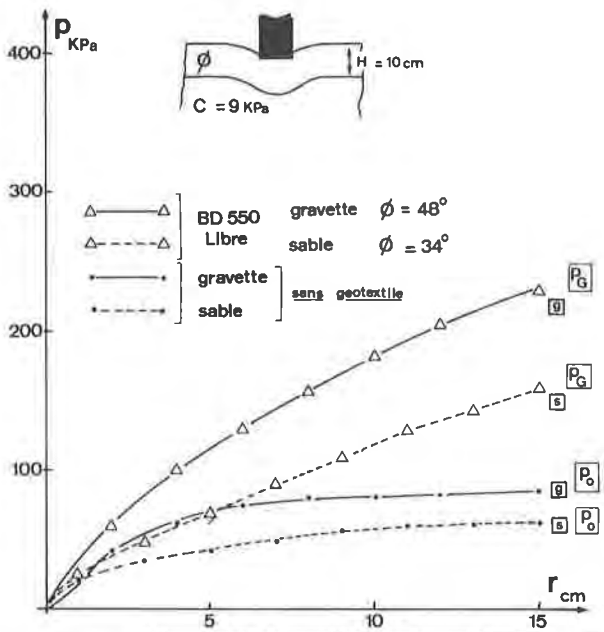


Figure 11 : Influence de la qualité de la couche de forme

Figure 11 : Pour mêmes couche de fondation et géotextile, $\Delta p/p_o$ apparaît plus important lorsque la qualité de la couche de forme s'améliore.
Figure 12 : Le mode de fixation du géotextile influence le gain de portance : $\Delta p/p_o$ augmente lorsque l'on fixe les extrémités G et G' du géotextile, à condition que celui présente un glissement u_G notable, dans le cas ou G et G' sont libres. Dans ce cas, le fait de fixer G et G' mobilise une tension en bout T_G.

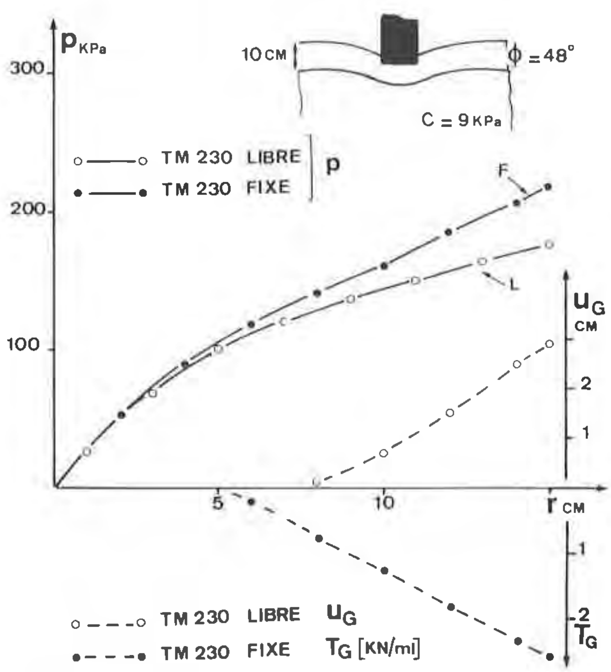


Figure 12 : Influence du mode de fixation du géotextile

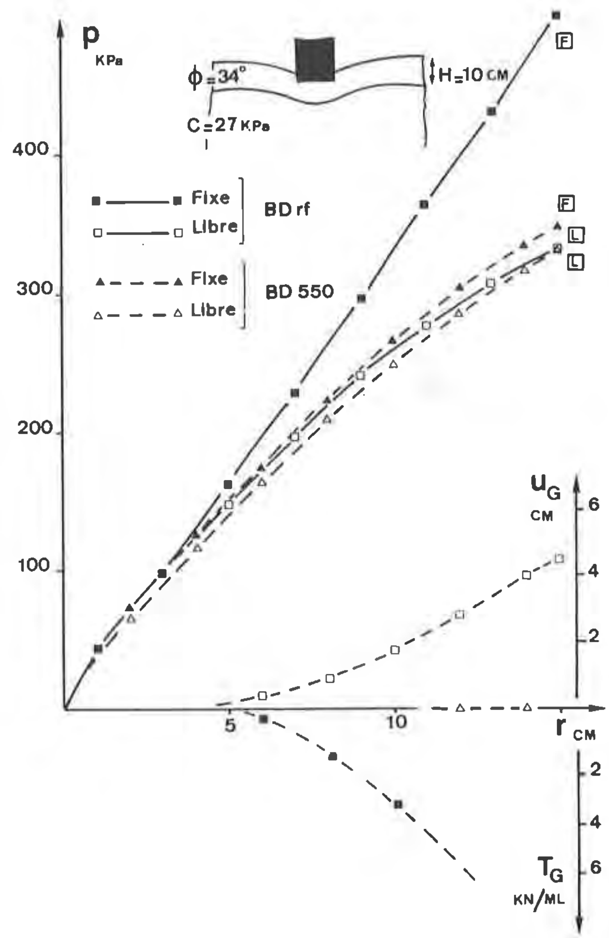


Figure 13 : Influence du module et mode de fixation du géotextile

Figure 13 : Pour une nappe fixée à ses extrémités, Δp croît avec le module. Pour une nappe simplement ancrée dans le sol adjacent, il semble bien exister un module optimal K à longueur d'ancrage donnée au dessus duquel Δ_p se stabilise : BD rf de module supérieur au BD550 tend à glisser en G et G', et le gain de portance est équivalent à celui du BD550 dont les extrémités ne glissent pratiquement pas. Notons que BD550 et BDrf ont même φ
Figure 14 : Une longueur d'ancrage insuffisante amène une diminution du gain de portance.

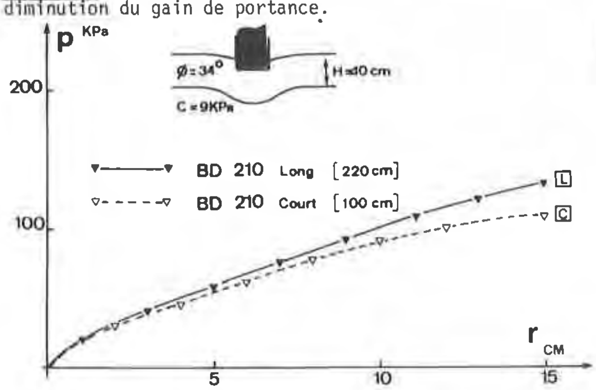


Fig. 14 : Influence de la longueur d'ancrage du géotextile

Figure 15 : On peut envisager de placer le géotextile autrement qu'à l'interface avec la couche de fondation (fig. 3). L'utilisation d'une nappe à mi-épaisseur de la couche de forme de faible rigidité (sable) est très profitable : un double BD210 apporte un gain de portance supérieur à un BD550 unique de même texture et de masse de fibre supérieure (550 g/m²). Un tissé de laminette t110 de module supérieur au BD210 est plus performant car il rigidifie davantage la couche de forme.

Enfin la technique du conteneur apparaît très intéressante. Elle pourrait permettre d'utiliser pour la couche de forme des sols de récupération à priori impropres à cet usage.

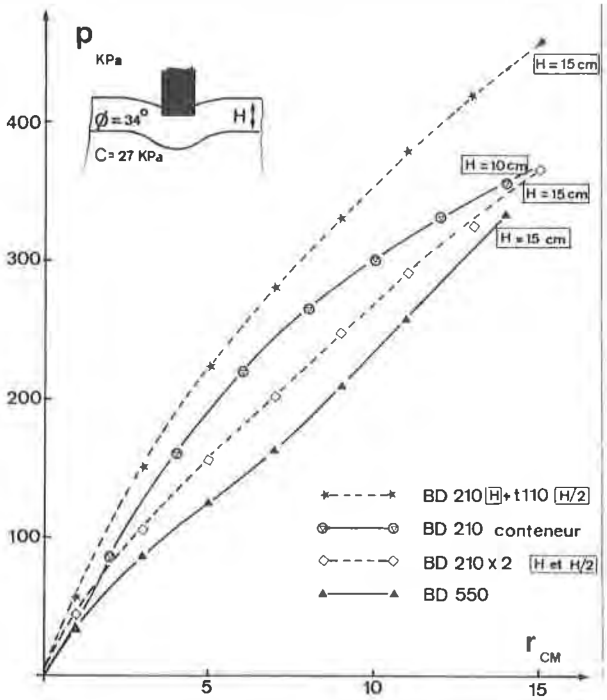


Figure 15 : Différentes mises en oeuvre du géotextile

VI. SIMILITUDE :

L'ensemble des essais présentés a permis d'analyser le fonctionnement d'un géotextile en situation et d'élaborer une méthode de calcul. Cependant les gains de portance Δ_p/p_o obtenus sur modèle réduit seront systématiquement supérieurs à ceux obtenus en réalité, si le géotextile utilisé est le même dans les 2 cas : les principes de fonctionnement mécanique établis plus haut vont nous permettre de le démontrer :

Soit $L/L_r = 1/L^*$ l'échelle du modèle réduit, avec L_r une longueur pour le cas réel et L la longueur correspondante pour le modèle réduit.

Le modèle réduit devra présenter les mêmes déformations :

$$\varepsilon^* = (\Delta L)^*/L^* = 1 \qquad (12)$$

1. Sols en place, on prendra couramment $\gamma^* = 1$
Ceci impose pour les contraintes gravitaires et par conséquent pour l'ensemble des contraintes : $\sigma^* = \gamma^*.L^* = L^*$ (13)
Comme il s'agit d'un problème en grandes déformations, on s'imposera de respecter la similitude pour les contraintes à la rupture du massif de sol :

matériau pulvérulent : $\tau_{max} = \sigma_N . tg\phi$
avec $\tau^*_{max} = \sigma^*_N = \sigma^* = L^*$
$\rightarrow \boxed{\phi^* = 1}$ (14)

On utilisera le même matériau pulvérulent dans les 2 cas (quoique ϕ varie légèrement avec σ_N)

matériau cohérent : $\tau_{max} = C_u$

d'où $\boxed{C^*_u = L^*}$ (15)

Les cohésions devront être dans le rapport des longueurs : pour une cohésion $C_{ur} = 27$ kPa ($I_{CBR} = 1$), on prendra $C_u = 9$ kPa pour le modèle réduit à l'échelle 1/3.

2. Géotextile, les lois de comportement enoncées au § III devront respecter la similitude

Comportement en traction : $T^* = K^*. \varepsilon^*$
avec $\varepsilon^* = 1$ et $T^* = \sigma^*.L^* = L^{*2}$

d'où $K^* = (L^*)^2 = K_r/K$ (16)

Les modules des géotextiles devront donc être dans le rapport du carré des échelles. Pour un modèle réduit à l'échelle 1/3, considérer le même géotextile que dans la réalité revient à considérer un géotextile 9 fois trop rigide.

Loi d'interaction sol-géotextile :

La similitude impose $\varphi^* = 1$, $\mathscr{C}^*/C^*_u = 1$ (17)

et $u_p^* = L^*$ (18)

En fait nous avons vu (§ III.2) que pour des contraintes normales inférieures à la contrainte critique, le contact sol-textile est imparfait, ce qui minore φ et : pour de petits modèles, on veillera à bien appliquer le géotextile sur le sol, afin de vérifier (17).

La condition (18) (déplacements au palier à l'échelle) apparaît difficile à vérifier au vu de la fig. 5 et des résultats de (4).

3. Bicouche renforcé : Pour le bicouche sans géotextile, on vérifiera la similitude, une fois vérifiées (14) et (15) $\phi^* = 1$ $C^*_u = L^*$

$p_{or}/p_o = L^*$ $(p_c.B)_r/(p_o.B) = (L^*)^2$ (18)

Nous négligerons le problème lié aux déformations de ce bicouche.

Le géotextile ancré doit vérifier les relations (2) et (3) : ceci impose $\boxed{\varphi^* = 1\ \mathscr{C}^*/C^*_u = 1\quad u^*_p = L^*,\ K^* = L^{*2}}$

Si l'une de ces conditions n'est pas vérifiée, la relation $T_A = f(u_A)$ n'est plus vérifiée en similitude.

De même, pour le géotextile en membrane, la vérification de l'ensemble des conditions ci-dessus permet de vérifier les conditions (8'), (9') et (10') :

$$\varepsilon^* = 1 \quad T^* = (L^*)^2 = Tr/T \qquad (19)$$

$$(\Delta p_M.B)^* = (\Delta p_M.B)_r/(\Delta p_M.B) = (L^*)^2 \qquad (20)$$

Soit (18) et (20) : $(\Delta p_M/p_o)^* = 1$ (21)

Dans le cas où le géotextile utilisé est le même pour le modèle réduit, aucune relation simple ne permet le passage du modèle réduit au réel, on peut seulement affirmer que la modélisation amplifie le gain de portance.

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Geotextiles in Unpaved Roads: Design Considerations

L'usage des géotextiles dans les pistes de chantier

Plate load tests and truck loading tests were carried out on a full scale model of an unpaved road on very soft subgrade and incorporating geotextiles. The analysis of deflections and fabric strains and the plane strain behavior of geotextiles in a specially developed laboratory tensile tester resulted in a design method for such unpaved roads.

The contribution of the geotextile as reinforcement can play a major role when the fabric is allowed to deform and to be tensioned in situ during construction. Fabrics with high tensile strength and intermediate modulus will usually give the best combination of fabric properties for reinforcement of such structures.

The criteria in selecting an effective geotextile, typical cost comparisons and construction choices are discussed.

On décrit des essais de plaque et de circulation par un camion sur un modèle en vraie grandeur d'une piste de chantier en matériau granulaire sur sol support peu portance en utilisant des géotextiles. On a analysé les déformations de l'interface sol/textile et mesuré les allongements permanents du géotextile. On a étudié le comportement mécanique de textile dans un appareil de laboratoire construit spécialement pour simuler les constraintes du géotextile dans de telles structures.

Le géotextile augmente la portance de l'ensemble s'il peut se déformer et peut développer des tensions sensibles in situ pendant l'exécution des travaux. Des textiles avec une résistance assez élevée et de module intermediaire unissent normalement les caracteristiques mécaniques les plus efficaces pour l'armature des pistes de chantier.

On discute encour les critères pour choisir le géotextile le plus utile, les comparaisons de frais de diverses constructions, et les considérations pour la mise en oeuvre.

1 BACKGROUND

Unpaved roads include: access roads to timber areas, mines or drilling sites, temporary roads to construction sites and similar. Often such roads cross regions where the soils have very low shear strength.

Traditionally, construction of unpaved roads has consisted of placing the most readily available base material over the underlying subgrade. Traffic often caused intermixing of the base and subgrade, necessitating regular regrading and placement of additional base material. More recently, geotextiles placed between the base and subgrade have been utilized to reduce the thickness of the base material required for initial construction and to lower the maintenance required during the life of the road.

Methods for design and construction of unpaved roads with base materials placed directly on a subgrade have been presented by Alvin and Hammit (1), National Crushed Stone Institute, USFS; and others. Utilization of geotextile fabrics in low volume roads has been discussed (2)(3)(4)(5)(6) but the reported results varied and so did the views as to the fabric contribution to the unpaved road structures (6a). The bearing capacity in a model test increased when using a fabric (2)and so did the road performance although it did not depend on the mass per unit area of the fabrics used (6b). Another study found that fabric did not reduce failure load in model tests but did decrease deflections (6c). Similar reductions in deflections were also noted with prestressed fabrics (6d). The importance of allowing larger deflections for better fabric performance was noted by Jessberger (6e) and this is also implied in the design considerations by Giroud(7) and Koerner (8). The present contribution uses a design procedure making use of specific mechanical properties of geotextiles and is based on extensive studies of full scale model roads.

2 FABRIC FUNCTIONS

The four main functions, i.e. separation, reinforcement, filtration and inplane water transport can all contribute to the improved performance of unpaved roads.

Separation: The fabric by providing a mechanical barrier between the base and the subgrade, prevents base loss into the subgrade and intrusions of fines into the base layer.

Reinforcement: The geotextile contributes to the load-carrying capacity due to the mechanical characteristics of the fabric. The tension developed in the fabric results in forces that increase the load-bearing capacity of the system. The mechanical properties of the geotextile are the major consideration.

Filtration: By retaining fine particles while allowing water to pass (water pressures to dissipate) the fabric acts as a filter. In this function, the fabric pore sizes and their distribution are the major parameters.

Water Transport: By allowing water to move in the plane of the fabric, it can thus act as a drain. Only certain thick, highly porous nonwoven fabrics allow effective water transport.

The full scale tests here referred to involved the separation and reinforcement functions of geotextiles. Filtration was not prevented but was not specifically included in the study. Water transport was essentially absent because test design prevented such an occurrence. Additional tests are needed to study the contribution of these latter functions in unpaved roads under various conditions.

3 FULL SCALE TEST ARRANGEMENT

A brief description of the full scale tests on which the design concept was based will be included here - although some of the aspects are also referred to in another contribution (9). The tests were carried out and the design concept developed by a team at Law Engineering Testing Co. acting as consultants to Monsanto Textiles Company.

The tests were performed under controlled conditions on full scale sections of unpaved roads. A special concrete test pit was constructed which was 5.5m wide by 9.1m long and 1.2m deep. A removable truss loading system capable of withstanding vertical reaction forces up to 130 kN (Fig. 1) was used for the plate load tests.

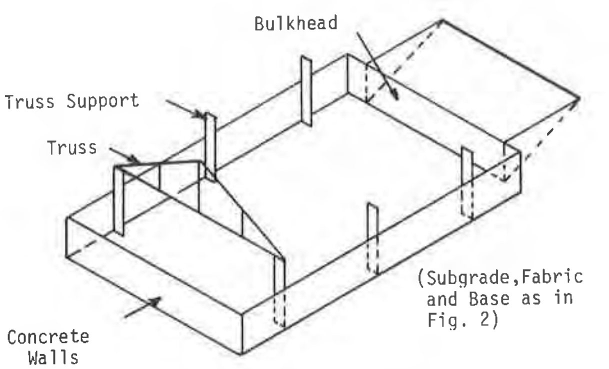

Fig. 1 Diagram of Pit for Plate Load Tests.

The pit was filled with very soft, uniformly mixed plastic clay*. The Bidim® fabric**, marked with a reference grid to allow measurement of permanent strain at the end of testing, was then positioned over the leveled subgrade. Next, the base material† was placed in three lifts with intermediate vibratory compaction. The approaches to the test level were sloped to allow truck access as shown in Fig. 2. The initial base thickness varied from .2 to .6m and the fabric area density from 0.150 to 0.340 kg/m^2. Control sections without fabric were similarily constructed.

4 PLATE LOAD TESTS WITH FABRIC

A detailed study of deflections with increasing loads, decreasing loads and after unloading, as well as the effect of rate of loading and number of loading cycles was carried out. A typical test series consisted of increasing the force on a rectangular plate the size of a dual 10.00-20 truck tire footprint. The force was supplied by a hydraulic ram, the reaction of which was

* Footnote: The subgrade had a cohesion of 3.4 kPa and was of National Standard Bentonite and 450% water.

**Footnote: Bidim® is a registered trademark of Monsanto Co. for polyester nonwoven fabrics.

† Footnote: Crusher run rock with maximum particle size of 44.4mm. In place density 85-90% max. dry density.

taken up by the truss, and was held constant at convenient intervals (about 10 to yield)$^{\Delta}$ to take displacement readings. After passing the yield point, the unloading was started. Then followed 20 cycles between zero force and 80% of yield with deflection readings at maximum force and finally a reading of the unloaded deflection.

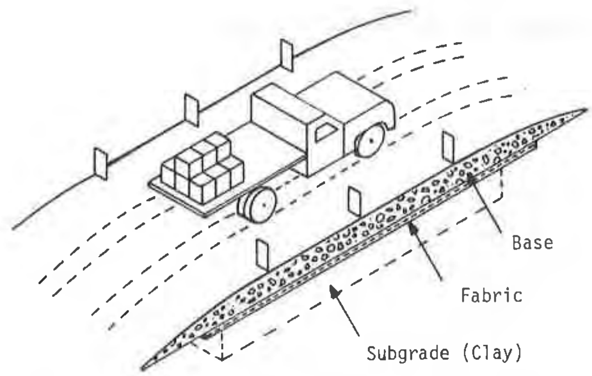

Fig. 2 Diagram of Pit for Truck Loading.

This constituted one loading sequence. A second or third or subsequent sequences were then added until the unloaded deflection reached a rut depth of 150mm. When this point was reached, repair was effected by adding additional base material into the rut area and compacting it to the original level of the top of the base material. That concluded a loading series and testing was recommenced and proceeded as above. A loading series is shown diagrammatically in Fig. 3.

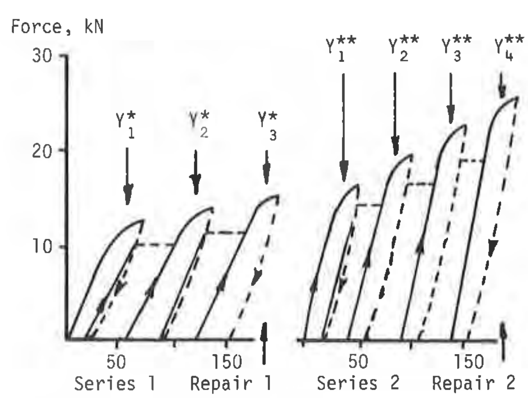

Fig. 3 Force Displacement Curves for 2 Series of Plate Load Tests (with Geotextiles).

5 TRUCK LOAD TESTS WITH FABRIC

The truck loading was accomplished using a vehicle with the weight concentrated on the rear dual wheels. It was similar to the plate load procedure and consisted of incremental and repeated loading as shown in Fig. 4.

Δ Footnote: Yield was defined as the point where the force-displacement curves showed a sudden increase in slope (at Y in Fig. 3)

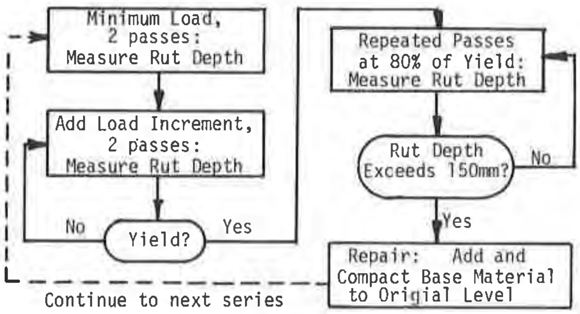

Fig. 4 Diagram of Truck Load Test Sequence.

With both types of truck loading, surface rutting was measured at regular intervals across the test section. The results of a typical incremental truck loading sequence in a series are shown in Fig. 5.

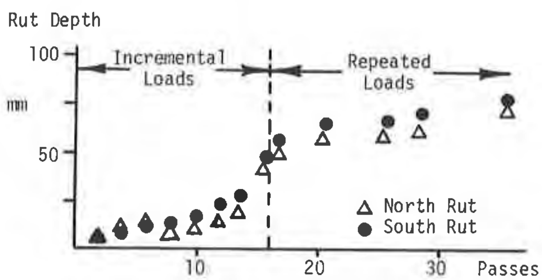

Fig. 5 Rut Depths for a Truck Loading Sequence.

The incremental loading portion of Fig. 5 and the same region for the next two loading series are shown in Fig. 6 and illustrate the large improvement brought about by the gradual tensioning of the fabric and repair of the ruts.

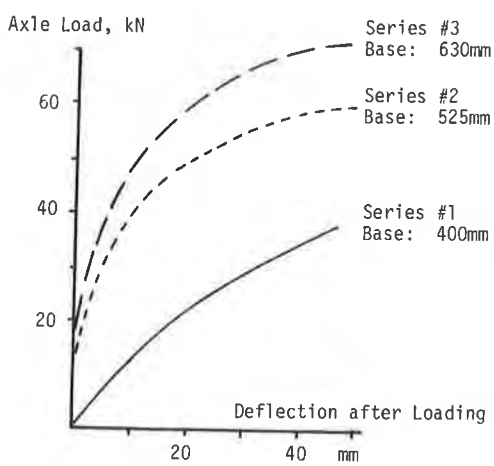

Fig. 6 Rut Depth for Incremental Truck Loading in Successive Series.

At the end of the final test series the base material was carefully excavated to expose the fabric and to make profiles of the fabric/subgrade interface. Fig. 7 shows a typical profile and indicates that maximum fabric displacement had occurred between the wheels (upward) (Fig. 7A). Analysis of such profiles led to the derivation of B', the distance between the points of inflection of the fabric.

The exposed fabric grids gave residual fabric strains between 12% and 15% across the central three-fourths of the test section and can therefore be considered essentially constant in that region (Fig. 7B).

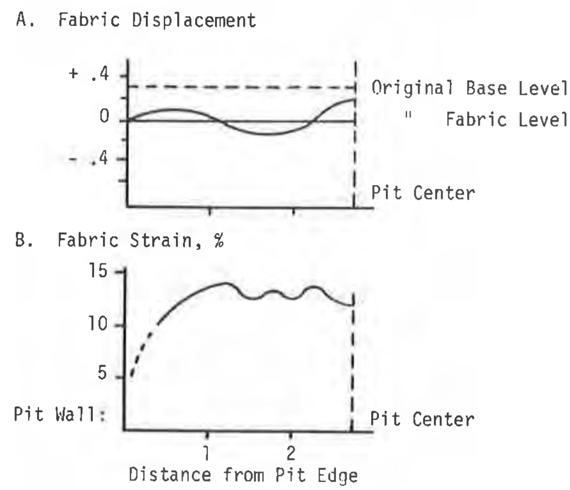

Fig. 7 Vertical Fabric Displacement and Fabric Strain after Truck Loading Tests.

6 LOAD TESTS WITHOUT FABRIC

6.1 Plate Load Tests

Tests were carried out on adjacent regions with and without geotextiles. In these early tests, only a few cycles (1 to 5) were carried between consecutive yield measurement. The course of the yield values in systems with and without fabric is shown in Fig. 8.

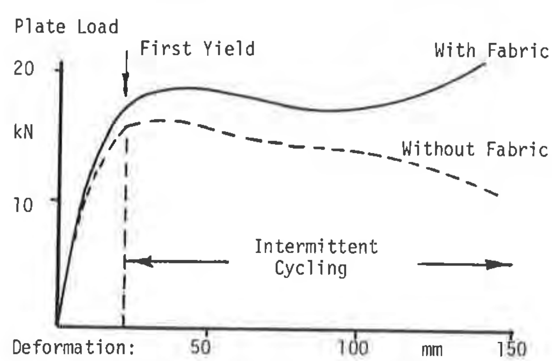

Fig. 8 Comparison of Plate Load Testing with and Without Fabric.

The presence of fabric did not significantly modify the initial system modulus but increased (by about 15%) the initial yield force. The yield points with intermittent cycling showed an accelerating downward trend without fabrics, but moved upward with fabric present. The initial yield showed a punching failure with cycling. The surface failure cracks moved away from the plate during cycling in the system with the geotextile, but no such effect could be observed without fabric.

6.2 Truck Load Tests

Few such tests were carried out because of the obvious disadvantages of irreversible damage to subsoil and base, and possible loss of a truck. However, the observations and measurements on truck load tests confirmed the findings of the plate load tests: somewhat earlier initial failure and subsequent rapid rutting and deteriorating if loading was continued near the yield value.

7 FABRIC FUNCTION DURING LOADING TESTS

Plate load tests and truck loading led to the following conclusions:

1. Only very small amounts of fine particles were visible on top of the fabric at the end of the most extensive tests, hence the fabric provided an effective separating barrier between the underlying, soft clay subgrade and the overlying base material.
2. The presence of fabric in the road structure showed only minor effects up to the first yield. Up to that point the fabric was not sufficiently deformed or tensioned to generate significant supportive forces.
3. Large increases in load bearing capacity were noted when the road system was subjected to several cycles passes of traffic at loads near yield values. This cyclic load application increases the area over which the fabric is tensioned and the resultant forces also increase.
4. After rutting and repair subsequent loading results in even larger increases in load strength. After repair the fabric is taut and in a geometrical configuration so that even small tensile forces can be converted to considerable vertical support forces.
5. The observation of uniform fabric strain (transverse to the traffic flow only) requires a knowledge of fabric stress-strain behavior under plane strain conditions. Results from other tests methods may introduce serious errors.

8 DESIGN FORMULA AND FABRIC CONTRIBUTION

8.1 General Considerations

The design concepts as developed by Sowers et al (9) are based on the traditional Terzaghi-Meyerhof (10) formula for computing soil bearing capacity but modified to include an additional term for the geotextile contribution. Thus, at the point of incipient failure at the base/subgrade interface, the resultant of the applied load (tire pressure) q_T is just balanced by the load bearing capacity of the system due to the cohesion of the soil, q_C, the contribution of the fabric, q_G, and a surcharge effect, q_S. Thus:

$$q_T = q_C + q_S + q_G \qquad (1)$$

The term q_T can be computed using the concept of the stress pyramid (11). If the tire load is F_T, the tire print dimensions B (breadth) and L (length) and the base angle of the pyramid is θ (see Fig. 9), then the effective subgrade pressure for aggregate depth z becomes

$$q_T = F_T/(L + 2z\cot\theta)\ (B + 2z\cot\theta) \qquad (2)$$

The quantity, q_C, is the product of the cohesion, c, and the cohesion bearing capacity factor N_C. The value of N_C is selected to be appropriate for the type of failure considered. $N = \pi$ for local failure (without fabric) and $N = \pi+2$ for general failure.

The surcharge term q_S was found to have a small positive value but the contribution of this term is very small except for very large values of z.

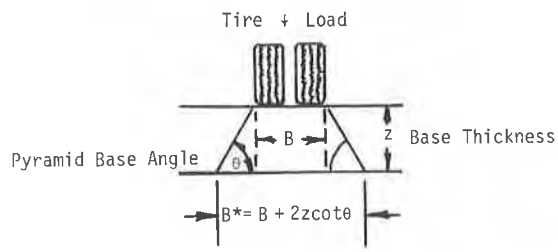

Fig. 9 Relation of Stress Pyramid Parameters.

8.2 Support Due to Fabric: q_G

The tension in the fabric contributes to support the load in two ways: directly under the wheels, the concave deflection of the fabric results is a net upward force. On the convex portions on either side of the stress pyramid, the net force is downwards and helps to restrain the soil.

The fabric contribution can be expressed as:

$$q_G = \lambda\alpha B^* \qquad (3)$$

where α is the band tension of the fabric†, and λ is the fabric strain factor related to the degree of deformation of the fabric, and $B^* = B + 2z\cot\theta$ is the breadth of the base of the stress pyramid (Fig. 9).

The factor λ was derived from an analysis of the deflections of the base/fabric/subgrade interface and the fabric strains resulting from the loading tests. It was assumed that:

1. The deflected shape of the fabric can be approximated to a series of parabolas: concave under the wheel loads and convex on either side. The points of inflection were assumed to remain at the original fabric level. The relationship between B' (the distance between the points of inflection) and B* (shown in Fig. 10) was obtained from the measurements of fabric deformation.
2. The maximum vertical deflection of the fabric, D, is related to the surface deflection of the base material by equating the displaced volumes at these two levels (Fig. 10B). The shape (volume) of the surface rut was determined experimentally.
3. The total fabric contribution is composed of $(q_G/2)$, the vertically upward resultant of the fabric tension under the wheels, and two downward resultants, $(q_G/4)$ each, under the convex half-parabolas on either side of the points of inflection.

† Footnote: Band Tension = tensile force per unit width of fabric measured under plane strain conditions.

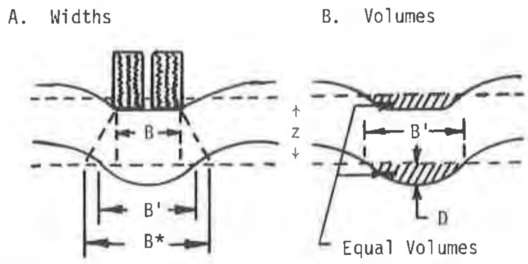

Fig. 10 Fabric Deformation Parameters.

With these assumptions and the simplifying substitution $4D/B' = \beta$ the strain factor λ can be given as

$$\lambda = 8\ (\sqrt{1+\beta^2} - 1)/\beta \qquad (4)$$

If the original fabric breadth was B' and is then parabolically deformed to a depth D the average fabric strain ε is given by (12):

$$\varepsilon = ([\sqrt{1+\beta^2} + \beta^{-1}\ \ell n\ (\beta + \sqrt{1+\beta^2})]/2) - 1 \qquad (5)$$

The value of λ for any given fabric strain ε can thus be computed.

9 MECHANICAL PROPERTIES OF THE GEOTEXTILE

None of the standard test methods for textiles (13) subject fabrics to stress conditions similar to the plane strain observed in these tests. Hence, a new test apparatus was developed in which fabrics are tensioned in plane strain (14). The test results, when converted to band tension, α, vs. strain, ε, can then be used in rational design decisions.

They are used to select values of fabric strain appropriate to the intended use and to select the most suitable fabric for optimum performance or economy. The choice of the design operating region of the fabric (i.e., the values of α and ε) will depend on the maximum traffic expected, installation and maintenance procedures and the factor of safety considered appropriate. The creep properties of the fabric must also be borne in mind.

Equation (3) shows that the greater the band tension and the greater the strain, the greater will be the fabric support. Thus, contrary to some other applications of geotextiles where deformation is not desirable, in an unpaved road fabrics with high strength and intermediate modulus show the best combination of properties. For two different fabrics with equal breaking strength but different moduli, the one with the lower modulus will give greater reinforcement (and greater rutting) for the same working stress. The compromise between reinforcement and rutting leads, for most applications, to a fabric with an intermediate modulus (a 12 to 18% strain for the working range of 30 to 60% of breaking force).

10 SAMPLE DESIGN CURVES

Some specific examples will illustrate the method for designing with geotextiles. By combining equations (1), (2), and (3) with appropriate values for N_c, q_s, α, and λ, computations can then be carried out to derive values for c for any given value of z. Some of the resulting design curves (Fig. 11) illustrate the relationship between the amount of base material required for different soil strengths for the cases of no fabric layer and three needled nonwovens of similar construction but different area densities (i.e., mass per unit area).

These design curves assumed a compacted base material of density 2160 kg/m^3, a normal force on each axle of 130 kN on dual 10.00 x 20 tires and fabric strains corresponding to about 50% of failure strength. The construction of the road is assumed to be effected with in situ tensioning of the fabric and the ruts repaired when they reach 150mm in depth. (For normal use a 150mm minimum base thickness is appropriate but this should be increased for curves and breaking areas.)

The diagram illustrates that there are large differences in base thickness required when using different fabrics for roads over very soft soil, but that the differences decrease or even vanish for firmer soil conditions. With a soil cohesion of 14 kPa it would require .15m for the B5 (area density μ = .55 kg/m^2), .23m for B3 (μ = .34 kg/m^2) and .45m for B1 (μ = .15 kg/m^2). Without fabric more than 1.5m of base material is needed.

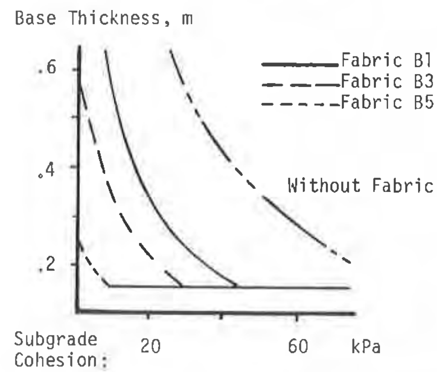

Fig. 11 Design Curves for 130 kN Axle Load on Dual 10.00-20 tires: With and Without Fabrics.

Such design curves and cost data of fabric, base material and labor can be used to calculate costs of alternative constructions. Fig. 12 illustrates the case for very soft soil c = 6 kPa (Fig. 12a) and soft soil c = 22 kPa (Fig. 12b). Cost savings of 88% for the former and 56% for the latter can be achieved using fabric 5%. The cost differences between using different fabrics can be considerable (as in Fig. 12a) or quite minimal (as in Fig. 12b).

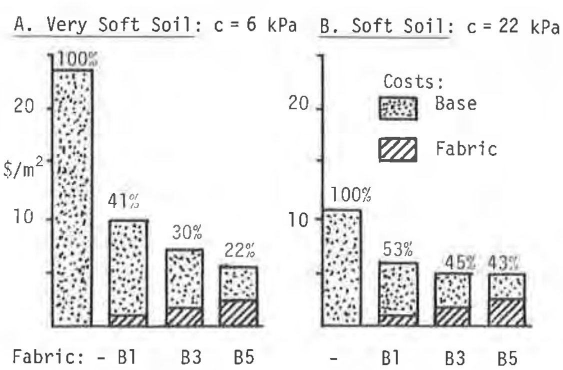

Fig. 12 Construction Costs with and Without Geotextiles.

In cases where construction costs are similar between alternative designs, the system using a heavier fabric will be a better choice, as in the case of unexpected conditions of overload or soil strength loss, the heavier fabric will continue to function when the lighter fabric may rut excessively or even fail. For firm subgrades there may appear no economic advantage in using the heavy fabrics, although here again the additional insurance built into the design may be an important consideration.

The case illustrated here is for one type of loaded vehicle. Similar curves can be constructed for other situations. In all cases the poorer the soil, the heavier the load and the greater the aggregate costs - the greater will be the cost differences and hence potential savings in using geotextiles in unpaved road constructions. In the extreme cases the strongest (greatest area density) fabrics will achieve the lowest total costs.

11 CONSTRUCTION CONSIDERATIONS

The process of in situ fabrics tensioning allows optimum use to be made of the geotextile properties. By controlled rutting of roads during construction, they can be built at times when traditionally construction has been suspended. With geotextiles road construction during the worst soil conditions, such as during the rainy season or just after the thaw, becomes not only possible but advantageous.

The importance of fabric deformation during construction (or during the initial service runs) followed by the addition of base material into the ruts cannot be overemphasized. By this process the tensile contribution of the fabric is brought into action and, in effect, the road is constructed with additional base material and additional tensile support in just those areas where it is most needed (Fig. 13), i.e., in the main traffic lanes and in the areas of weakest soil support. During construction the engineer can learn much from the pattern of rutting of the road and make most effective use of the base material.

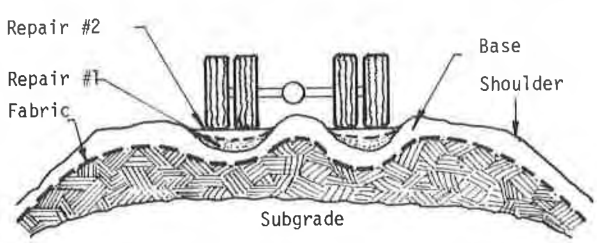

Fig. 13 Cross Section of Road with In Situ Tensioned Geotextile.

As shown in Fig. 4, several passes with a vehicle not quite filled to capacity will be more effective in promoting a firm base and larger fabric contribution than just a single pass of the load causing yielding (cracking) of the surface.

In many situations the construction traffic carries heavier loads than subsequent traffic. In that case, the road once built will be overdesigned and no significant deterioration with time will occur. Some of the base material may even be subsequently removed without detriment to the road.

12 SUMMARY

The experiments in the full scale pit together with special fabric test methods have resulted in a design procedure which makes explicit use of mechanical properties of geotextiles.

Geotextiles most suited to optimum reinforcement combine high strength and intermediate modulus in the restrained (plane strain test) mode but adequate deformability when unrestrained (grab or strip test) to allow for installation requirements. Some nonwovens because of their more random fiber arrangement and looser bonding have these capabilities.

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A Laboratory Study into Pumping Clay through Geotextiles under Dynamic Loading

Une étude en laboratoire du pompage de l'argile à travers des géotextiles, sous chargement dynamique.

A laboratory assessment of the type of test described by Snaith and Bell (1) has been carried out using a high plasticity clay subgrade to investigate the ability of such a test to give repeatable results under specific test conditions and also the effect of using different materials and loading conditions on the test results, and to investigate the mechanism causing the particle migration.

The test was found to give a generally low degree of repeatability for a given fabric under fixed test conditions and to be very dependent on material and loading conditions. Significant particle migration occurred even with a thick needle-punched fabric. Results indicated that a reduction in the strength of the subgrade at the points of contacts with the sub-base may be the major cause of particle migration and that this reduction is caused by factors including local shearing and soil suction.

Une mise à l'épreuve en laboratoire du type de test décrit par Snaith et Bell (1) a été effectuée utilisant une sous-couche d'argile à haute plasticité pour étudier la capacité d'un tel essai a donner des résultats répétables dans des conditions données, la répercussion de différentes matières et conditions de chargement sur les résultats du test, ainsi que le mécanisme causant la migration de particules.

Le test s'est avéré donner un degré de répétabilité généralement limité, pour un textile et des conditions d'expérimentation donnés, et être très dépendant de la matière et des conditions de chargement. Une migration de particules non négligeable est apparue, même dans le cas d'un textile épais finement perforé. Les résultats ont indiqué qu'une réduction de la résistance de la sous-couche aux points de contact avec la couche de base pourraient être la cause majeure de la migration de particules, et que cette réduction est dûe a des facteurs comprenant cisaillement local et suction dans le sol.

INTRODUCTION

It is well accepted that intrusion of cohesive material into the voids of a granular sub-base beneath a flexible pavement can have a severely detrimental effect on the performance of the pavement, and that contamination is more likely to occur with a sub-base deficient in fines and with a subgrade of high plasticity.

Geotextiles have been suggested as a method of separating the sub-base from the subgrade soil and preventing significant migration of particles from occurring. The mechanism by which a fabric membrane can achieve this for unidirectional flow conditions by inducing the formation of an internal soil filter is well documented (2). For severe turbulence with dynamic loading or reversing flow conditions (such as could occur under a heavily trafficked pavement), however, there is much less agreement on how a fabric soil system can provide separation and filtration (3).

Snaith and Bell (1) used a test to investigate the ability of fabrics to prevent soil migration under such conditions of dynamic loading at a particular site using particular test conditions. Their results showed that subgrade soil did migrate through the fabric under the action of the dynamic load and that different fabrics varied in their abilities to reduce particle migration.

Ayres and McMorrow (4) in discussion of these results pointed out the dangers of making inferences about the pumping action of fine grained subgrades in general from the results, since only a low plasticity subgrade was tested (containing 58% sand). They suggested that the fabric might have been forming a filter against the sand fraction with the sand itself forming a self-induced filter within the soil, and that without the sand the mechanism of filtration might have been very different.

The basic form of the test was adopted for the present work at the University of Birmingham but a high plasticity subgrade was used (5). A series of tests were performed with the object of investigating the repeatability of the results under conditions of test similar to those used by Snaith and Bell, of investigating the amount of particle migration which occurred when the various parameters involved in the test (such as the magnitude and frequency of the sinusoidally applied loading, the volume of ponded water above the subgrade, and the types of sub-base material) were varied, and of investigating the mechanism causing the particle migration.

EQUIPMENT, MATERIALS AND TEST PROCEDURE

A typical test arrangement is shown in Figs. 1 and 2.

The model subgrade material chosen for the tests was a locally occurring high plasticity marl (plasticity index 30%, plastic limit 26% and natural moisture content approximately 25%). The grading curve for the soil is shown in Fig. 3. This soil was chosen to contrast with the low plasticity material used by Snaith and Bell. The soil was placed in two layers each approximately 4 cm deep in the bottom of the 24 cm diameter mould and compacted under standard compactive effort (BS 1377 Test 12). Measured dry unit weights of the prepared samples ranged from 15.2 to 15.9 kN/m^3 with (natural) moisture contents varying between 23.8 and 26.9%. The soil was calculated to be 99% saturated.

A sample of fabric with a plan dimension greater than that of the mould was placed on top of the soil surface and a fixed mass of sub-base material placed on the fabric layer. The model sub-base was of three different types as indicated in Table 1. After placing the sub-base a specific amount (normally 0.5 litres) of water was poured gently into the mould and the sample left for 24 hours for any excess pore pressure resulting from the compaction process to dissipate.

The dynamic load was applied to the sample by a rigid loading platen using an electronically controlled servo hydraulic loading ram so that the average vertical stress S on the surface of the sample varied sinusoidally between the set limits (two stress levels were adopted viz. 20 ± 10 kN/m^3 and 50 ± 25 kN/m^2). The dynamic load was cycled at various frequencies f ranging between 0.5 and 10 hertz and maintained for different numbers of cycles n ranging from 13,500 to 216,000 cycles.

During the dynamic loading the subgrade soil could penetrate the fabric and contaminate the sub-base. The *'Soil Contamination Value' (S.C.V.) was taken as the weight of subgrade soil (gms) passing the fabric per unit area of fabric* (units of gm/m^2). By drying and weighing the contaminated sub-base and fabric at the end of a test and knowing the initial weight of fabric and sub-base material, the S.C.V. was calculated.

In a limited number of tests the pore water pressure close to the sub-base subgrade interface was measured during the load cycling. This was achieved using two small pore pressure transducers, one with its axis horizontal and the other vertical. The transducers were connected to an ultra-violet light recorder.

The moisture content profile of the top 25 mm of subgrade was obtained at the end of certain tests by pushing a thin walled tube with an internal diameter of 10 mm into the subgrade and recovering the sample. The samples were divided into 2 mm lengths and individual moisture contents taken.

Three fabrics were used in the tests and the details are given in Table 2 and Fig. 4.

Table 1 : Details of Sub-Base Materials

Designation	Description
SB1	20 mm single sized crushed rock
SB2	2-0.06 mm crushed limestone (See Fig. 3 for grading curve)
SB3	17 mm uniform spherical glass balls

Table 2 : Details of Fabrics

Designation	Description and Manufacturers Data
F1	Non-woven melt-bonded (140 gm/m^2). Terram 1000 (See Fig. 3 for grading curve)
F2	Non-woven needle-punched (thin). Neomer T425 EOS = O_{95} = 0.10, O_{90} = 0.09, O_{50} < 0.06 mm
F3	Non-woven needle-punched (thick). Neomer PB127 EOS = O_{95} = 0.11, O_{90} = 0.07, O_{50} < 0.06 mm

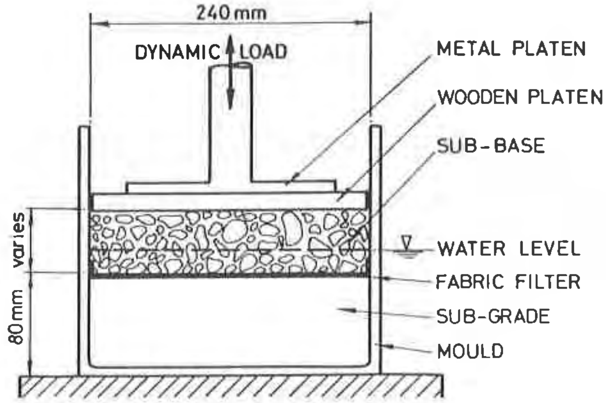

Fig. 1. Dynamic Loading Test : Section through Sample

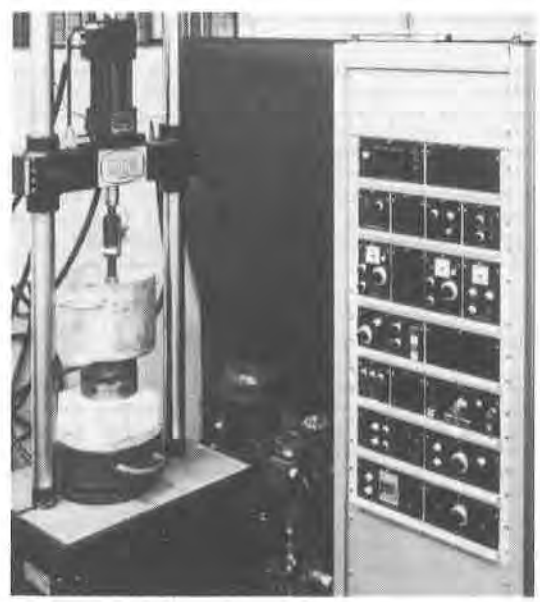

Fig. 2. Dynamic Loading Test : General View

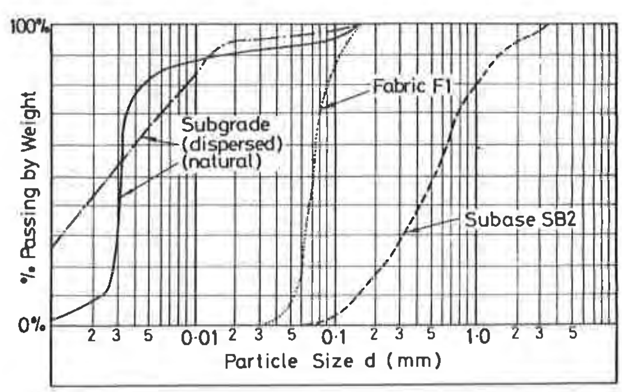

Fig. 3. Grading Curves

RESULTS

Table 3 summarises the abbreviations and definitions used in presenting the results.

Fig. 5 shows the results of tests conducted using loading ($S = 50 \pm 25$ kN/m^2) and sub-base material (SB1) as adopted by Snaith and Bell. There is too much variation between individual SCVs under these conditions for the test to be used to give a measure of a fabric's ability to control clay pumping. Since the stress level is estimated to be higher than would be obtained in practice in a pavement, a reduced stress level was then adopted ($S = 20 \pm 10$ kN/m^2) and this value was used thereafter throughout the test programme. Results using this stress level are also shown in Fig. 5, and although the SCVs still have a considerable spread the variation is not so great.

Results for a series of tests to investigate the effect of changing the number of applied stress cycles (with all other factors held constant) are shown in Fig. 6 for a range of sub-base types. These indicate a linear increase in SCV with log number of loading cycles. Since there is no rapid increase of soil migration at any number of load cycles, it may be concluded that soil liquefaction is not occurring.

Fig. 7 investigates the sensitivity of the test to changes in loading frequencies for a range of sub-base types (with all other factors including the total number of loading cycles held constant). The results are widely spread and indicate that this parameter has no significant effect on the SCV.

Table 3 : Abbreviations and Definitions

Abbreviation	Definition/Meaning
S.C.V.	Soil Contamination Value : the weight of subgrade soil passing the fabric per unit area of fabric (gm/m^2).
S	loading : applied vertical stress range (kN/m^2)
f	loading : frequency (Hertz)
n	loading : number of applied cycles
SB	Sub-base : (See Table 1)
F	Fabric : (See Table 2)
V	Volume of water added (litres)

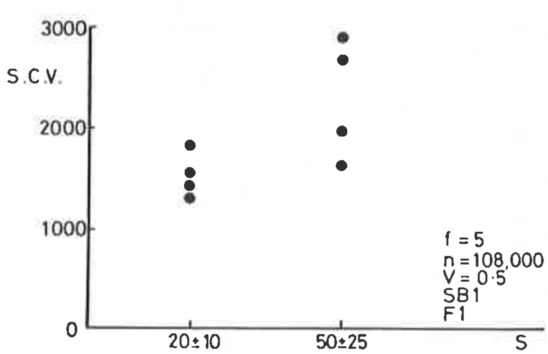

Fig. 5. Relationship between Cyclic Applied Stress Level S and S.C.V.

Tests to investigate the effect of varying the amount of water standing on top of the subgrade during the tests were conducted and the results shown in Fig. 8. They show that the volume of ponded water has no significant effect on SCV provided that there is sufficient water for the soil to "wet-up".

Results shown in Fig. 9 for a range of fabric types show that the thicker the fabric the less the SCV, but that even the thick needle-punched fabric (F3) was not capable of restraining soil migration under the conditions of test.

Pore water pressure was measured in five tests and its variation with time is shown in Fig.10. The pore pressure dissipates with time and eventually becomes equal to atmospheric pressure.

A typical deformation curve (measured as the change in height of the rigid platen with time during cyclic loading) is shown in Fig.11. It shows an initial high rate of penetration of the sub-base into the subgrade decreasing with number of applied cycles.

The moisture content profile of the top 25 mm of subgrade both directly under and at a distance away from a point of contact of a sub-base particle with the subgrade was obtained by the method previously outlined and the results shown in Fig. 12. These results are discussed later.

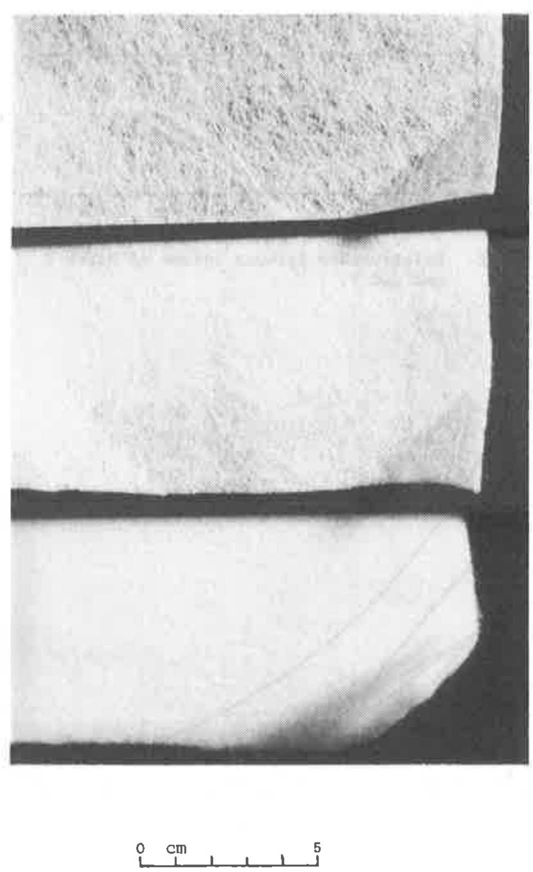

Fig. 4. Fabrics included in the Tests.

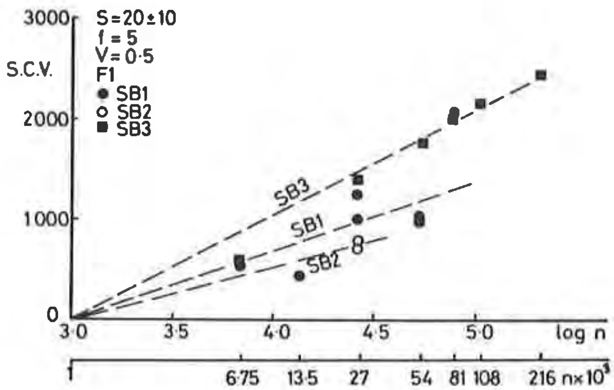

Fig. 6. Relationship between Number of Applied Cycles n and S.C.V.

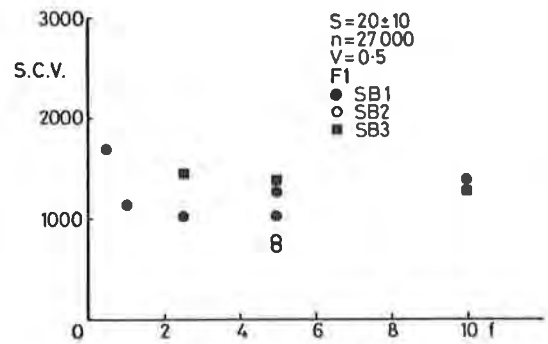

Fig. 7. Relationship between frequency of Applied Cyclic Loading f and S.C.V.

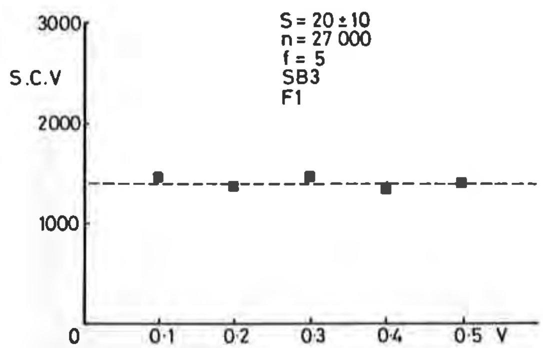

Fig. 8. Relationship between Volume of Water V and S.C.V.

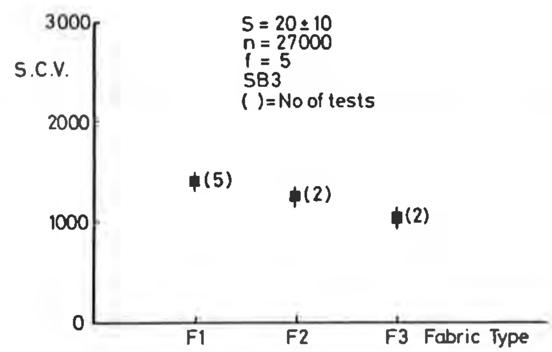

Fig. 9. Relationship between Fabric Type and S.C.V.

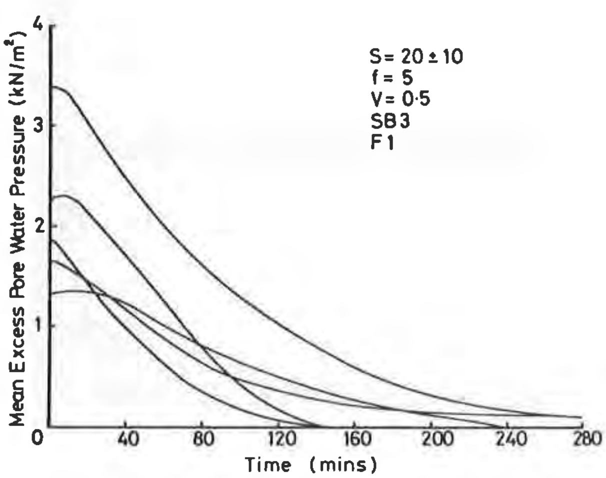

Fig. 10. Dissipation of Mean Excess Pore Water Pressure with Time.

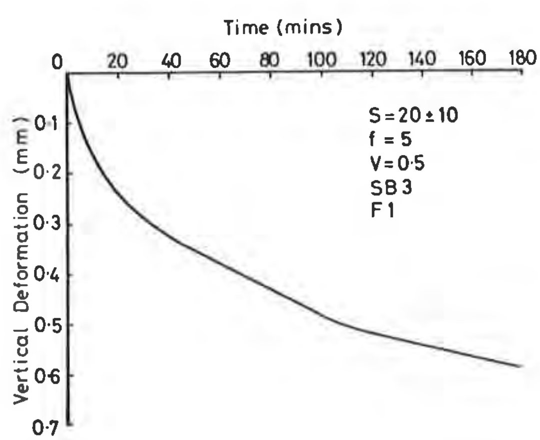

Fig. 11. Vertical Deformation of Plunger with time.

DISCUSSION OF RESULTS

Application of available fabric filter design rules (as reviewed by Hoare (3)) when applied to the high plasticity clay subgrade (containing 40% of clay and only 5% of sand) with the fabrics adopted, would indicate that soil migration is to be expected under the test conditions. The results and observations made during the test programme confirm this and endorse the comments of Ayres and McMorrow (4) referred to in the Introduction to this paper. Fig. 6 shows that soil migration was still continuing after 216,000 cycles and this indicates that the stabilising effects such as would be expected if an internal soil filter were forming (Marks (2)) is absent. Since the pore size of the fabrics is greater than that of the soil continued migration occurred.

Fig. 6 also shows the difference in SCV that can be obtained using different sub-base materials, and this when taken together with Figure 13 which shows a typical fabric specimen recovered after a test, indicate that the stress conditions and area of contact may be important factors. A close inspection of the recovered fabric revealed that the soil had passed through the fabric only at points where the sub-base particles had been in contact with the fabric. Since the area of the contacts will differ for different sub-base gradings and grain shapes, and might differ between tests using the same sub-base (especially for a normal crushed rock sub-base (e.g. SB1)), this could account for the variation in SCV obtained for otherwise identical conditions. The use of the spherical glass balls as the sub-base would give stress conditions and area of contacts similar for each particle contact, and this did give SCVs that were much more consistent than for the coarse crushed rock.

The pore water pressures induced in the subgrade during the cycling (Fig. 10) will also affect the stress conditions at the fabric interface. High initial pore pressures reduce the effective stresses between the soil particles making it easier for subgrade particles to be removed. Indeed this progressive increase in effective stress might help explain why the rate of soil migration decreases with time (Fig. 6). In some cases the pore water pressure measured had a magnitude greater than the maximum applied vertical stress. This may be explained by local higher contact stresses at the points of particle contact than the mean applied total stress level across the whole sample. This would also help explain the observation referred to previously that migration occurred at points of particle contact.

The deformation of the sub-base into the subgrade shown in Fig. 11 is compatible with the results shown in Figs. 6 and 10 and the above discussion.

The moisture content profile of the subgrade will also effect the ability of the system to resist the applied cyclic loads. From Fig. 12 it can be seen that the subgrade surface is in a very wet state in all the tests and that its value under a point of contact (48%) is approaching the liquid limit of the soil (56%). The very high moisture content values can be explained simply in terms of the local shearing which occurs as a sub-base particle punches into the subgrade causing heave of the soil around the particle. This shearing of the soil with accompanying dilation would cause a progressive increase in moisture content of the soil. Away from a point of contact the surface moisture content is about 34% and this is of a similar magnitude to the equilibrium moisture content for a soil of similar properties which has "wetted-up" due to soil suction (estimated value 30% (Black and Lister (6)). Below a depth of about 10 mm the moisture content values remain constant and equal to the moisture content of the sample after compaction (25%). At this depth the soil is neither being sheared nor has an

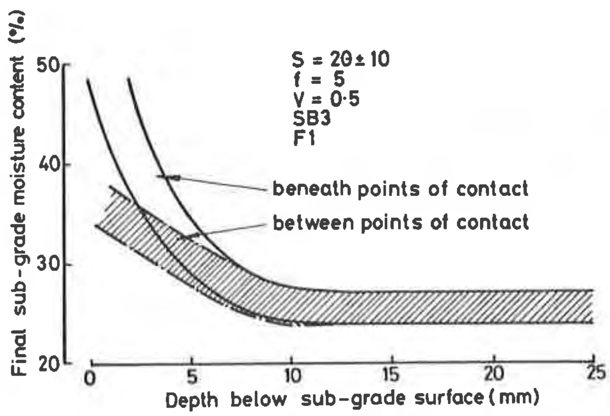

Fig. 12. Moisture Content Profile of Subgrade.

Fig. 13. Typical Fabric Specimen after a Test: showing migration of subgrade soil only at points of contact with sub-base particles.

adequate supply of water to satisfy the soil suction. At these high moisture contents a remoulded soil of these properties will have a very low shear strength estimated from Black and Lister (6) as 3 kN/m^2 at 46% moisture content, 19 kN/m^2 at 34%, 160 kN/m^2 at 25% moisture content. Irrespective of the accuracy of these figures, their order of magnitude demonstrate that the mechanism of soil squeezing through the fabric will occur most readily at points of particle contact. Since also the fabric may be stretched in this area of particle contact the effective pore size of the fabric will increase locally further decreasing its resistance to soil migration.

If punching of the sub-base into the subgrade is the fundamental cause of the soil migration through the fabric, then reducing the stresses at the points of contact should reduce the punching effect and hence reduce the SCV. The results shown in Fig. 5 do show that reducing the stress level reduces the SCV, and also the results shown in Fig. 6 demonstrate that using a finer sub-base material (SB2) which will increase the number of particle contact points and hence reduce the contact stresses also reduces the SCV.

CONCLUSIONS

1. The amount of soil migration measured as the Soil Contamination Value (SCV) is very variable when 20 mm single sized crushed rock aggregate is used as the sub-base especially at high applied stress levels. The use of spherical glass balls for the sub-base gives much more repeatable SCVs and the use of this type of material is recommended as a basis for future comparative tests of the performance of different fabrics.

2. The frequency of the applied loading has little effect on the SCV when the frequency is in the range 2.5 to 10 Hz.

3. The effect of varying the volume of water used to simulate the effect of ponded water on the subgrade surface seems to have little effect on the SCV provided there is sufficient moisture available for the soil to wet up.

4. The SCV increases linearly with the log of the number of applied cycles.

5. Soil migrating through the fabric did so at the points of contact of the sub-base particles with the subgrade.

6. The moisture content of the soil at the points of contact with a sub-base particle increases to a value close to its liquid limit. This may be a result of soil suction and local shearing caused by the high stresses at the points of contact.

7. The phenomenon of clay pumping is thought to be caused by this wet (and hence soft) soil on the surface of the subgrade squeezing through the fabric.

8. The SCV may be reduced by decreasing the stresses at the points of contact of the sub-base particles on the subgrade. This may be achieved by either reducing the applied stress levels or by using a sub-base with a finer grading or higher fines content which will increase the number of points of contact and hence reduce the stresses at each point of contact.

9. The SCV was not significantly reduced by the use of a heavier needle-punched fabric rather than a light melt-bonded fabric.

ACKNOWLEDGEMENTS

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Measurement of Anticontaminant Property of Fabrics Under Simulated Traffic Load

Mesure du pouvoir anticontaminant d'un géotextile sousmis à un trafic simulé

The paper reports an experimental investigation for controlling the anticontaminant power of 13 differents geotextiles. The study consist of dynamic a loading test carried out on a soil-gravel system with fabrics placed at the subsoil-granular material interface. The subgrade materiel is a silt at low bearing capacity (CBR = 2) and the subbase materiel is an uniformly graded gravel with 1,2 void ratio. The used loading device is "la dynaplaque" which simulate an equivalent effect of axle load traffic. Measurements are mainly based on sieve analyses and comparison of size distributions before and after test of the granular subbase material. It is shown from the test that the anticontaminant propertie exist for non-woven fabrics and woven fabrics with apertures of some millimeters and at large deformation. However there is no anticontaminant effect if cracking or tear of geotextiles occur.

Cette communication présente une étude expérimentale destinée à caractériser le pouvoir anticontaminant de 13 géotextiles différents. L'étude a consisté à charger dynamiquement une structure constituée d'une couche de gravier reposant sur un sol naturel, les 2 matériaux étant séparés par le géotextile à tester. Le sol est un limon de très faible portance (CBR = 2) et le matériau granulaire un gravier homométrique d'indice de vide de 1,2. Le chargement dynamique a été réalisé à l'aide de la dynaplaque, appareil simulant les sollicitations engendrées par l'essieu d'un véhicule lourd. Les mesures ont principalement consisté en des analyses granulométriques du gravier avant et après les essais de chargement. D'après ce test, il apparaît que le pouvoir anticontaminant existe bien pour les tissés et non tissés même lorsque ces produits présentent des pores de l'ordre du mm et ont subi de grandes déformations. En revanche, ce pouvoir disparaît brutalement lorsqu'une rupture du géotextile se produit.

I - INTRODUCTION

Dans la contruction des routes, des voies ferrées, des pistes de chantier, des aires de stockage et de stationnement etc... il est généralement nécessaire de mettre en oeuvre une ou plusieurs couches de matériaux granulaires plus ou moins élaborés sur les sols en place car les caractéristiques de ces derniers ne sont pas suffisantes pour assurer les fonctions dévolues à ces différentes infrastructures.

Dans le cas très fréquent où les terrains en place sont argileux, l'épaisseur de matériau granulaire à épandre doit tenir compte à la fois du niveau des caractéristiques demandées à la structure ainsi constituée et du risque de voir ces caractéristiques diminuer au cours de la vie de l'ouvrage du fait de l'interprétation progressive d'une certaine quantité de sol argileux dans les interstices du matériau granulaire sous l'action des contraintes dynamiques (pétrissage) engendrées par le trafic.

En effet, l'expérience de la géotechnique routière a montré qu'il suffit généralement d'introduire une faible proportion de sol argileux (de l'ordre de 5 à 10 %) dans un matériau granulaire pour lui conférer un comportement mécanique analogue à celui du sol argileux.

Pour se prémunir vis-à-vis de ce risque, l'ingénieur routier a le choix entre deux possibilités : soit surdimensionner l'épaisseur de matériau granulaire c'est-à-dire admettre qu'une partie de ce matériau sera perdue au cours de la vie de l'ouvrage, soit réaliser à l'interface sol argileux-matériau granulaire une couche d'interposition appelée généralement couche anticontaminante. empêchant le développement du phénomène. Les inconvénients de la première solution sont son coût et l'imprécision de l'estimation que l'on peut faire du surdimensionnement réellement nécessaire ; ceux de la seconde résultent de l'insuffisante fiabilité de son exécution pratique.

Le grand intérêt des géotextiles est de constituer une technique de réalisation de couche anticontaminante parfaitement fiable sur les plans technique et opérationnel et économiquement plus satisfaisante dans un nombre toujours croissant de cas de chantier.

Ceci explique le grand développement qu'ont connu ces matériaux dans cette application depuis plus d'une dizaine d'années ; encore fallait-il, devant l'élargissement continu de la panoplie des produits géotextiles proposés sur le marché, être en mesure de caractériser objectivement le pouvoir anticontaminant de chacun d'eux.

C'est précisément l'objet de cette communication de décrire un essai imaginé spécialement pour quantifier le pouvoir anticontaminant de ces produits et de présenter les résultats obtenus sur un éventail représentatif des principaux géotextiles utilisés actuellement comme interface anticontaminant.

2 - ETUDE REALISEE

L'étude est réalisée au Centre d'Expérimentations Routières de Rouen (France). Elle consiste en des essais en vraie grandeur dans des fosses d'essais. Le plan d'expérience comporte deux modalités témoins (structure non munie de géotextile) et une série de modalités avec 13 géotextiles de différents types.

2.1 - Principe de l'essai

L'essai consiste à solliciter, au moyen d'un dispositif de simulation de trafic, une structure comportant une couche de 30 cm d'épaisseur de graviers propres mise en oeuvre sur un sol-support en limon à très faible portance.

Le géotextile testé est mis en place à l'interface des couches de graviers propres et du sol fin.

La mesure du pouvoir anticontaminant est réalisée en effectuant des prélèvements à différents niveaux dans la couche de gravier et en procédant à leur analyse granulométrique pour y déceler la présence éventuelle de limon provenant du sol-support.

2.2 - Dispositif de simulation du trafic

Pour solliciter dynamiquement la structure de manière analogue à ce qu'elle subit sous l'action d'un trafic, on s'est servi de l'appareil appelé "Dynaplaque" utilisé couramment en France pour mesurer la portance des plateformes support de chaussées.

Le principe de cet appareil (figures 1, 2, 3) est d'appliquer sur la plateforme à ausculter une sollicitation dynamique analogue en intensité et en fréquence à celle provoquée par le passage d'un essieu chargé à 130kN se déplaçant à 60 km/h. Celle-ci s'obtient par la chute d'une masse que l'on laisse tomber sur une plaque d'appui garnie d'une couronne de ressorts.

On mesure la réponse de la plateforme à cette sollicitation à partir du coefficient de restitution énergétique du choc exprimé simplement par le rapport entre les hauteurs de chute et de rebond de la masse. La durée d'un essai n'excède pas 2 à 3 minutes.

Si en un même point on applique un grand nombre de chocs on simule l'effet d'un trafic et c'est dans ce sens que la Dynaplaque a donc été utilisée dans l'étude du pouvoir anticontaminant des géotextiles dont il est question ici.

2.3 - Structure d'essai

2.3.a - Description

Les essais sont effectués dans des fosses d'essais de dimensions en surface de 3 m x 3 m. Ils sont réalisés par série de 4.

(figures 4, 5).

fig. 1 Vue d'ensemble de la dynaplaque

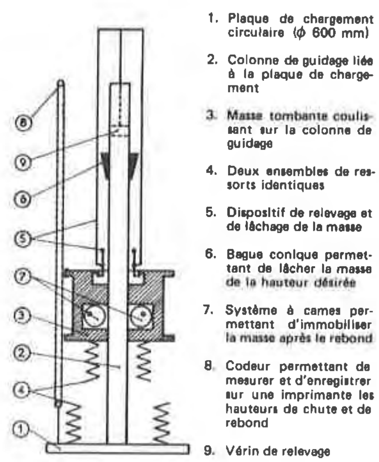


fig. 2 Schéma de principe de la dynaplaque

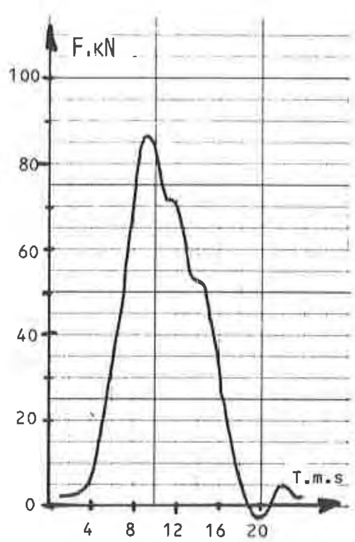


fig. 3 Graphe de l'effort en fonction du temps

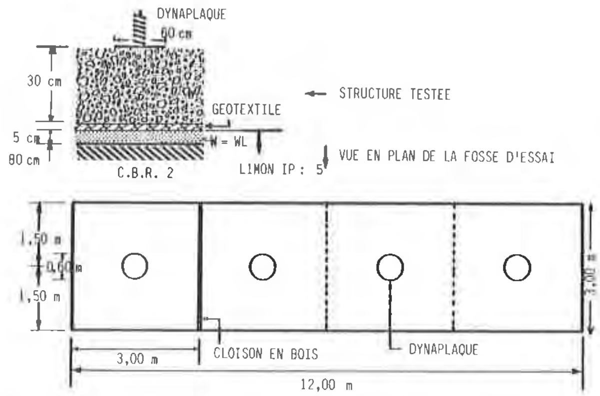


fig. 4 Structure d'essai et fosse d'essai

fig. 5 Vue de la dynaplaque en cours d'essai

2.3.b - Caractéristiques des matériaux utilisés

b.1 - sol-support

Le sol-support est un limon d'indice de plasticité IP = 5 (WP = 25, WL = 30). L'indice portant immédiat CBR est de 2. Pour favoriser la contamination de la couche de graviers propres à travers le géotextile, une couche de 5 cm du même limon à la teneur en eau de limite de liquidité est mise en oeuvre à la surface du massif support (cf figure 1).

b.2 - graviers propres

Il s'agit d'un gravier silico-calcaire semi-concassé 3/8 mm. Le choix d'un matériau de granulométrie étroite permet de s'affranchir de l'évolution importante de densité du matériau au cours de l'essai de chargement et d'avoir un matériau à indice des vides élevé ($e_0 \simeq 1,2$), donc très favorable au développement du processus de contamination.

(figure 6).

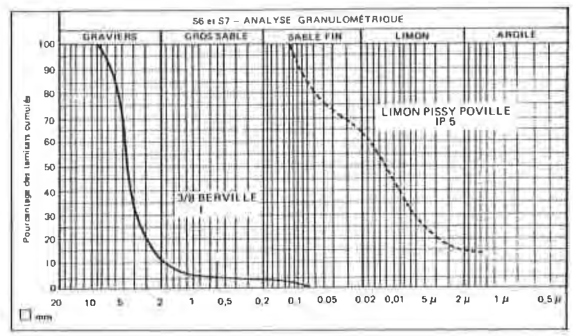


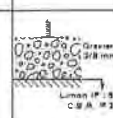
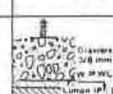
fig. 6 Courbes granulométriques du sol-support et des graviers utilisés

2.4 - Géotextiles testés

Le tableau figure 7 donne les caractéristiques des géotextiles testés au cours de ces essais. Ce sont :

- des non-tissés aiguilleté, lié ou thermosoudé en polyester ou en polypropylène de masse surfacique variant de 140 à 350 g/m² (8 échantillons)

tableau 7. Modalités d'essais réalisés

N° des modalités	Structure d'essai	GEOTEXTILE UTILISE				MODALITE
		Type	Matière	Grammage (g/m²)	Caractéristiques particulières	
1						témoins sans géotextile
2						
3		non tissé aiguilleté	Polyester	340		avec géotextile
4		tissé de bandelettes	Polypropylène	140		
5		non tissé	Polyester	350		
6		non tissé thermosoudé	Polypropylène 75 % + Nylon 25 %	210		
7				140		
8		tissé de bandelettes	Polypropylène	220		
9		non tissé aiguilleté	Polyester	150		
10		non tissé	Polypropylène 95 % + Polyester 5 %	150		
11				300		
12		non tissé aiguilleté	Polypropylène	150		
13		grille Fils 1 100 dtex	Polyester	140	maille de 2 mm x 4 mm	
14			(enrobage polyvinyle)	70	maille de 4 mm x 4 mm	
15		grille	Polyéthylène basse densité	450	maille de 8 mm x 8 mm	

- des tissés de bandelettes de masse surfacique de 140 et de 220 g/m^2 (2 échantillons
- des grilles de masse surfacique variant de 140 à 450 g/m^2. La surface de maille varie de 8 mm^2 à 36 mm^2 (3 échantillons)

2.5 - Paramètres mesurés

Les paramètres mesurés sont :

- La teneur en fines (% < 80µm) après essai
- Les tassements en fonction du nombre de chocs
- Le coefficient de restitution de la dynaplaque en fonction du nombre de chocs

Ces 2 derniers paramètres permettent surtout de contrôler les conditions d'essais.

2.5.a - Mesure de la teneur en fines

L'analyse granulométrique est effectuée sur des prélèvements par couche élémentaire des graviers 3/8 mm après 128 chocs. L'épaisseur de ces couches élémentaires prélevées varie d'une modalité à l'autre. (figure 8).

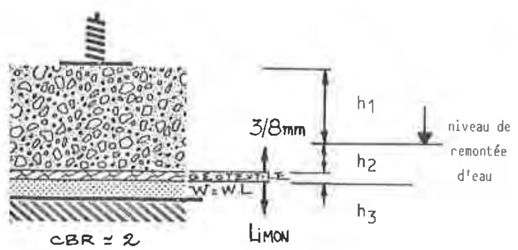


fig. 8 Prélèvements pour analyse granulométrique

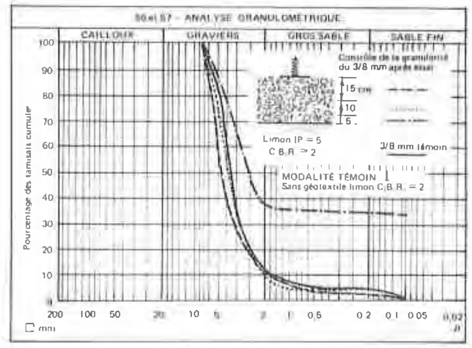
3 - RESULTATS D'ESSAI (Tableau 2)

3.1 - Structure témoin sans géotextile (modalités 1 et 2)

La contamination des graviers est observée dans les 2 cas. Celle-ci est nettement plus importante dans le cas de la modalité 2 que dans celui de la modalité 1. Pour les 5 cm inférieurs de la couche de graviers, la proportion de limon est de l'ordre de 30 % en poids pour la modalité 1 et de l'ordre de 95 % pour la modalité 2. La présence du limon liquide à l'interface accentue nettement la contamination de la couche supérieure (figure 9).

3.2 - Structure avec géotextile (modalités 3 à 15)

Pour ces modalités, on distingue d'une part les géotextiles tissés et non-tissés et d'autre par les géotextiles de type grilles.



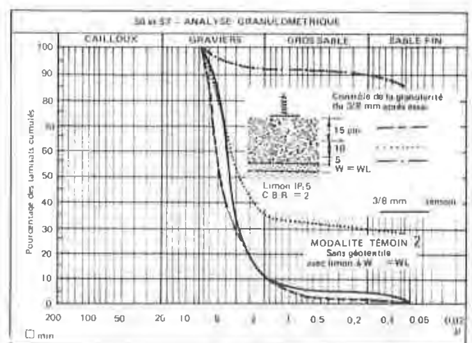


fig. 9 Mise en évidence de la contamination dans le cas des structures témoins

3.2.a - Cas des géotextiles tissés et non-tissés

Pour les modalités où il n'y a pas eu de rupture du géotextile, il y a toujours eu anticontamination parfaite. (figure 10).

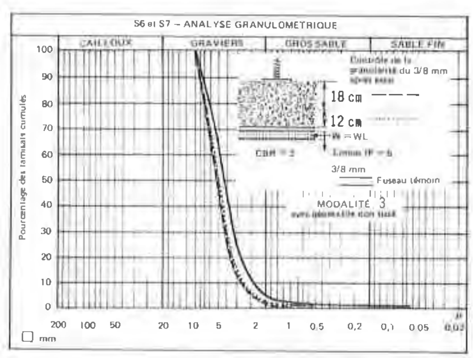


fig. 10 Absence de contamination dans le cas d'une structure comportant un géotextile qui ne s'est pas rompu

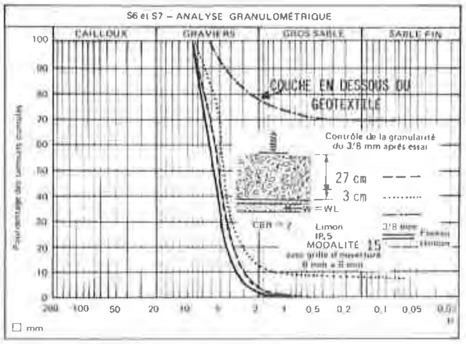


fig. 11 Mise en évidence de la contamination dans le cas d'une structure comportant un géotextile (grille) qui s'est rompu

Dans le cas où le géotextile s'est rompu, il y a contamination de la couche de gravier.

Elle reste relativement faible par rapport à celle constatée dans la modalité 2.

3.2.b - Cas des grilles

Pour la grille de maille 2 mm x 4 mm, il n'est pas observé de rupture du géotextile, ni de contamination de la couche de gravier.

Pour les grilles de 4 mm x 4 mm et de 6 mm x 6 mm de dimensions de maille, il y a eu rupture et contamination de graviers dans les 2 cas. La contamination est du même ordre dans les 2 cas et inférieure à celle des modalités témoins. L'ensemble des résultats est résumé au tableau 12.

3.3. - Considérations sur les mesures des tassements et des coefficients de restitution

La figure 13 donne les tassements cumulés mesurés sur les structures témoins (sans géotextiles) avec et sans limon à $W = W_L$ et sur 3 structures avec un géotextile de masse surfacique de l'ordre de 140 g/m^2.

On constate qu'après 128 chocs de dynaplaque, les tassements sont de l'ordre de 20 cm et ne sont pas significativement différents pour les structures avec et sans géotextile.

Cette valeur relativement élevée des tassements confirme à la fois la validité de la dynaplaque en tant que simulateur de trafic et la grande aptitude de la plupart des géotextiles à conserver leur pouvoir anticontaminant même lorsqu'ils sont soumis à de grandes déformations.

En ce qui concerne les valeurs des coefficients de restitution, elles sont restées, quels que soient les modalités et le nombre de chocs, comprises entre 10 et 20 %, valeurs qui traduisent une portance très faible (correspondant en gros à un module à la plaque de 5 à 10 MPa). Ceci s'explique à la fois par la portance du sol-support et par le comportement particulier du gravier 3/8 se rapprochant de celui d'un ballast (très amortissant et très déformable). Il n'a donc pas été possible de proposer une interprétation plus poussée de ces mesures.

tableau 12. Tableau récapitulatif des résultats d'essais

N° des modalités	GEOTEXTILE UTILISE	CONSTATATION SUR LA CONTAMINATION DU 3/8 mm				CONSTATATION SUR LE GEOTEXTILE
		épaisseur e	1ère couche h_1	2ème couche h_2	3ème couche h_3	
1	néant	oui	non	non	oui 5 cm 35% limon	-
2	néant	oui	non	oui 10cm 25% limon	oui 5 cm 90% limon	-
3	non tissé aiguilleté	non	non	non	-	INTACT
4	tissé de bandelettes	non	non	non	-	INTACT
5	non tissé	non	non	non	-	INTACT
6	non tissé thermosoudé	non	non	non	-	INTACT
7	non tissé thermosoudé	non	non	non	-	INTACT
8	tissé de bandelettes	non	non	non	-	INTACT
9	non tissé aiguilleté	non	non	non	-	INTACT
10	non tissé	non	non	non	-	INTACT
11	non tissé	oui	non	oui 9cm 5% limon	-	DECHIRE
12	non tissé aiguilleté	non	non	non	-	INTACT
13	grille	non	non	non	-	INTACT
14	grille	oui	non	oui 2cm 5% limon	oui 3,5 cm* 83% limon	DECHIRE
15	grille	oui	non	oui 2cm 10% limon	oui 2cm* 75% limon	DECHIRE

* prélèvement au-dessous du géotextile

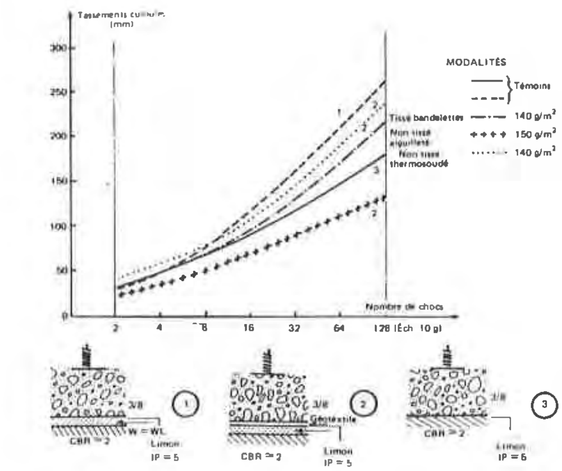


fig. 13 Exemples de tassements cumulés enregistrés pour quelques modalités

4 - CONCLUSIONS

L'expérimentation réalisée a permis de proposer un essai fiable et relativement simple pour tester le pouvoir anticontaminant d'un géotextile placé dans une structure devant supporter un trafic.

Pour les treize testés constituant un échantillon représentatif des géotextiles utilisés en couche anticontaminante en France, on a pu constater que :

- Le pouvoir anticontaminant de ces produits est parfaitement réel même lorsqu'ils présentent des pores de l'ordre du millimètre.

- Cette propriété est conservée dans le cas où le géotextile subit de grandes déformations mais elle disparaît brutalement dès qu'une rupture du produit apparaît.

La première conclusion relative au diamètre des pores assurant encore l'anticontamination peut éventuellement surprendre étant donné les dimensions beaucoup plus faibles des particules de sol dont on veut empêcher la migration.

Il faut toutefois être bien conscient que ces résultats ne concernent pas le problème de la contamination provoquée par le déplacement de particules dans un écoulement (filtration).

La seconde conclusion montre la nécessité de dimensionner judicieusement le géotextile utilisé comme couche anticontaminante eu égard aux sollicitations qu'il subira au cours de sa mise en oeuvre et durant la vie de l'ouvrage.

On renvoie pour cela aux règles définies dans les fascicules de Recommandations éditées par le Comité Français des Géotextiles [1] [2].

BIBLIOGRAPHIE

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[2] *Recommandations pour l'emploi des géotextiles dans les voies de circulation provisoire, les voies à faible trafic et les couches de forme. Fascicule édité par le Comité Français des Géotextiles* I.T.F. *35, rue des Abondances 92100 Boulogne BP 79*

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An Experimental Investigation of Sub-Base Protection Using Geotextiles

Une recherche expérimentale sur la protection des couches inférieures avec des géotextiles

Highway sub-bases or railway ballast can become contaminated under the action of traffic loading, from clay fines derived from cohesive subgrades. Such contamination should be minimized since it can produce substantial weakening of the sub-base layers. Although the incorporation of a geotextile filter layer between subgrade and sub-base has been widely suggested as a possible solution to the problem, little detailed information for this application is currently available.

As a result of some highway failures in Northern Ireland, a laboratory investigation of geotextile protection for sub-bases was begun. The initial results of this study have been published previously and the present paper reports on the latest findings.

The paper discusses the mechanisms controlling sub-base contamination and its prevention, and recommendations for the specifications of suitable geotextile filters are made. It is suggested that thick relatively incompressible geotextiles with low poresizes will be effective in limiting sub-base contamination to an acceptable level.

Les couches inférieures des routes, ou le ballast des voies Ferrées, peuvent être contaminés, sous l'effet de la charge du trafic, par des particules fines d'argile provenant des terrains de formation cohérents. Une telle contamination devrait être reduite aux minimum, car elle peut causer un affaissement considérable des couches formant la couche inférieure. Malgré les maintes suggestions, comme solution possible au probleme d'incorporer une couche filtranté en geotextile entre le terrain de formation et la couche, il existe à présent tres peu d'informations detaillées sur cet usage.

Par suite de certaines détériorations des routes en Irlande du Nord, une recherche, meneé en laboratoire, sur la protection par des geotextiles des couches enferieures a été entreprise. Les premiers résultats de cette étude out déjà été publiés; ce traité fait part des dernières detouvertes.

L'exposé traite des mécanismes reglant la contamination des couches enférieures dinsi que des moyens de l'éviter, et recommande certaines spécifications pour des filtres en geotextile appropriés.

1. THE NEED FOR SUB-BASE PROTECTION

Many highways and railways passing over cohesive subgrades are constructed with relatively coarse granular sub-base or ballast materials. Traditionally, such materials are considered to have good drainage properties and high shearing resistance. However their performance can be considerably impaired if the voids become filled with material from the subgrade. This can occur by direct penetration of the granular particles into the fine cohesive soil, or via a pumping action caused by the passage of traffic.

Subgrade softening will usually be a prerequisite for the occurrence of either of these processes. Softening can arise due to ingress of water from surface infiltration, from subgrade suction, or from artesian flows (1). Surface infiltration can occur at any time during the life of the structure. However during construction in wet weather when large areas of the formation or the permeable granular layers are exposed, large amounts of infiltration will occur. Wet weather construction may also hasten softening arising from subgrade suction (2). After construction infiltration rates will depend upon the efficiency of the pavement drainage system, and the potential for water flow into it.

Once sufficient softening has occurred the coarse granular particles can relatively easily penetrate the subgrade. This will weaken the road or railway structure by reducing its effective depth and by impairing the engineering properties of the sub-base. Not only will the permeability of the material be influenced, for example see ref (1), but also lateral gradients will be altered causing local depressions in which water will form small pools, (3). In some cases a more severe "bathtub" condition (4) may arise in which large areas of the subgrade are submerged under free water which cannot drain quickly owing to the low permeability of the sub-base and shoulder material.

In the presence of such water, and under a 'pumping' action caused by the passage of traffic, fines from the softened cohesive soil can form a slurry and migrate into the sub-base. This will result in further loss of strength and permeability. Large quantities of contamination are not required to cause damage. For example Bell, McCullough and Gregory (3) show that the static shear strength of three different sub-base materials was reduced by 20% to 40% when contaminated with cohesive fines in quantities ranging from approximately 2 to 4 per cent of the dry weight of the sub-base aggregate.

It is not always possible to prevent softening of the subgrade either during or subsequent to construction, perhaps because of the groundwater or soil suction characteristics at the site, or because of inadequate permeability of specified sub-base or ballast layers, eg (1).

Consequently some means of minimizing contamination of the sub-base from penetration of cohesive subgrades, or from fines pumping is often required. Granular filter layers of various types have been specified and used with success in the past. However within the last decade permeable fabric membranes (geotextiles) have also been proposed for use as separators and filters between cohesive subgrades and sub-bases (5,6). The present paper describes laboratory investigations of the use of geotextile and granular filters designed to prevent clay fines migration from the subgrade into the overlying granular layers.

2. BACKGROUND TO THE INVESTIGATION

The Queen's University of Belfast in conjunction with the Northern Ireland Roads Service first became interested in the prevention of clay contamination of sub-base material as a result of the inadequate performance of some stretches of major roads. Most of the recently constructed roads in N. Ireland are of the flexible type, typically consisting of layers as follows:

Total Depth 655 mm:
- 100 mm Asphalt (wearing & base course)
- 80 mm Dense bitumen macadam
- 225 mm Granular road base
- 250 mm Granular sub-base

Subgrades in Northern Ireland are frequently cohesive till, with natural moisture contents in the range 16%-33% index of plasticity from 15%-30%, and containing 10%-25% of clay sizes, with silt often being the predominant size range.

The road base and sub-base materials respectively are often specified as relatively single sized 50 mm (2") and 100 mm (4") crusher-run stones. They are quarried from the widespread basalt or gritstone (Greywacke) sources in the province, and have been successfully used in the past. In addition they are cheap, can be used in poor weather owing to their high drainage capacity, and their straightforward specification leads to simple and effective site control.

However with increased axle loads and traffic volume, rutting failure of some major stretches of recent new roads was noted within months of construction. Although rutting in general can be attributed to a number of causes, for the Northern Ireland failures it is believed to be associated with deformations in the granular layers or in the subgrade (7). Observations of the pavement layers in many of the failed sections of road revealed that quantities of clay were present in the base and road-base materials. It is thought that this contamination was a major factor contributing to the rutting and that it had arisen as a result of a combination of the frequent wet weather conditions during and subsequent to construction, and to the open voided nature of the sub-base and base materials.

As a result of these observations the Northern Ireland Roads Service with the Department of Civil Engineering at the Queen's University of Belfast jointly initiated a programme of research into the behaviour of sub-bases typical of those used in N.I. roads. Part of this work paid particular attention to the problem of clay contamination.

Firstly, laboratory tests were undertaken which confirmed the adverse influence of clay contamination on the shear strength of granular sub-base materials (3). Secondly, a laboratory study of the possible prevention of clay migration using fabric membranes was conducted. This indicated that the performance of some fabrics was poorer than expected (8). Laboratory tests by other researchers (9) using a subgrade containing considerably higher clay content than ref (8) suggested that no fabric currently available could prevent fines migration.

Thirdly a field trial, in damp conditions, was conducted which incorporated an unprotected control section, a section in which a granular filter was placed between sub-base and subgrade, and a section in which a nonwoven geotextile was used. Further details of the trial can be found in references (3) and (7). Although the trial was limited in terms of its size, deviation and traffic volume, certain findings emerged. It was demonstrated that clay contamination can occur in typical sub-base material as used in N. Ireland, both by penetration into a softened subgrade and by pumping of subgrade fines into the sub-base voids. Further the nonwoven was relatively ineffective in preventing clay fines contamination but was more successful in preventing penetration of the sub-base into the softened subgrade. The granular filter seemed to perform well, with no evidence of clay fines migration.

Although such findings did not support the adoption of geotextiles or subgrade filters in road construction the Queen's University of Belfast jointly with the Lambeg Industrial Research Association and the N.I. Roads Service are continuing to examine the problem. This is because of the need to gain a better understanding of both geotextile filtration and subgrade-sub-base interaction. It was also recognised that there are large potential benefits to be gained from using geotextiles because they are relatively cheap, they can be easily and cheaply placed on site, and yet their quality can be controlled off site.

3. DYNAMIC LOAD LABORATORY FILTER TESTS

The tests described in this section were conducted in order to compare the performance of several different filter types in preventing the migration of subgrade fines into the sub-base.

3.1 Subgrade and Sub-base Materials and Filters Employed in the Tests

The subgrade soils used in the tests are typical of subgrade types found beneath roads in Northern Ireland. The soil used for test series A and C (Table 2) was obtained from a road construction site at Whiteabbey, North Belfast, and in its natural state can be described as a stiff brown silty clay containing some cobbles and occasional boulders. Particles larger than sand size were removed before using the soil for the laboratory tests. A grain size distribution curve for the soil is shown on fig. 1 and classification and compaction test results are as follows:

Liquid limit	:	38%	
Plastic limit	:	19%	
Max. Dry Density	:	1,760 Mg/m^3	BS1377
Optimum moisture content	:	18%	'Standard' compaction

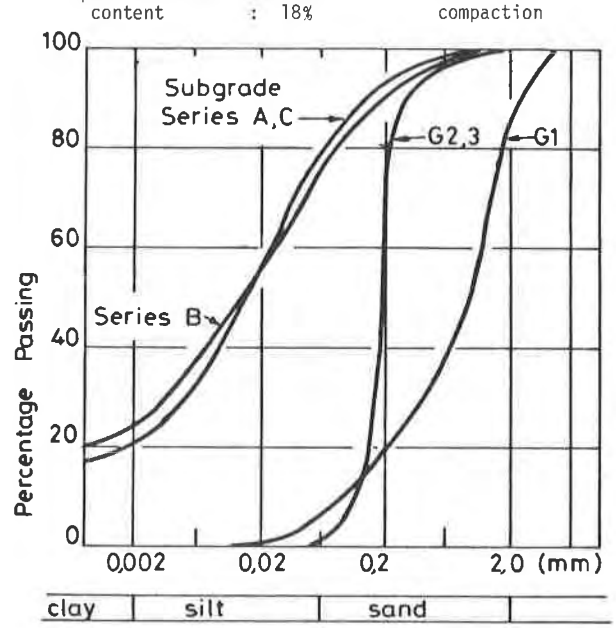

Fig. 1 Subgrades and Granular Filters: Grading Curves

The soil used in test series B (Table 2) was similar to that employed in series A and C. A grain size distribution curve for this material is also included in fig. 1 and basic test results for this soil are as follows:

Liquid limit	:	48%
Plastic limit	:	22%
Max. Dry Density	:	1.745 Mg/m^3
Optimum moisture content	:	19.5%

In order to conduct tests of manageable size it was decided to use scaled-down 'model' sub-base materials in the tests instead of the 2" and 4" single sized materials normally employed in full scale roads in Northern Ireland. A 20 mm single-sized crushed basalt stone was selected as a model sub-base material.

Both geotextile and granular layers were adopted as filters in the dynamic tests.

A range of nonwoven needle-punched polyester fabrics and woven polypropylene tape fabrics with basic properties as listed in Table 1 were used. The 95% pore size (O_{95}) values listed in Table 1 were obtained using a sieving method, ref (10).

In addition to the geotextiles two different granular materials were also used as filters. Grain size distribution curves for these materials are illustrated on fig. 1. Filter G1 was a crushed stone material with a maximum size of 3 mm and a coefficient of uniformity (U) of approximately 15. Filters G2 and G3 were formed using a natural wind blown sand obtained from a dunes area at Ballykinler, Co. Down. This material is highly uniform with a U value of 1.6, and was selected as a comparison with filter G1.

Table 1 : Filters tested in pumping model

Key : pp - polypropylene, e - estimated figure
figures in brackets eg (94x52) refer to no. of tapes/100 mm in warp x weft.

Filter Ref.	Filter Description	Mean Weight (gm/m^2)	Mean Thickness (mm)	Nominal O_{95} (U_m)
NW1	Needle-punched polyester (non-woven)	198	2.0	130
NW2	" " "	228	3.0	130
NW3	" " "	333	3.5	130
NW4	" " "	218	2.0	170
NW5	(2 layers of NW4)	436	4.0	170
W1	Woven pp tape (94x52)	120	0.3	300
W2	Woven pp tape (56x48)	255	0.6	430
W3	Woven pp tape (94x52) plus polyamide fleece	148	0.4	80
W4	woven pp tape (96x56)	107	0.3	60
W5	woven pp tape (56x36)	200	0.35	200
W6	woven pp tape (72x52)	140	0.32	140
W7	woven pp tape (60x24)	298	n/a	190
W8	woven pp tape (96x56)	174	n/a	100
G1	Stone dust	-	12.0	50e
G2	Ballykinler Sand	-	14.5	50e
G3	Ballykinler Sand	-	19.2	40e

3.2 Preparation and Testing of Filtered Subgrades

Samples for test were prepared by placing and compacting the subgrade soil into a circular mould, followed by placing of the filter-layer and the 20 mm model sub-base material.

Preparation of each sample was begun by placing the subgrade material in 4 layers in the bottom of a steel mould 355 mm in diameter. Each layer was initially compacted by tamping using a 2.5 Kg rammer conforming to BS1377 (11). Final compaction was achieved by applying a vertical static pressure of 500 kN/m^2 for a short period using a compression test machine. At this stage the volume of the subgrade soil was estimated and the weight of the subgrade and mould together was determined. Also a sample of the soil was taken for moisture content determination.

After compaction of the soil a mini-piezometer tip was inserted into the soil. The tip consisted of a saturated porous stone 3 mm deep, held by a rectangular brass support 12 mm x 9 mm x 3 mm (outside dimensions), and in which two small bore plastic tubes were located. For each sample the tip was placed with its top surface 3 mm below the top surface of the soil. This was achieved using a scraper and guide which enabled a 6 mm deep groove to be cut in the soil surface. After location of the tip, soil was carefully replaced around the tip and tubing and the subgrade surface was levelled. The pressure tubing and tip were then de-aired and connected to a water pressure transducer.

Once the subgrade was prepared a filter layer was positioned. For the geotextiles this was done by forming a sample approx 510 mm in diameter which, after weighing, was placed on the subgrade surface with the extraneous material turned up the sides of the mould. Geotextile W3 was positioned so that the needled fleece, which was attached to one face only, was in contact with the subgrade surface.

Each granular filter layer was formed by placing a known weight of the granular material on the surface of the soil. This was levelled and compacted under a static pressure of 500 kN/m^2. The thickness of the layer was then determined.

Schober and Tiendl (12) provide a theoretical method of obtaining a minimum poresize for granular materials which is dependent on grading and relative density. Estimates of minimum poresize, which can be considered closely equivalent to O_{95}, were made for the granular filters on the basis of their prepared dry densities and grading characteristics, and these figures are shown on Table 1.

Finally a known weight of air dried 20 mm single-sized aggregate was placed on the filter layer to model the road sub-base. The sub-base surface was carefully levelled to prevent possible rocking of the loading platen. To ensure simulation of ponded water, one litre of distilled water was poured into the mould. The sample (fig.2) was then left to stand for a period of 24 hours to ensure equilibration of porewater pressures caused by the preparation technique.

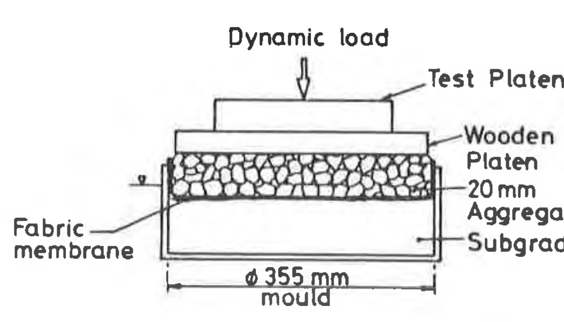

Fig. 2 Dynamic Loading Test with Membrane

After preparation as described above the test sample was transferred to the loading platform of an electronically controlled servo-hydraulic testing machine. This was used to apply a mean vertical pressure on the surface of the sample, which was varied sinusoidally at a rate of 5 Hz between 25 kN/m^2 and 75 kN/m^2 over a period of 24 hours after the technique of ref (8). During this period approximately 432,000 load cycles were applied to each sample.

Variations in excess porewater pressure were measured via the mini-piezometer and transducer. For the series A and B tests a penchart recorder was used for recording porewater pressures. Although this technique enabled the variation of mean porewater pressure with time to be monitored, it was not accurate enough for recording dynamic variations in porewater pressure response. For the series C tests ref (13) an ultraviolet recorder in conjunction with a cathode ray oscilloscope was used in order to more accurately monitor dynamic effects.

A comparative measurement of the amount of soil contamination occurring in each test was determined in the following manner. On completion of the dynamic loading test the sub-base layer and the geotextile or granular filter together with any contaminating fines were carefully removed from the mould. For the series A tests ref. (7) these materials were oven dried at 55^{o}C for 48 hours. A similar procedure was used for the series B tests, but for the series C tests it was decided to air-dry only. Periods of up to one week were found to be required to achieve this. Once dry the materials were weighed and the overall increase in dry weight, which was taken as the soil contamination value, in grams of dry weight of soil per square metre, was determined.

4. OBSERVATIONS AND DISCUSSION

4.1 Observations

A summary of the test results, including amounts of subgrade-derived comtamination passing the filter into the sub-base in terms of gms of dry soil per m^2 are listed in Table 2. The result for filter G2 is inconsistent with visual observations (see below) and is believed to have arisen from mismeasurement or from an unintended mixing of some subgrade material with the granular filter during removal of the latter from the mould after the test.

Table 2 : Summary of filter test results

m/c - mean moisture content
ρ_{dry} - mean dry density

Test Series	Filter Ref.	No. of Tests	Subgrade m/c (%)	Subgrade ρ_{dry} (Mg/m^3)	Mean Contamination (gm/m^2)
A	NW1	4	24.7	1.628	1290
A	NW2	4	24.6	1.614	1605
A	NW3	4	23.3	1.622	849
A	G1	4	23.1	1.644	410
A	W1	2	24.8	1.603	1922
A	W2	2	24.0	1.608	3766
A	W3	2	23.9	1.609	708
B	W4	3	24.1	1.619	1139
B	W5	3	24.2	1.625	1403
B	W6	4	24.0	1.624	2027
B	W7	1	21.1	N/A	1179
B	W8	1	21.6	N/A	737
C	NW4	2	23.6	1.620	777
C	NW5	3	22.4	1.656	839
C	G2	1	20.9	1.681	2854
C	G3	1	20.6	1.667	0

Although there are clearly several controlling variables, the influence of the O_{95} or effective opening size seems to be a major factor in controlling clay fines migration. Fig. 3 provides evidence for this which can be seen to be particularly strong in the case of woven fabrics, but is also apparent when comparing all of the filter types.

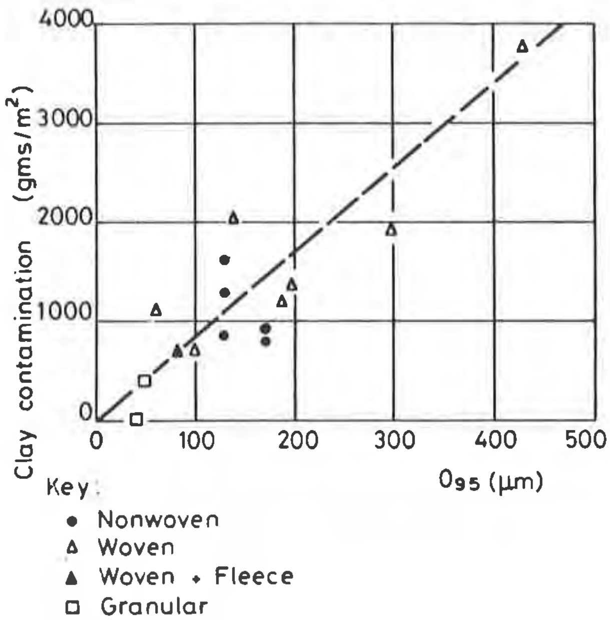

Fig. 3 Influence of O_{95} on contamination

The importance of some other factors was also revealed from the tests. Fig. 4 indicates a clear dependence of contamination value on initial subgrade moisture content for nonwoven filters. This effect was also noted for woven geotextiles ref (7) although it was less clearly defined. The moisture content range for the tests is on the wet side of optimum and the increased contamination may well arise from reduced soil undrained shear strengths.

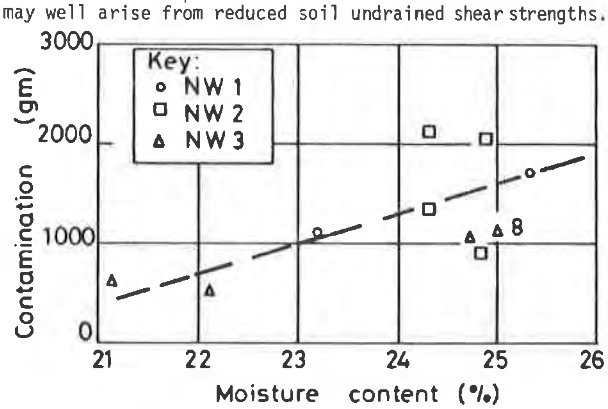

Fig. 4 Influence of subgrade moisture content on contamination

During testing of filter W2 some problems were encountered in control of the dynamic load. This occurred in three tests which have therefore not been included in Table 2. However the results of these tests provide some evidence, listed in Table 3 below, that fluctuations in dynamic load have a significant influence on the contamination value.

Table 3 : Test results for filter W2

m/c - mean moisture content
ρ_{dry} - mean dry density

No. of Tests	Subgrade m/c (%)	Subgrade ρ_{dry} (mg/m^3)	Mean contamination (after 24 hrs) (gm/m^2)	Dynamic Load Variation
2	24.0	1.608	3766	25-75 kN/m^2
2	25.0	1.604	6351	initially 25-75 kN/m^2 approx (0-12 hours), rising to 10-90 kN/m^2 approx (12-24 hours)
1	24.6	1.622	2526	initially 25-75 kN/m^2 approx (0-12 hours) falling to 50 kN/m^2 static approx (12-24 hours)

Measurements of porewater pressure during the tests indicated an initial sharp rise in excess porewater pressure ranging from 16 kN/m^2 to 88 kN/m^2. Besides possible errors in measurement such a range is considered to be due to variations in the stress distribution under the loading platen, and particularly also to the location of the mini-piezometer relative to the nearest point on the subgrade surface upon which a sub-base stone was bearing.

For all of the tests the initial excess porewater pressure dissipated as the test proceeded, but at very differing rates. Measurements of dynamic porewater pressure variation, recorded for the series C tests indicate that the dynamic component of porewater pressure varied from approx 2.5 kN/m^2 initially to approx 1.5 kN/m^2 towards the end of the test. The dynamic component varied, as would be expected, at a rate of 5 Hz.

Fig. 5 shows how the rate of porewater pressure dissipation, expressed as the time for dissipation of the initial excess porewater pressure seems to influence the contamination value. Apart from filter G3, the data for the figure is derived from the series A tests and mean values have been used in an attempt to clearly show the possible trends.

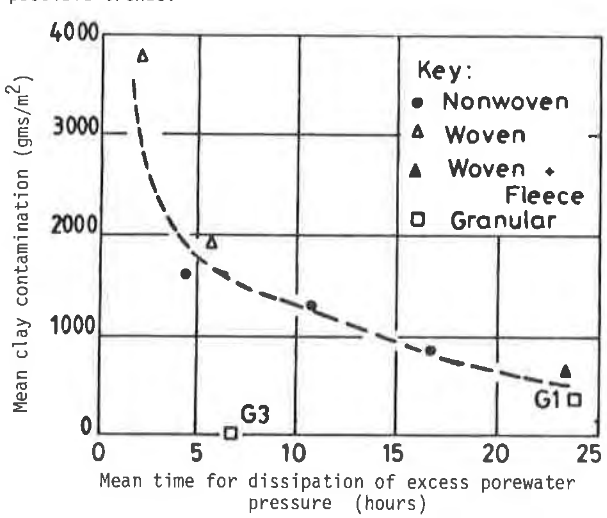

Fig. 5

It can be seen that in general high dissipation rates are associated with high amounts of contamination, and low dissipation rates with low amounts of contamination. The one clear exception is for the granular filter G3 as seen on the figure.

During post-test measurements a number of useful visual observations were made. It was seen that for all tests in which granular layers were used as filters the model sub-base removed from the mould was devoid of contamination. However for tests in which geotextiles were used, small amounts of clay slurry was found adhering to the sub-base materials. With the geotextiles, and this was particularly evident for the nonwovens, quantities of clay fines were seen to be held within the filter layer.

The appearance of the exposed post-test surface of the subgrade was different after granular filter tests from that revealed after geotextile filter tests. A level, even surface with an apparently uniform distribution of grain sizes was observed after granular filter tests. In contrast, after geotextile tests, the subgrade surface was found to be uneven with depressions up to 8 mm deep showing where individual sub-base particles had locally deformed the subgrade. In addition coarser subgrade particles were visible in these depressions, suggesting that fines migration had taken place beneath the high stress contact points. This suggestion was confirmed on post-test examination. Mud staining was clearly seen on the geotextiles at locations corresponding to the contact areas between sub-base particles, filter and subgrade. Between the contact areas little or no staining was visible. These observations were less apparent with the thicker non-woven filter layers.

4.2 Discussion

The problem of prevention of clay fines migrating from a cohesive subgrade into a granular sub-base differs from many other filtration problems in several respects. Firstly the subgrade, sub-base and the intervening filter layers are subjected to dynamically varying normal and shear stresses. Secondly, the flow through the filter may be of the reversing type. Thirdly the filter is essentially required to prevent the passage of what is believed to be a clay slurry and yet allow the free passage of water.

For these reasons it is unlikely that any economic filter method will completely prevent fines contamination. Tests reported elsewhere (3,4) show that relatively small amounts of contamination can appreciably reduce permeability and shear strength, and thus the object in using a filter beneath sub-bases must be to limit fines migration to an acceptable level.

The tests reported in this paper were run only for 24 hours, yet significant contamination occurred. Unfortunately data from longer term tests is not yet available. However Eniolorunda (14) has done similar tests which show only little evidence of a fall in the time rate of contamination for periods of up to 96 hours. It is estimated that for the 24 hour test contamination amounting to more than 500 gm/m^2 would be unacceptable.

Figs. 3 and 5 indicate that no geotextile filter achieved this amount of protection, and only the granular filters, notably G3, performed inside the limit. Nevertheless a number of useful trends have been established.

For woven and thin non-woven filters (i.e., $\not>$ 200 gm/m^2 or 2 mm in thickness) behaviour is apparently similar. These filters have poresize and permeability properties which will not alter by large amounts upon application of normal loads, i.e. they can be classed as relatively incompressible. Their permeability will be related to their poresize characteristics, and their performance would appear to depend upon O_{95} (fig. 3). For filters with large O_{95} values, contamination was large and porewater pressure dissipation was rapid. The latter is believed to be due not only to the relatively high geotextile permeability but also to loss of fines from near the subgrade surface this enhancing the drainage properties of this layer. As previously noted, areas beneath individual sub-base stones indicated visual evidence of fines loss.

With thick non-woven filters, which can be classed as relatively compressible filters, there are thought to be three effects on filtration besides that of the laboratory determined O_{95} value. Firstly, the thickness of the filter can assist in making the pressure applied to the subgrade more even than otherwise would be the case i.e., a load spreading ability is possible. Secondly the nominal effective opening size O_{95} may become locally reduced, beneath individual sub-base particles, due to the compressible nature of the filter. Associated with this would be a third influence of three dimensional filtration. Although such effects might be expected to assist in reducing fines migration the test results are not clear on the influence of nonwoven thickness on performance (see table 2), and further data is required to clarify this effect.

The granular filters also are quite thick, but can be classed as relatively incompressible. Hence their properties are unlikely to change much during loading. Also their loadspreading capabilities can be relied on. Poresize seems to be relevant with these, the best performers, having estimated O_{95} values smaller than any of the fabrics. Although only one test has been run using G3, results are included because this filter appears to have prevented any contamination, and yet initial excess porewater pressures dissipated in a relatively short time (figs. 3,5). It is of interest that the coefficient of permeability, K, of this filter estimated using the well-known expression $K=100D_{10}^2$ (eg ref. (15)) where D_{10} is the 10% passing particle size is approximately 80% higher than that of filter G1.

It is thought that this filter worked well because of a combination of large thickness, acceptable permeability and low poresize. There was no evidence of substantial softening or slurry formation at the subgrade surface beneath this filter.

It is of interest that the granular filters and the geotextile W3, which performed quite well, conform to well known piping criteria for granular and geotextile filters. The criteria Cedergren (15), are as follows:-

$$\text{granular: } \frac{D_{15}\ (\text{filter})}{D_{85}\ (\text{soil})} \leqslant 5$$

$$\text{geotextile: } \frac{O_{85}\ (\text{filter})}{D_{85}\ (\text{soil})} \leqslant 1$$

5. CONCLUSIONS

Laboratory investigations of model sub-base protection using both granular and geotextile filters have been conducted. Although a number of factors such as test duration and sub-base grading have not yet been examined, several relevant findings have emerged. For granular and geotextile filters, contamination seems to depend on effective opening size or its equivalent. There is strong evidence particularly from the granular filters that the mechanical properties of the filter may also be important. Measurement of porewater pressure dissipation times in the subgrade showed that these periods were rather long for the better filters. However a thick, uniform, granular filter, not only prevented contamination but allowed relatively rapid dissipation of excess porewater pressures.

Accordingly it would appear that a successful geotextile filter, for the application examined in this paper should have good loadspreading ability, acceptable cross-plane permeability and a low pore-size range. It seems possible that the piping criterion for filter fabrics suggested by Cedergren (15) may be helpful in respect of the latter. Thick relatively incompressible geotextiles with suitable filter properties can be manufactured using composite construction. Several filters of this type are currently being examined at the Queen's University of Belfast.

6. ACKNOWLEDGEMENTS

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Separation Function and Bearing Capacity of Non-Woven Fabrics in Special Geotechnical Practice

La fonction de separation et charge portante de géotextiles en génie civil

The report gives four examples on the application of non woven fabrics in civil engineering practice; predominantly the subsoil is consisting of water saturated and very compressible fines. 1. Pile foundations: In peats and soils of a soft plastic to liquid consistency constriction or uncontrolled flow out to the sides of the fresh concrete into the soil can be avoided by wrapping a stretchable non woven fabric around the reinforcement cage of the bored pile. This method has also proved successful in subsoils with cavities and for reducing negative skin friction. 2. Vertical drains for the acceleration of consolidation settlements, consisting of non woven tubes filled with sand. 3. Foundation of metal culverts on soft grounds: a carefully compacted encasement can be gained by using geotextiles. 4. Railway construction: by means of non woven fabrics the "pumping up" of fine particles from the subsoil into the stony ballast (caused by dynamic loads) is prevented.

Ce rapport nous donne 4 exemples d/utilisation de nappe, tires de la pratique des travaux du genie civil. Le sous sol est compose avant tout de granules finis, satures d/eau et a forte compressibilite. 1. Fondations a l'aide de pieux: dans les sols maus au tourbeux, en entaurant l'armature du pilier par une nappe flexible. On peut eviter, lors du versement du beton qu'il y ait un resserement ou des fuites incontrolees dans le sol. Cette methode au aussi fait ses preuves pour des sols a cavites et permet une diminution des forces de frottement negative. 2. Drainages verticaux servant a l'acceleration du tassement, ce sont des tuywaux nappes remplis de sable. 3. Fondation pour les "kulverts" metalliques: sur des sols mous l'utilisation de geotextiles permet une solide insertion grace a un excellent tassement dans les sols mous. 4. Construction de voies ferrees: la nappe empeche la migration de particules fines provenant du sol et leur penetration dans le ballast (migration provoquee par des charges dynamiques).

1 PILE FOUNDATIONS

The use of non-woven fabrics for bored (or driven) piles of cast-in situ concrete represents an Austrian innovation. In peats and silts of a soft plastic to liquid consistency, constriction or uncontrolled flow out to the sides of the fresh concrete into the soil has to be avoided. This can be achieved by wrapping a stretchable non-woven fabric around the reinforcement cage (Fig. 1). The hydraulic pressure of the new concrete presses the non-woven fabric outward, when the pipe (casing) is tensioned, thus guaranteeing sufficient cover over the steel. Fig. 2 shows such a pile encasement in critical soil layers.

This method has also proved successful recently in loess areas, in the spanning of former cellars, century old underground escape corridors, old hollows etc. Fig. 3 shows a section of a bored pile in an ancient cellar passage. Non-woven fabrics are also being used, instead of pretensioned metal, in built-up urban areas, where one can expect various underground cavities.

Furthermore, such an encasement with a non-woven fabric can clearly reduce the negative skin friction of piles (for example in high bridge abutments), resulting from the fill, embankment resp.

Special care has to be taken in the welding of the fabric overlap. In large-scale 1:1 trial tests, unjoined seams partly opened up. Furthermore it is necessary for the fabric encasement to be sufficiently embedded (approximately $\geq$ 1 m) into the layer of soil, which occur below the cavities (cellars etc.) or under soft plastic overburden (top covering). Otherwise, as experience has shown, there is the danger of the non-woven fabric being lifted to the top layers during the tensioning process of the casing (pipe), and thus permitting the new concrete to flow out to the sides.

2 VERTICAL DRAINS

Vertical drains accelerate primary consolidation of soft, highly water saturated soils. As they don't improve secondary creeping they are hardly suitable for peats but well for weak clays, silts and organically contaminated soils. Generally they are used in common with a temporary overburden of the foundation area, embankment etc.

Among the various techniques the non-woven fabrics drains, developed in Austria, represent an innovation: They look like long hoses being filled with sand and lowered into vertical holes (Fig. 4). The usual diameter is 10 cm, the width of the grid approximately 2 - 4 m, according to the kind of soil. The hole is drilled

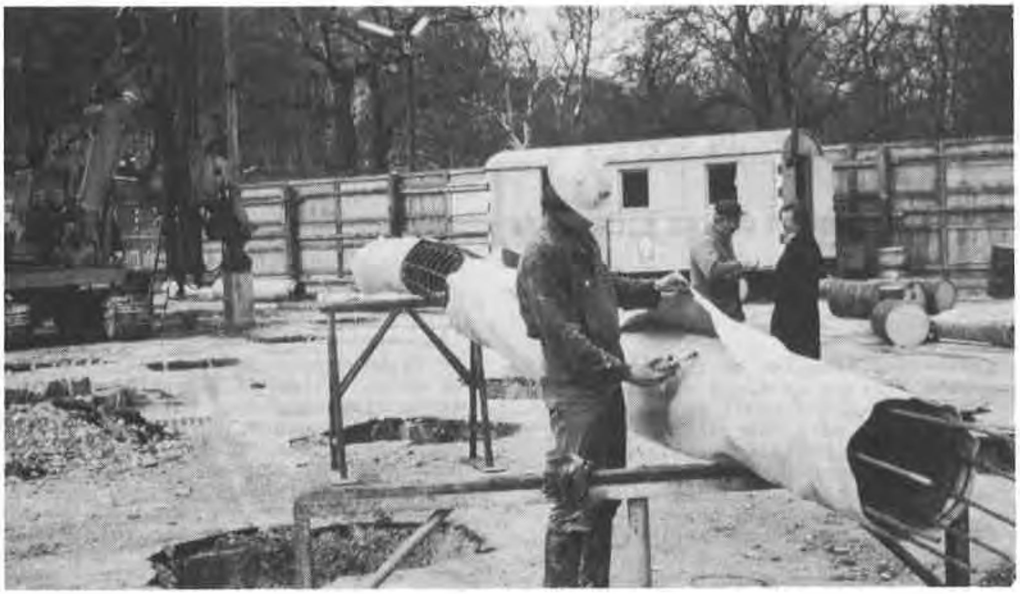

Fig. 1 Encasement of the reinforcement cage of a bored pile with a non-woven geotextile; welding of the longitudinal overlap with a blow torch.

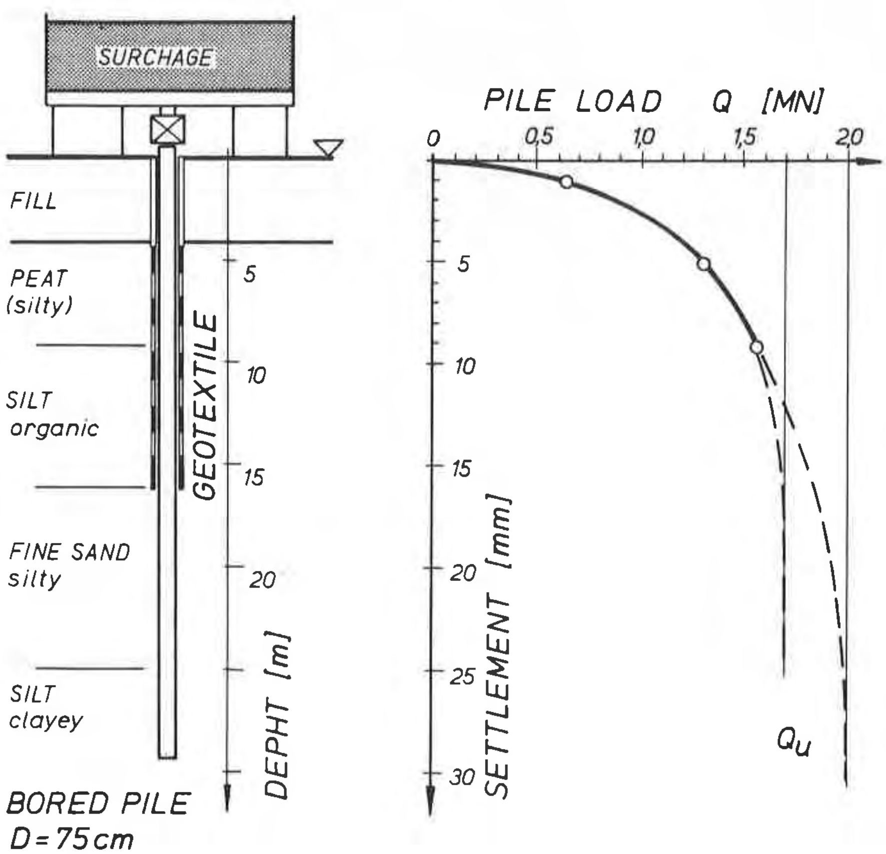

Fig. 2
Load test on bored pile; reinforcement wrapped with stretchable non-woven geotextile within the peats and "liquid" silts.

Fig. 3 Bored pile (Ø 90 cm) with the reinforcement encased with a non-woven geotextile, during the tensioning of the pipe encasing in an old cellar passage (loess-subsoil).

Fig. 4 Lowering of a vertical non-woven drain (filled with 2/7 mm - sand) into the borehole.

by circulation methods to avoid pollution of the borehole shaft, or by driving method. As filter sand the grain size of 2 - 7 mm is prefered. Installation of a drain with 10 m length is performed within some minutes. According to their greater inflow cross section the effectiveness of these drains surpasses that of narrow strips or wicks; their mutual distance is of greater influence than an alteration of their diameter within few centimeters.

3 SEWER PIPES AND CULVERTS ON SOFT SUBSOIL

3.1 Sewer pipes

Pipes bedded into the subsoil can withstand stresses of static and dynamic load better, the better they are imbedded. The most unfavourable conditions occur (maximum bending moments in the pipe), if the pipe is not supported on the sides, but only rests on a linear line. In soft and inhomogenious subsoil, there is the additional danger of the pipe resting on uneven and yielding ground, which can cause localized excessive stresses, especially in the connection sockets.

If a non-woven fabric is laid onto the soft subsoil, the base of the pipe trench can be covered with sandy gravel, which can be sufficiently compacted. Thus the bearing capacity of the pipe is increased considerably and differential settlements are reduced.

3.2 Metal culverts

These flexible culverts are able to absorb very high radial pressures without any significant deformation, if the allround - embedding is carefully compacted (Fig. 5). Moreover they are relatively resistant to differential settlements.

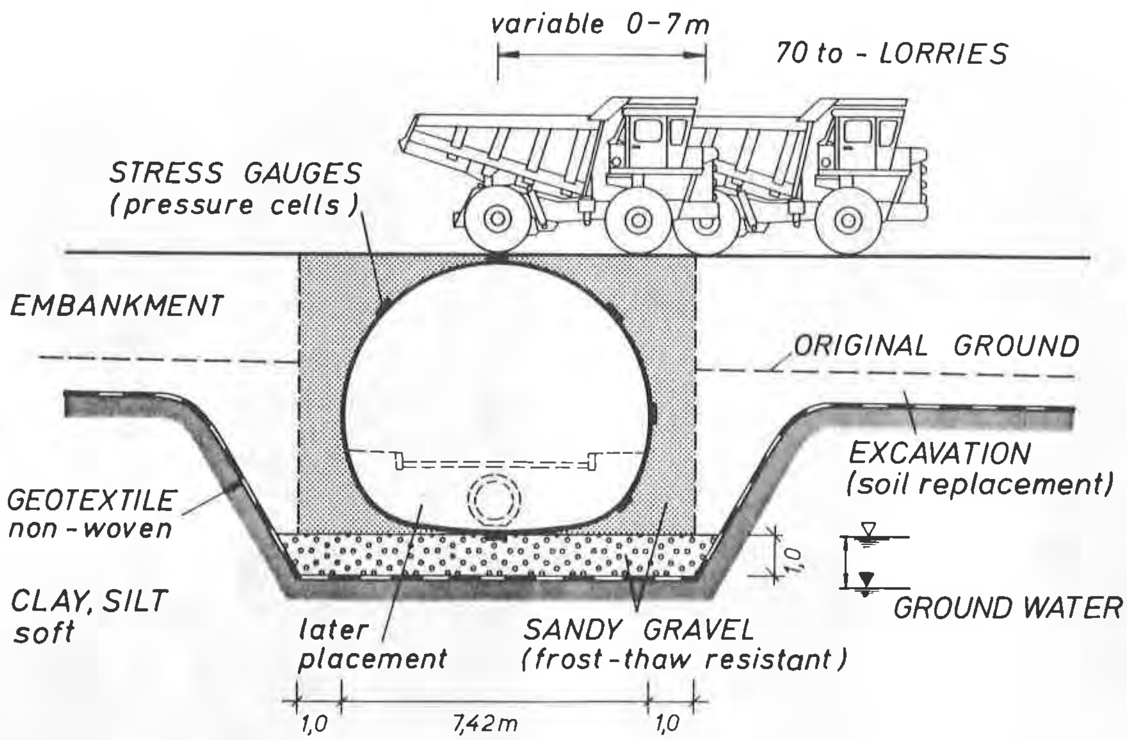

Fig. 5 Steel culvert (length l = 40 m) onto soft subsoil and geotextile: load tests with 70 to - trucks (one lorry behind another).

If the subsoil conditions are poor, non-woven fabrics can be used instead of deep reaching soil replacements. These geotextiles enable a sufficient compaction of the base bedding and a good drainage of the water saturated subsoil during consolidation. With diameters of 4 - 8 m flexible culverts have proved their value instead of rigid bridges of reinforced concrete founded on piles: The advantages are lower cost, shorter construction period and reduced differential settlements between embankment and culvert. Fig. 5 shows 1:1 tests which have been performed with culverts being covered on their peak only with 10 - 60 cm sandy gravel. The deformations were max. 5 cm and reversible.

The great deformability of the flexible pipes and culverts is of advantage for heterogeneous soil conditions and especially for embankments on soft subsoil. In such a case the culvert is placed upon an only thin replaced layer (on geotextiles) ahead of filling the dam. According to the settlements expected by calculations the culvert is placed superelevated as well as cambered with a pitch of top up to the range of dm. Fig. 6 shows such an example of the construction of a motorway on peats. After consolidation of the subsoil this culvert (and neighbouring objects too) is now lying horizontally.

4 RAILWAY CONSTRUCTION

Due to the heavy dynamic load resulting from train traffic, the stony ballast is very often contaminated by rising fine particles from the underlying soil. This "pumping up" not only leads to a deformation of the ground, but also to greater susceptibility to freezing and thawing (in the ballast) and, eventually, causes a gradual decrease in the bearing capacity of the upper structure. Graduated sandy - gravel filters and synthetic non-woven fabrics can be used as separating layers (Fig. 7). Rigid soil stabilizers (cement, but lime too etc.) have proved as not very satisfactory and there is also uncertainty about the long-term effectiveness of bituminous sealing of the track bed.

Non-woven fabrics with a low stretch property should always be covered with a thin protective layer of coarse sands, in order to prevent performation by the sharp stones of the ballast (Fig. 7). This danger especially exists during laying (particularly with tamping machines), when the subsoil is not homogeneous. It also exists later on caused by constant dynamic load. Finally, the coarse sand should protect the non-woven fabric against damage caused by the chains of the machines, which clean the ballast track bed (after years of surface pollution and long term abrasion or crushing of the ballast stones).

According to the experience gained so far, it is not to be expected that fine parti-

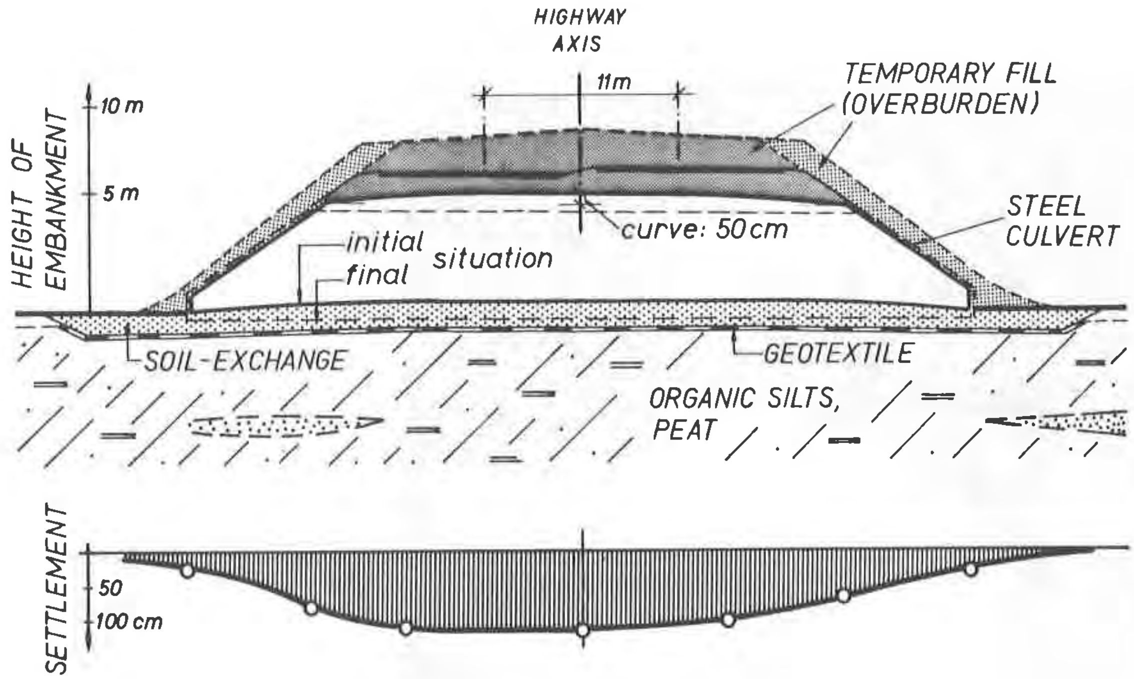

Fig. 6 Flexible steel culvert (d = 7 m) on soft subsoil: originally placed in an superelevated and cambered position with a 50 cm - pitch of top - according to expected settlements; temporary overfill of the embankment (for settlement acceleration).

cles will continue to rise from the subsoil, through the non-woven fabric, into the ballast, when "filterstable" non-woven fabrics are used, since these must stand up even to the stresses of turbulance in hydraulic structures. The flow of the finest grain quickly tries to achieve a dynamic equilibrium. On the other hand, their function as a separation of layers between upper structures and substruotures is completely maintained, even if the fibre tangles are clogged. For investigating these problems a special dynamic load test has been developed in Austria (Fig. 8).

Longterm observations in the field (Fig. 9) have shown that neither staple fibre non-woven fabrics nor thin rigid non-woven fabrics are suitable, in the long run, for railway construction: Staple fibre non-woven fabrics because of material fatigue (of the chemical bonding) and thin rigid non-woven fabrics because of the likelihood of mechanical damage.

The long-term tests on a railway trail section have been running since August 1973, different geotextiles being investigated. It was found that only geotextiles made of endless fibres which are only mechanically bonded could fulfil the expectations. Thermically bonded materials have been destroyed because of their poor adaptability and the binding agent of chemically bonded material failed under the longterm stress. The result of a further test section with styrofoam slabs instead of geotextiles was also not satisfying. Experience has shown generally that damage only occurs after a certain number of load cycles.

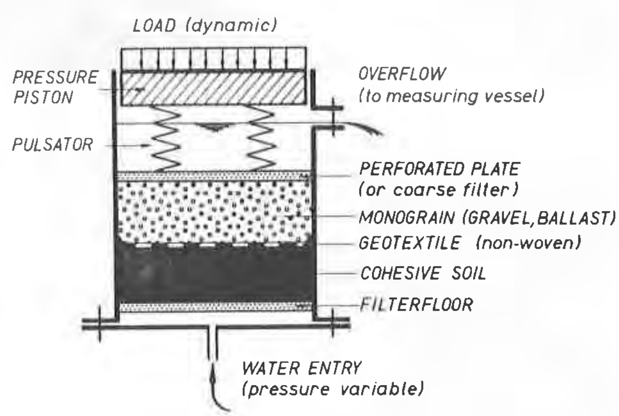

Fig. 8 Equipment for testing the filter-effectiveness of non-woven fabrics towards dynamic loads (mainly for railways).

5 CONCLUSION

In chapter 1 to 4 some specific examples were taken out of the great many possible applications of fabrics in soils engineering. Whereas for pile foundations and vertical drains the separating function is in the fore, the bearing capacity and deformability of geotextiles also are of great importance for the foundation of culverts and railways on soft subsoil.

Fig. 7
Railway construction: non-woven geotextile covered with frost-resistant sandy-gravel protective layer.

Fig. 9
Non-woven fabric uncovered after 7 years under railway - running (a protective layer of 7 cm sand had been laid between geotextile and sharp-edged stonyballast).

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Laboratory and Field Work Relating to the Use of Geotextiles in Arid Regions

Essais de laboratoire et experience de chantier sur l'utilisation des géotextiles en régions arides

This paper briefly reviews applications of geotextiles in arid regions and is mainly concerned with the use of geotextiles in inhibiting salt and moisture movement in the capillary fringe. Capillary breaks of gravel often fail because their pores become filled with fine particles or because salt precipitation causes the gravel to disintegrate. The use of single layers of non-woven fabric reduces the impact of the former difficulty while multilayer fabrics permit both filtration and drainage and can successfully overcome both problems. Details are given here of experiments carried out to monitor the influence of single and triple layered fabrics on salt and water movement in the capillary fringe. The soil beds used were up to a metre thick and the experiments show that the rise of both salt and moisture is prevented by the composite fabrics but not by single layered materials. Lateral movement of moisture above open-sided fabric structures is very slow. The results allow recommendations to be made on the positioning of the fabrics in the capillary fringe and the experiments have been followed up by field trials which are briefly described here.

Cet article resume les applications des geotextiles dans les regions arides et traite plus particulierement de leur utilisation pour eviter les remontees de sel et d'eau au niveau de la frange capillaire. Les couches anti-capillaires en grave peuvent devenir inefficaces par colmatage du a des particules fines ou parce que la precipitation de sel provoque la desintegration de la grave. L'utilisation d'une seule couche de geotextile non tisse resoud le premier probleme. Des multicouches de geotextile, assurant a la fois la filtration et le drainage, peuvent resoudre ces deux problemes et eviter l'utilisation d'une couche de grave. Des details sont donnes sur la experiences realisees pour controler l'influence des geotextiles dans ce genre d'utilisation celles-ci montrent que les remontees de sel et d'eau peuvent etre evitees par des geotextiles multicouche et non pas, par un simple geotextile. L'evacuation laterale de l'eau par des structures ouvertes realisees en geotextiles multicouches et tres lente. Des recommandations sont donnees pour determiner le niveau de ces geotextiles dans le frange capillaire. Les experiences ont ete completees par des essais in situ.

1. INTRODUCTION

The use of geotextiles originated in situations involving water, either in the form of soft soils with a high moisture content, or in the protection of slopes from hydraulic erosion by rivers or seas. Consequently the rapid growth in this use throughout the world might have been expected to avoid the arid regions apart from dock and coastal protection works. However, there has been a wide application of geotextiles in areas without significant surface water (1). This paper outlines some of these applications and the related laboratory development work.

2. RANGE OF APPLICATIONS IN ARID AREAS

A. Need for Geotextiles

The primary function of geotextiles is usually filtration, fabric filters being easier than rock filters to install reliably from the technical point of view. In the countries surrounding the Arabian Gulf, cost has also been a significant factor, as rock strong enough to be durable in a filter is extremely scarce and consequently expensive. This naturally stimulated the early adoption of geotextile filters in many conventional uses in coastal works.

Rapid expansion of construction in the Gulf countries has led to the development of special engineering techniques to cope with agressive chemical attacks from the groundwater. The geotextile technology being used in coastal works was adapted to deal with these new problems.

B. Examples of applications

(i) Irrigation - Drainage Filter protection of the drainage media on Lower Khalis Irrigation scheme, Iraq (2). The drainage was installed to enable the salt, which had built up in the ground over centuries of watering, using river water, to be leached out using modern methods of irrigation (figs. 1 and 2).

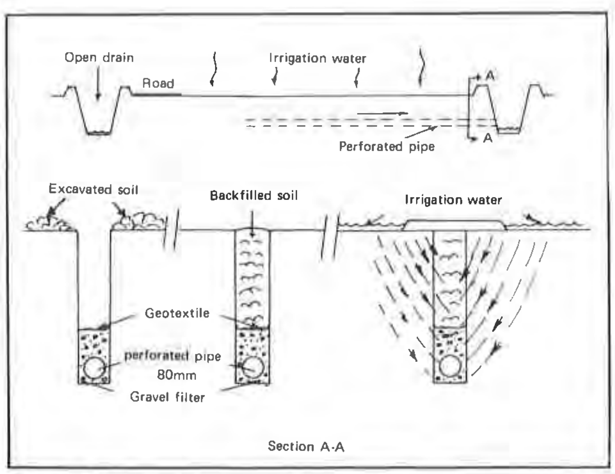

Figure 1 Idealised section - Lower Khalis irrigation/drainage

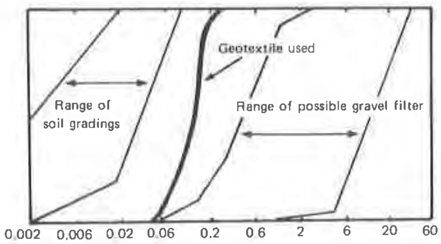

Figure 2 Lower Khalis - grading of filters after Bailey and Rycroft (1978).

The filter design was based on the modern approach of Winger and Ryan's procedure involving coarse gravels, which need to be protected from infiltration of adjacent fine soils. The geotextile combines this protection with high drainage efficiency. A similar re-vitalization of valuable agricultural land which has become salt-laden through drainage problems is being undertaken in Egypt. A 100 mm diameter geotextile tube filled with single sized stone has been used on a small scale on farmland near Tanta. A large scale trial is now in progress at Zawia on the reclamation of virgin salt marsh where several field drain systems are being compared. The Hydraulic Research Station in England is assisting with this evaluation.

Egypt: Zawia, 100 mm geotextile tube drain.

(ii) Irrigation - Supply Canals These are often concrete lined and used only during the irrigation season. Consequently uplift pressures when empty necessitate a reliably filtered drain which will remain efficient over a long working life without maintenance. Large scale usage has been on the Lower Khalis scheme in Iraq and the Jizan and Jubail canal systems in Saudi Arabia, the last employing Filtram in a drainage mode. A particular application of this principle was the protection of the drains below the dry docks at Dubai. For this project the vital requirements of long term filtration efficiency was met by extensive testing carried out with in situ soil and various geotextiles prior to final selection of the latter (fig. 3).

(iii) Road Construction A completely different application of geotextiles has been made by the Public works department in Kuwait where they have been used to reinforce the bitumen resurfacing of the Kuwait section of the Basrah road. This is undergoing reconstruction to withstand the much increased

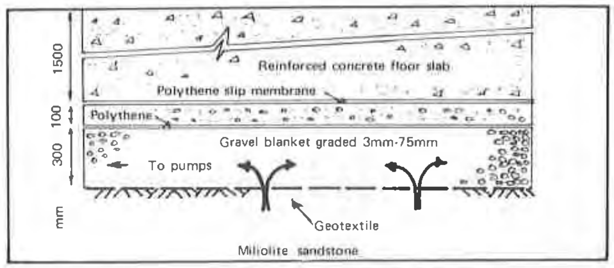

Figure 3 Idealised section - Dubai Dry Dock.

heavy truck traffic. The purpose of the geotextile is to inhibit reflective cracking in the new surface arising from existing cracks in the road. Prior to this work, sections of the road in the city of Kuwait were used to evaluate the benefit of geotextiles in this inhibition of cracking in trials lasting several years. The high standard of site supervision required to monitor correct tack coat application is of course also necessary in arid areas. The high ambient temperatures facilitate the tack coating and the generally low frost incidence means that the temperature range is not excessive, which favours the long term effectiveness of this method of road maintenance.

(iv) Roof Insulation The 'upside down' roof concept familiar in Europe and North America for retaining heat below roofs has been transferred to hot countries to keep the heat above roof level. The techniques employed are generally much alike with several proprietory insulating blocks being available. Geotextiles are used in two ways; firstly applied over concrete roofs to protect the overlying waterproof membrane from puncture by roughess of the concrete finish, and secondly, laid over the top of the insulating blocks to protect the joints from incursion of dust and larger particles from the overlying paving stones or garden area. This in turn protects the non-mineral elements of the roof from the Sun's effects.

(v) Capillary Breaks The rapid increase in cultivation of land adjoining the Gulf, where the saline water-table is close to the surface has led to the provision of long-lasting capillary breaks to prevent the rise of the saline water to the surface. If such breaks are not provided the high rate of evaporation causes concentration of salts in the soil near the surface with highly deleterious effects on all but the most hardy types of vegetation. The capillary breaks have traditionally been formed by installing a layer of single sized rock without material finer than 0.1 mm. A filter must be provided above this layer to prevent downwash of fine material by irrigation and rain-water, and frequently a filter is also placed below the break

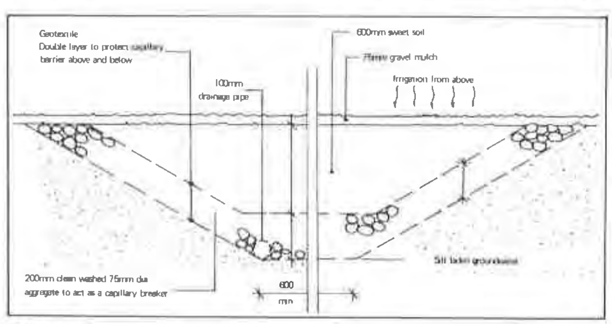

Figure 4 Typical Section, geotextile protected capillary break (after the Landscape Design Group).

in sabkha-type soils. Geotextiles are the most reliable way of ensuring that such filters are provided and their use in this application has developed strongly in the last few years. The planned layout of many cities incorporates landscaping and tree provision with the use of geotextiles as part of the capillary break. The Dubai Petroleum Company headquarters was an early example of such an application where the fabrics have been successfully protecting the capillary break since 1977. Cultivation has been facilitated on the Sabkhas near the Red Sea and the Gulf Coasts of Saudi Arabia by similar means. A recent example of such work is in the grounds of the Meridien Hotel at Alkhobar, Saudi Arabia.

Saudi Arabia: Al Khobar, geotextile capillary break Meridien Hotel grounds.

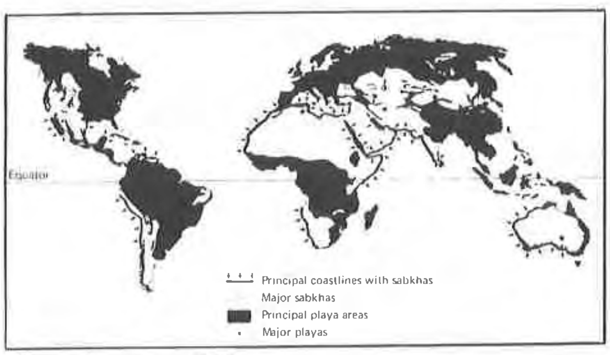

Figure 5 The distribution of arid regions and Sabkha-type coastlines with highly saline groundwater (after Cooke 1981).

3. EXPERIMENTAL WORK ON THE CONTROL OF SALT AND MOISTURE MIGRATION

A. Introduction - Occurrence of saline groundwater and salt precipitation

Saline groundwater occurs throughout the worlds arid regions (figs. 5 and 6). In these regions, where the capillary fringe reaches the ground surface, there is usually precipitation of sodium chloride and in some instances substantial layers of gypsum may develop. Infiltration of rainwater may temporarily remove soluble salts, but these can reappear within hours or days - brought to the surface by evaporation and the capillary transfer of moisture. Gypsum may grow over a period of weeks or months and tends to

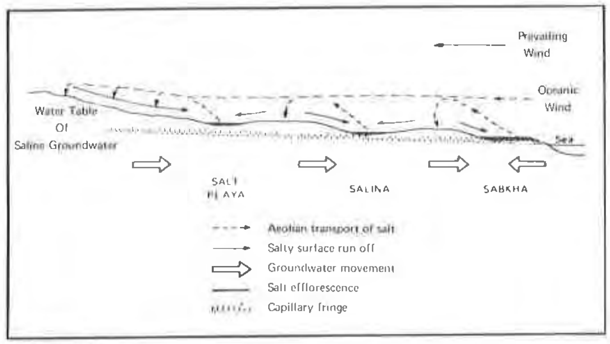

Figure 6 Cross sections of typical sabkha deposits showing the salt cycle (after Cooke 1981).

crystallize in a dryer environment than does halite. Soil or sand placed on the salty surface rapidly becomes saline by capillary transfer and it has been found, in the Middle East for example, that halite and gypsum crystallize in 'fill' placed in areas where the original capillary fringe reached the original ground surface

B. Implications of groundwater salinity

Elevation of the levels of sulphate and chloride clearly has serious implications for foundation stability, the stability of fill and for vegetation set in the area. Experiments have been carried out in which soluble ions have been transferred through rocks for vertical distances of more than a metre by capillary pressures. Providing these rocks are not exposed to the atmosphere, no damage is done, but wherever contact with the atmosphere is made, the rocks crack and disintegrate. Piles of aggregate or single pieces of rock (or concrete) standing on a saline soil bed may also be decomposed readily by precipitation of phases such as halite and gypsum (4 and 6). This phenomenon is seen commonly in the field as 'salt weathering' of rocky eminences standing above the zone of capillary suction. Capillary breaks made of gravel may also suffer the same kind of deterioration by precipitation of salts and it has been recorded that roads in arid regions are often damaged by salt precipitation either within the road base or in the aggregates of the surface layer (5). Thus for many applications the development of a fabric that can be used as a capillary break is highly desirable. 'Filtram' is one such material and it is this fabric that we have tested most thoroughly under laboratory conditions to define its ability to prevent salt as well as moisture movement.

C. Experiments on the control of salt and moisture distribution

Two series of experimental systems have been set up. The first has involved soils up to 550 mm thick and in the second series the beds are up to 900 mm thick. The small experiments give useful results within months, but the larger soil beds require some 30 months to become stable and are long term experiments that are expected to last for several years.

In most of the experimental systems natural seawater has been fed into the base of the soil tanks to provide a constant level water table several centimetres above the base of the tanks (figs. 7 & 8). Layers of gypsum have been placed in some soil beds

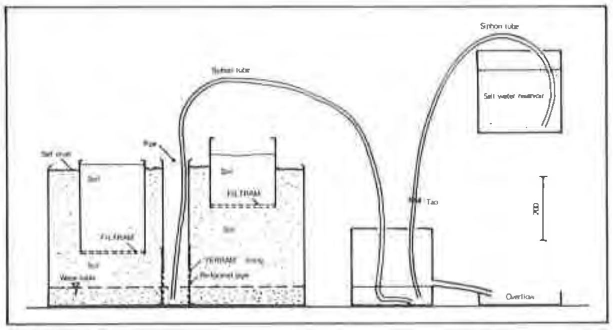

Figure 7 Design of smaller experimental soil beds. These beds are approximately 500 mm thick. The fabrics were placed at various levels in the soils and cemented to vertical impermeable frames. In some tanks the soils contained carbonate as well as the siliceous soil, and in some tanks solid NaCl and Gypsum were added as layers in the soil. In one experiment a layer of gravel was placed beneath the fabric.

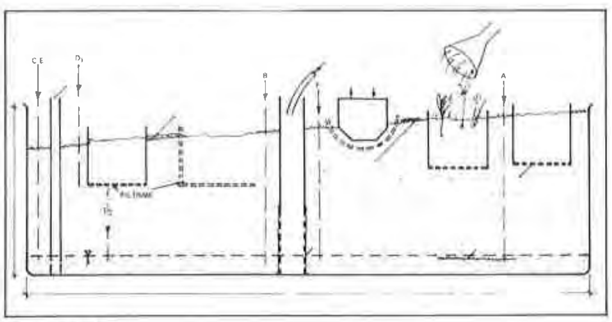

Figure 8 Design of the larger experimental soil beds. The fabric is shown as a broken line. The positions A to E are the locations of vertical cores taken for chemical analysis and the results of this analysis are given in figure 11. The diagram also shows in sketch form the design of some of the individual experiments.

but generally the soils used are granular quartz silt and sand and mixtures of microporous carbonate and quartz. Fabrics have been placed in the soil beds at various depths and distances from the water table. Some have been loaded and others covered with horticultural soil or with impermeable surface layers. Experiments have also been set up to study the lateral movement of salts and moisture above horizontal sheets of Filtram placed in the soil. Early results of these experiments have been described previously (3), and the paragraphs that follow provide more recent data obtained from the larger experiments. The essence of the results is that the fabric makes an effective barrier preventing the upward and lateral migration of salt and moisture that may remain useful for at least a period of years even where surface infiltration of moisture occurs. Detailed results are given in figures 9 and 10 and can be summarized as follows:-

(i) Establishment of salt and moisture profiles Sea-water has been added to each soil system via a central pipe (fig. 8) and in the vicinity of the pipe a salt and moisture profile becomes stable within several months in the larger soil beds. However, it appears to take very much longer for the profile to become

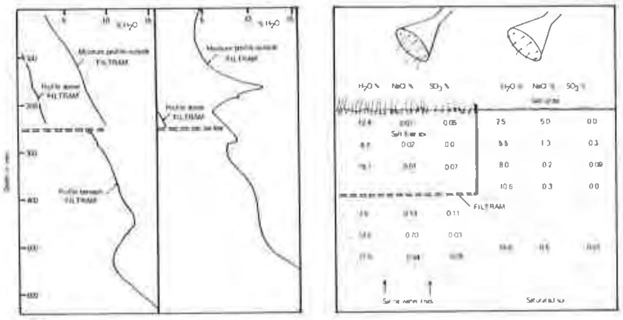

Figure 9 The left figure provides two moisture profiles through the soil beds of the larger experiments. The profiles are taken from above, alongside and beneath the Filtram. It should be noted that these results are for experiments in which the seal between the fabric and the impermeable frame was broken. They therefore represent the worst rise of moisture found in the experiments.
The right hand diagram shows the typical results for the salt and moisture concentrations above, below and to the side of layers of Filtram. The salt levels found above the fabric are those of the original soil.

established at a distance from the feed pipe. The profile eventually produced is illustrated in figure 10, which also shows a typical corresponding profile where a layer of Filtram is placed in the soil bed. Halite precipitation occurs at or near the surface and gypsum is precipitated in very much smaller amounts than the halite and for depths of several centimetres. The experiments show that thin layers of gypsum and possibly halite are formed towards the surface of the soil while the thin layers between have low sulphate. The level at which halite precipitation takes place is partly governed by humidity. At high humidity the salt appears on the surface whereas with a relative humidity of around 30%, the salt layer appears a few millimetres below the soil bed surface. When it grows within the soil, the halite is fibrous, but more equant crystals develop at the soil surface.

(ii) Proximity of fabric to the water-table The fabrics have been placed at various distances from the water-table and it has been found that the capillary break is satisfactory providing that the fabric is placed a few centimetres above the water-table and above the level at which the moisture content approaches saturation. The barrier is not effective if placed within soils approaching moisture saturation. Local damage to the fabric does not invalidate this conclusion and in some experiments where the fabric was not properly cemented to its securing frame, minor salt and moisture passed through. However, the barrier remained 90% effective for at least 18 months in the areas of imperfect jointing.

(iii) Influence of open ended structures Horizontal fabric sheets placed in the soil bed have so far behaved much as the fabrics attached to impermeable frames. There has been only minor lateral movement of moisture and salt over the fabric layer. Where Filtram is used as a vertical side wall, it acts as an efficient barrier preventing lateral migration of moisture and salt.

(iv) Vegetation in the soil beds Various plants readily became established within the soil beds initially but were rapidly eliminated as the salt levels rose. Soil beds placed above Filtram continued to support

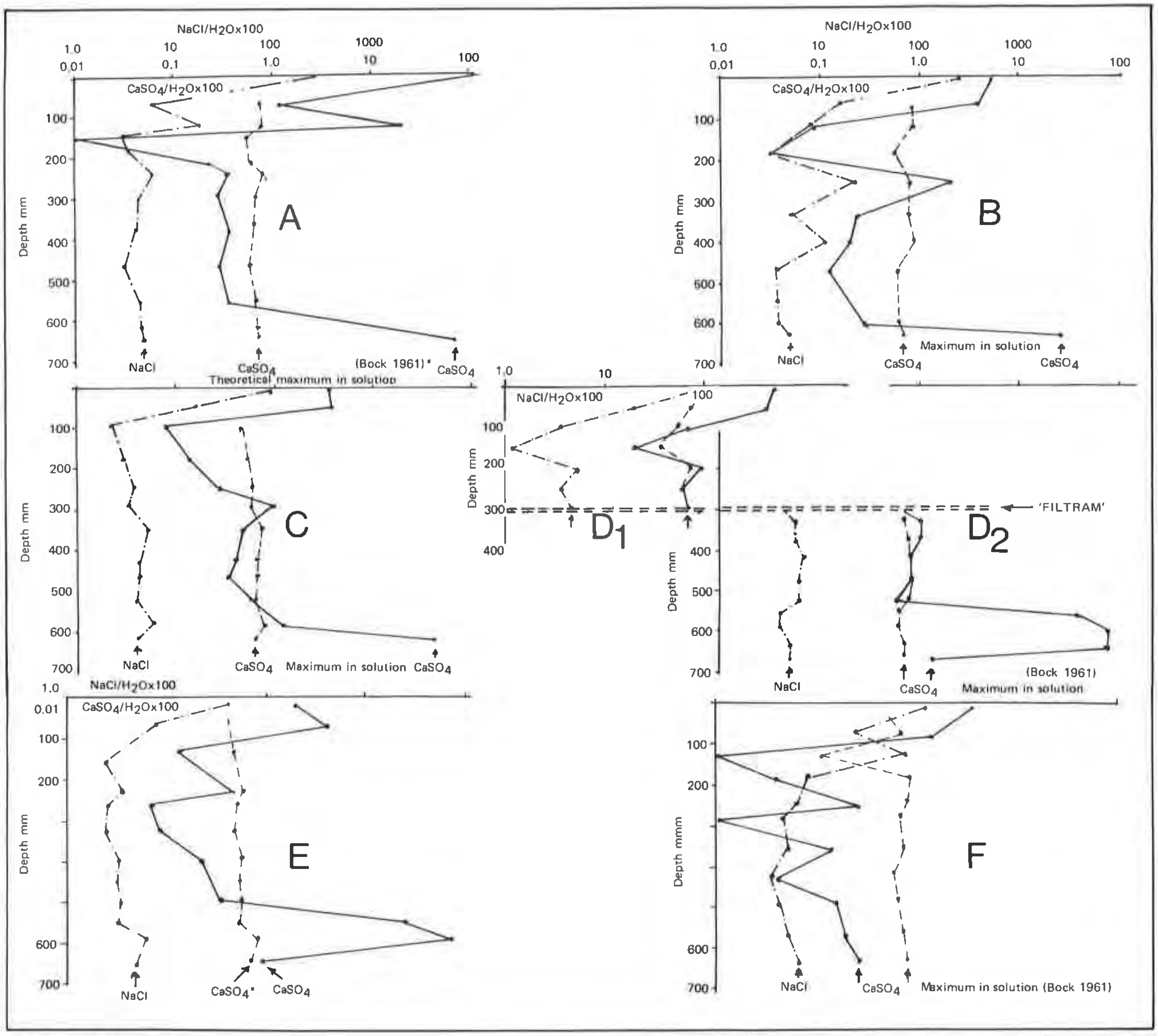

Figure 10 A, B, C, and D, are all from a single soil bed containing carbonate and siliceous material. The graphs represent profiles through the soil beds and show the ratios of NaCl and $CaSO_4$ to water in the profiles. Each diagram represents a separate profile and the locations of the profiles are shown in figure 8. D1 represents a profile from near a Filtram layer and D2 is the profile from beneath the fabric. E and F are from a second tank in which the soil was wholly siliceous. In each of the six diagrams the third line represents the ratio of $CaSO_4$ to water at which the solution becomes saturated in gypsum for the found level of NaCl. To the right of this line gypsum will be precipitated. The erratic variation in the upper part of these curves is not error but shows that the sulphate content is separated into layers alternating with bands of very low sulphate content.

vegetation throughout the entire period of the experiments. Watering with sweet water did not bring increase in salt concentration within the soils.

(v) Irrigation of the soils Experiments showed that watering soil beds with sweet water above Filtram could be carried out without salt accumulating in the soil and without water passing downwards through the fabric. It has been estimated that a substantial surface flood would be needed before the possibility arose of the sweet water passing downwards through the fabric. However, damaged fabric will break down more readily if the irrigation volume is high. In one experiment the soil was allowed to dry out between waterings and in this case salt precipitation occurred beneath the fabric and a strong layer, some 15 mm thick, of halite and gypsum was produced. Nevertheless, salt did not pass through the fabric nor did it enter the pores of the fabric.

(vi) Surface of soil impermeably covered Experiments have been carried out in which concrete paving and impermeable plastics have been placed on the surface of the soil beds above Filtram and away from the area of Filtram. Where the Filtram is present, the sand remains totally dry and salt-free and where no fabric is present the rate of salt and moisture rise is reduced to about one twentieth of that outside the covered area in the early months. The build-up of the salt and moisture profile is obviously retarded by the presence of the cover and is entirely prevented by covering the surface and interposing the fabric.

(vii) Soil surface surcharged Some preliminary experiments have been carried out in which the soil above the fabric has been loaded with concrete slabs. The loads applied so far have been applied for about a year without influencing the properties of the fabrics as a capillary break.

D. Conclusions drawn from experiments

The experiments carried out so far have shown that multilayer fabrics such as Filtram can provide an efficient capillary break that prevents the rise of salt and moisture. The fabric must be placed above the zone in which the soil is nearly saturated but otherwise is best placed as deeply as possible in the soil. The reason for this is that salt can build up beneath the fabric and this takes place most rapidly if the barrier is placed in the dryer parts of the capillary fringe. The creation of such a layer of salt crystals may have disadvantages, say, in the base or sub-base layers of a road structure, for here the crystallization may cause the mechanical disintegration of certain types of rock used as constructional materials. Generally irrigation does not cause the breakdown of the barrier and indeed it is a problem that, if brackish water is used for watering, salts will build up progressively in the growing soil. It must be stressed, however, that these difficulties may also occur, and be more serious, when the barrier is made of rock and fabric, for salt precipitation is likely to occur within the layer of drainage material.

4. FIELD TRIALS

As outlined in the first part of this paper geotextiles have been widely used for horticultural purposes in the Middle East and recently the experiments described above have been extended to field trials in which bituminous roads have been constructed which contain a barrier of multilayer fabric. In Bahrain, for example, a section of road, the adjacent car park, and the verge have Filtram placed between layers of road-base materials. The road is built on an embankment some 800 mm high at its maximum elevation above a sabkha surface. This sabkha surface is generally damp with saline groundwater and there is usually a salt crust present. The embankment materials conduct both moisture and salt by capillarity for at least 600 mm upwards above the sabkha and into the embankment. The structure of the embankment is as follows. Resting on the sabkha is 500 mm of fill made of limestone rubble which includes everything from cobbles to dust. Above this is 300 mm of material graded from 150 mm to dust in the centre of which is a layer of Filtram. Above this layer of road base material is a layer of 150 mm thickness which grades from rock pieces up to 75 mm in size and down to dust. A bituminous layer some 75 mm thick succeeds this and is to be topped by a 50 mm thick wearing course. To date the road has not been excavated, but the shoulder above the fabric appears to be dry in comparison with the shoulders of the road where the fabric is absent.

5. GENERAL CONCLUSIONS

Commercial uses of geotextiles in arid regions have become commonplace and may be expected to increase. This paper reviews some of the most common applications. One of the most important growth areas for the use of fabrics is likely to be in their use as a capillary break and field and laboratory tests reported here show that multilayer geotextiles can make an efficient capillary break for both horticultural and engineering applications. Once established such breaks should remain effective for at least several years if correctly positioned. They can provide a surface upon which soluble minerals grow if placed too close to the water-table. Well compacted soil above such fabric layers remains dry and salt free, especially if covered by a pavement of some kind. If not covered the fabrics permit the transport of water vapour but not liquid and hence prevent the rise of salts.

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Geotextiles in the Sports Grounds

Les géotextiles dans les terrains de sports

Stabilised sports grounds are generally used for training purposes, their surfaces are composed of sandy material (sieve analysis 0/3 percent by weight of filler less that 30%) shale or puzzolane.

- Appoints of the geotextile :
  flexibility and elasticity. These two parameters are mesured by an apparatus konw as "Sportest".
- Functions of the geotextile :
  . No contamination between the sandy material and the draining material
  . Filtering ability
  . Hydraulic regulator

These parameters bring to select a needled nonwoven geotextile whose the permeability will be

$$\frac{Kn}{Hg} = 2.10^{-2} S^{-1}$$

and treated in such a way to retain some water in is thickness.

Les terrains de sport en sol stabilisé sont généralement utilisés pour l'entrainement des joueurs, leur surface est composée de matériaux sableux (0/3 concassé pourcentage de filler inférieur à 30%) de schiste ou pouzzolane.

- Apports du géotextile :
  . souplesse et élasticité, ces 2 paramètres sont mesurés par un appareil appelé "Sportest".
- Fonctions du géotextile :
  . Anti-contamination entre le matériau sableux et le matériau drainant.
  . Filtration
  . Régulateur hydraulique.

Ces paramètres ont amené à selectionner un géotextile nontissé aiguilleté dont la perméabilité sera

$$\frac{Kn}{Hg} = 2.10^{-2} S^{-1}$$

et traité de façon à retenir un peu d'humidité dans son épaisseur.

Les sols stabilisés mécaniquement étaient anciennement appelés sols semi-stabilisés ce qui indique que leur stabilisation mécanique n'est pas très poussée. Ces terrains doivent permettre un entrainement intensif afin d'alléger l'utilisation des terrains gazonnés, tout en ne nécessitant qu'un entretien simple. Ils doivent aussi être souples et perméables et avoir suffisamment de tenue pour permettre l'utilisation de chaussures à crampons.

Il est bien rare que l'on trouve un sol permettant, sans être mélangé à d'autres constituants, de réaliser un terrain de jeux. Ceci est parfois possible avec certaines qualités de sols particulières : arènes granitiques, faluns,schistes, mais extrêmement rare. Le plus souvent les chapes sont des mélanges binaires ou ternaires.

Les mélanges sont constitués d'un matériau concassé de granulométrie 0/3 étendue, pourcentage de filler inférieur à 30%, et de schiste ou pouzzolane.

Après compactage ces sols sont en général assez durs et donc peu confortables pour les joueurs. Afin d'obtenir une bonne souplesse et une bonne élasticité on a dans un premier temps utilisé des matériaux scoriacés (machefer bien calibré) mais devant la difficulté pour les trouver sur place cette solution a été abandonnée.

C'est en 1979 que le laboratoire des sols sportifs du Ministère du Temps Libre de Paris a réalisé divers essais mettant en oeuvre des géotextiles nontissés afin de caractériser les textiles apportant la souplesse et l'élasticité souhaitées selon le type de discipline sportive pratiquée.

Ces essais ont été réalisés avec l'appareil SPORTEST (photo 1)

- La souplesse se détermine par la déformation maximale du revêtement.
- L'élasticité se mesure par la vitesse de retour à l'état initial du revêtement.

Ces deux paramètres traduisent l'énergie fournie au sol par le pied du sportif et l'énergie restituée par le sol à ce pied.

L'appareil est un simulateur d'athlète. Le poids de la masse, sa hauteur de chute, et les ressorts "jarret" ont été déterminés pour obtenir des efforts voisins de ceux qui sont produits par le pied d'un athlète de 65 Kg courant le 100 mètres en 13 secondes.

En plus de ces éléments mécaniques, le Sportest est équipé :

- d'un capteur de déplacement électronique relié à une unité d'affichage indiquant la déformation du sol en 1/100 de mm et la vitesse de retour en mm/s.
- d'un oscillographe enregistrant sur papier photographique la courbe de déformation.

Le tableau ci-après (1) indique les résultats de ces essais à 4 emplacements différents.

. Emplacement A, structure traditionnelle sans géotextile
. Emplacement B, structure avec géotextile non tissé aiguilleté
. Emplacement C, structure avec géotextile non tissé thermo-lié
. Emplacement D, surface en gazon naturel.

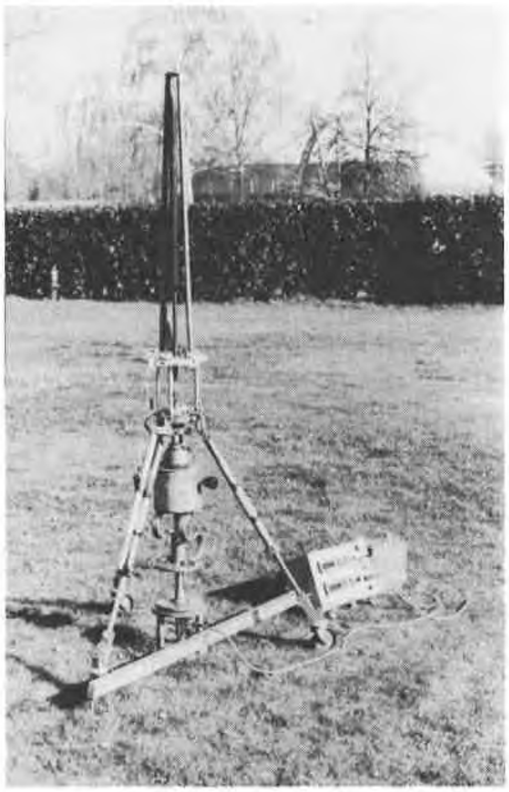

Photo 1 - Appareil SPORTEST

Tableau 1 résultat essais sportest

Emplacement	A	B	C	D
Déformation en mm	0,35	1	1,4	6,68
Vitesse en mm/s	11	38,5	52,5	93,35
Energie renvoyée	0	0	4,9	4,5
Tassement ou déformation permanente en mm	0	0,15	0	0

L'emplacement B (avec géotextile aiguilleté) présente, du point de vue qualités sportives, les meilleurs avantages alliant une forte souplesse à une forte élasticité et une énergie nulle par rapport à l'emplacement C où l'énergie renvoyée est forte et est sensiblement identique à celle d'une surface engazonnée.

Ce procédé (B) utilisant un géotextile nontissé aiguilleté placé entre la couche drainante et la chape permettra, grâce surtout à l'énergie renvoyée nulle, de diminuer la fatigue musculaire et d'alléger éventuellement le temps d'entraînement.

Il permet également d'éviter l'inconvénient de certains sols trop durs (matériaux stabilisés classiques) ou trop élastiques (matériaux synthétiques).

ROLES DU GEOTEXTILE

En plus de ses apports à la souplesse et à l'élasticité de l'ensemble du terrain, le géotextile joue 3 rôles.

a . Anticontamination (séparation)
b . Filtre (filtration)
c . Conservateur d'une certaine teneur en eau.

a . Anticontamination

C'est un rôle parfaitement connu aujourd'hui, dans tous les domaines d'application notamment en construction routière. Dans ce cas précis d'utilisation, le géotextile évitera, au moment de la mise en oeuvre et notamment lors du compactage, aux éléments fins de la chape de pénétrer dans la couche drainante. De même, pendant les périodes d'utilisation, en cas de détérioration de la chape, il évitera la remontée en surface des éléments de la couche drainante qui risqueraient de blesser les joueurs.

On tiendra compte pour le choix du géotextile de sa porométrie O_n, on utilise pour ce choix un seul diamètre O_{95}, qui est par définition tel que 95% des pores aient un diamètre inférieur à O_{95} et 5% un diamètre supérieur.

Le textile au contact du sol naturel aura un O_{95} d'au moins 400 μ

Le textile placé entre la couche drainante et la chape aura un O_{95} d'au moins 125 μ

b . Filtre

Le rôle de filtre est d'assurer la permanence du fonctionnement des ouvrages de drainage

- en évitant la contamination du système de drainage par les éléments fins du sol environnant
- en évitant l'érosion régressive du terrain situé autour du drain, risquant, dans certaines conditions, d'évoluer vers la formation de cavités.

Le géotextile doit avoir une perméabilité supérieure aux sols environnants. Les valeurs de permittivité établies par le Comité Français du Géotextiles dans le fascicule "Recommandations d'emploi des géotextiles dans les espaces verts, terrains de sport et de loisir" se calculent ainsi :

Le débit traversant une surface S de textile est égal à $\frac{Kn}{Hg}$ avec : Kn coefficient de perméabilité du géotextile
Hg épaisseur du géotextile

Ce rapport $\frac{Kn}{Hg}$ est appelé permittivité du produit et s'exprime en S^{-1}

Cette permittivité sera :

- Pour le textile au contact du sol naturel d'au moins $2.10^{-2}\ S^{-1}$
- Pour le textile entre la couche drainante et la chape d'au moins $5.10^{-2}\ S^{-1}$

c . Maintien d'une certaine teneur en eau

Le géotextile devra également exercer un rôle de régulation hydraulique. Pour cela, il devra être capable de retenir en son sein une certaine quantité d'eau afin d'éviter une évaporation importante d'une part et, d'autre part de maintenir la chape à une humidité constante. Cela est facilité par l'emploi d'un géotextile aiguilleté qui pourra être traité par enzymage.

En plus de ces trois rôles, les géotextiles devront répondre à certaines caractéristiques mécaniques de résistance à la traction et de résistance à la déchirure, ces caractéristiques étant surtout nécessaires au moment de la mise en oeuvre et du compactage de la chape.

La résistance à la traction αf dépendra surtout,

- pour le géotextile au contact du sol naturel de la qualité de celui-ci, si le CBR est inférieur à 2 αf = 16 kN/m et allongement $\varepsilon_R > 20\%$
- pour le géotextile placé entre la couche drainante et la chape : αf = 8 kN/m et allongement $\varepsilon_R > 15\%$.

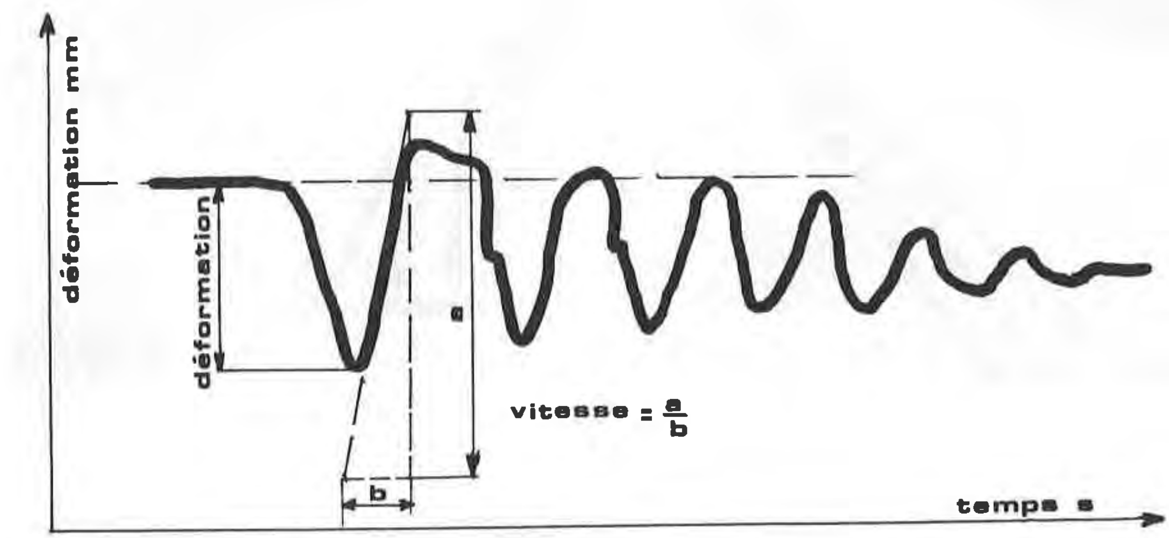

Fig. 2 Enregistrement Sportest sur emplacement B

La résistance à la déchirure amorcée F_t, déterminée selon la méthode d'essai C.F.G. sur éprouvette trapézoïdale de 670 et 225 mm pour les longueurs et 445 mm pour la hauteur; la déchirure étant amorcée sur une longueur de 50 mm au milieu de la petite longueur.

Cette résistance F_t sera de 0,3 kN/m pour les deux géotextiles.

Mise en oeuvre

Le géotextile au contact du fond de forme, ainsi que celui protégeant les tranchées drainantes sera mis en oeuvre avec un recouvrement de 0,1 à 0,2 m. La couche drainante, d'une épaisseur de 0,1 m sera parfaitement réglée.

Le géotextile placé entre la couche drainante et la chape sera légèrement humidifié avant la mise en oeuvre de celle-ci. La chape sera étalée manuellement et compactée à l'OPN avec un cylindre de 1 t/m.

L'emploi d'un géotextile nontissé aiguilleté dans la réalisation de terrains de sports stabilisés permet, en plus des rôles traditionnels de séparation et de filtration, d'améliorer la souplesse et l'élasticité de l'énergie renvoyée de l'ensemble. Enfin il assure une maintenance moins onéreuse en ayant un bilan hydrique supérieur à celui d'un terrain traditionnel.

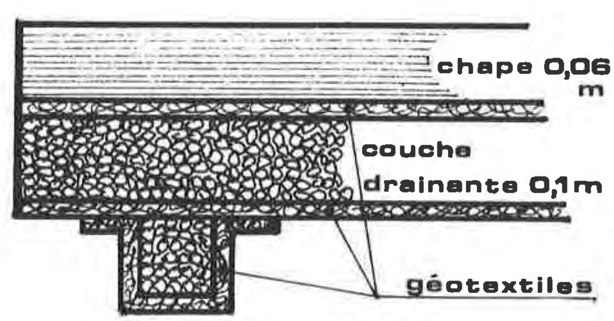

Fig 3 coupe terrain sport sol stabilisé

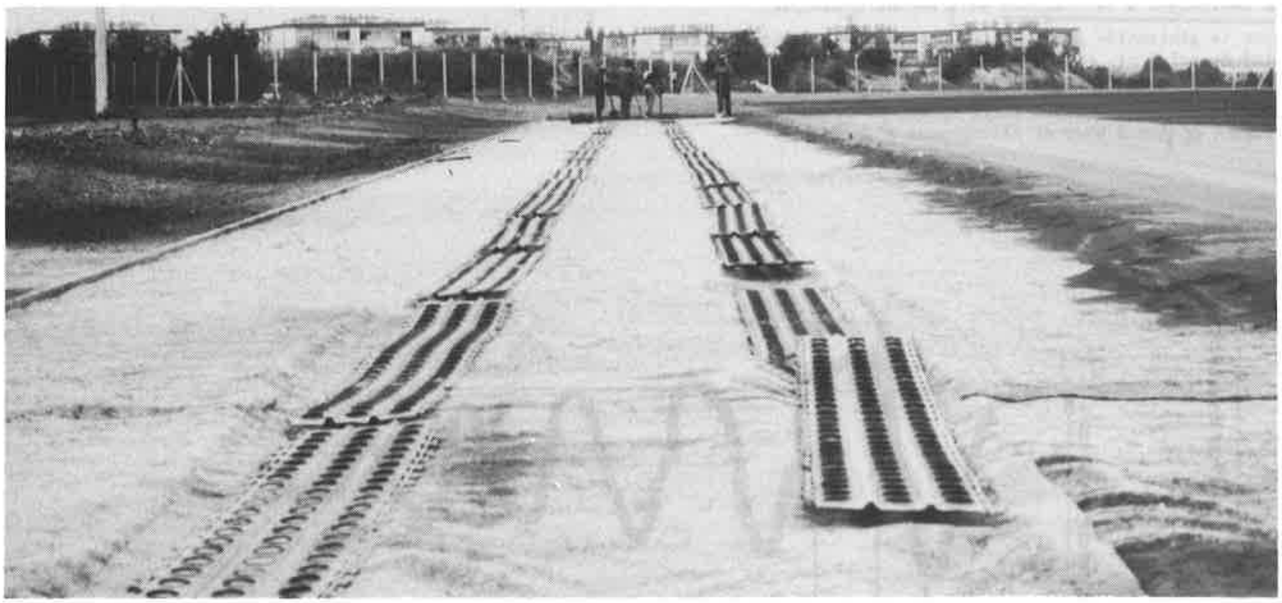

fig. 4 Mise en oeuvre

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Fig. 5 Géotextile placé entre la couche drainante et la chape

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Synthetic Fabrics as a Concrete Forming Device

Tissu synthétique utilisé comme système de moulage du béton

Since the advent of concrete as a construction material, engineers and contractors have been restricted in its use by the rigid forms of steel and wood required to contain the concrete until its initial set. In the early 1960's another man-made material, synthetic fabric was pioneered as a flexible forming system for concrete placement thus greatly expanding the potential uses of concrete. Furthermore, the relatively low cost of the synthetic fabrics has resulted in more economical concrete placement costs. This paper will trace the nearly 20 years of synthetic fabrics as a forming system covering its history, fabric development, design and installation criteria and significant case histories in the following areas.

1. Mattresses for slope protec tection.
2. Pile Jacketing.
3. Tubes and bags for underwat er concrete placement
4. Columns through mines and l imestone cavities.
5. Miscellaneous uses.

Depuis le béton soit utilisé dans la construction sont les ingénieurs et entrepreneurs limités par la nécessité de retenir le béton fluide dans des moules rigides de bois et d'acier jusqu'à sa prise initiale. Au début des 1960's est apparu un nouveau matériel fabriqué de main, le tissu synthétique, qui s'est introduit comme système flexible de moulage du béton, élargeant le rôle potential du béton à la fois que le coût assez bas du textile synthétique a mené à un coût plus économique de la mise en place du béton. Le présent rapport racontera l'histoire de vingt ans du tissu synthétique utilisé comme système de moulage du béton, sa développement, des critères pour le dessin et l'installation aussitôt que des dossiers importants dans les domaines suivantes:

1. Matelas pour protection de pente.
2. Enveloppement du pieu.
3. Tubes de mise en place du béton sous marin.
4. Colonnes à travers des mines et cavernes calcaires.
5. Usages divers.

INTRODUCTION

The first U.S. patent for utilizing fabric as a forming system was issued in 1906 to Robert Cummings of Beaver, Pennsylvania for building concrete piles. A pipe was driven in the ground, a sleeve of pervious coarse bagging was inserted into the pipe, concrete was poured into the sleeve-lined pipe, and the pipe withdrawn. In the early 1920's a Norwegian, John Store, developed a flexible fabric form for the placement of concrete in a subaqueous environment. In addition, the 1926 Danish book of inventions described a method using fabrics for casting large underwater concrete foundation blocks for the purpose of constructing bulkheads and wharfs.(1) So even though fabrics as a concrete forming system was recognized as a viable technique at about the same time that concrete was coming into use as a construction material, fabric as a forming system had limited use prior to the early 1960's when a new generation of synthetic fibers were introduced. The superior strength and performance qualities of the woven petroleum based yarns resulted in a great expansion of fabric formed applications in the area of construction. In the ensuing years, many patents were issued; so much so that, of the many uses of fabrics in construction, the use as a form is the most heavily patented.(2)

1.0 MATTRESSES FOR SLOPE PROTECTION

With the availability of the new generation of high strength synthetic fibers, attention was concentrated on the development of a dual-walled fabric envelope into which concrete could be placed for utilization in erosion control applications. Prior to 1970 numerous methods were suggested to join the fabric layers, for example, H.F.J. Hillen of the Netherlands, suggested using nails and washers. Other methods tried were sewing, gluing, thermoplastic adhesion, weaving and the utilization of integral spacer cords. Actual field applications have proven that weaving and/or integral spacer cords provide the most economical and strongest method of joining two layers of fabric.

With the weaving method, the two layers of fabric are joined intermittantly through the mechanical function of the weaving loom. The current method of joining fabric by the weaving process is by joining the layers together in a grid pattern. These 4.4 cm. (1.75 in.) circular joining points are spaced on 12.7, 20 or 25 cm. (5, 8, or 10 in.) centers and control the maximum thickness of the concrete inflated mats.

With the integral spacer cord method the two layers of fabric are interconnected by spacer cords. The length of the spacer cords may be adjusted during the weaving process to produce a fabric form of desired thickness. Within each square yard of fabric the spacer cords provide a minimum of 178 kN. (20 tons) of tensile strength, and in some cases exceed 333 kN. (37.5 tons). For the relief of hydrostatic pressure, non-corrosive weep hole assemblies may be placed in the fabric prior to concrete placement, at desired locations.

Where slopes are not subjected to continuous heavy flows and/or where environmental conditions or aesthetics dictate, an alternate fabric formed revetment mat may be utilized. This form work consists of interconnected tubes of concrete, the diameter of which may be varied according to the application. The areas between the interconnected concrete tubes provide a means for the placement of soil and ground cover and vegetation. This results in an aesthetically pleasing armourment.

1.1 Concrete Design

As the space between the two fabric layers may be as close as 7.6 cm. (3 in.), with internal cords at 7.6 cm. (3 in.) spacings and has to be pumped in place, a very fine-grained fluid concrete has been mainly utilized. A typical mix may be as follows:

Typical Mix:

6 - 10	Sacks Cement 2.5 kg. (94 lbs./sack)
68 - 115 kg. (50 - 150 lbs.)	Flyash (when available)
1,000 - 1,200 kg. (2,200 - 2,650 lbs.)	Sand (depends on FM)
180 - 225 litres	Water (48 - 60 gal.)
4 - 12%	Air (Depending on Geographical Area)

Some fabrics for the mattresses are specifically designed to allow the water in the mix to bleed through the fabric without allowing the cement to escape thus causing a lowering of the water cement ratio and an increase in compressive strength.(3)

1.2 Design, Construction and Cost Criteria

The revetment should be sized using the same criteria as established for conventional cast-in-place slabs. Based on experience in navigation canals carrying heavy traffic, the thrust and waves caused by ships is of particular importance in the design and dimensioning. As a guide for sizing mats thickness, the following is suggested:

a) Inland waterways, such as canals and rivers: 1.91 - 2.87 kPa (40 - 60 lbs./sq. ft.)

b) Costal areas and inland water with strong current and wave action: 2.87 - 7.18 kPa (60 - 150 lbs./sq. ft.)

Actual case histories show that reventment mats 9 cm. (3.5 in.) thick have been subjected to velocities between 4.6 and 5.5 m./sec. (15 and 18 ft./sec.) and those 60 cm. (24 in.) thick have withstood typhoon forces. Where hydraulic energy is a design consideration, the "n" value as used in the Manning formular will range between 0.014 and 0.030, depending on the type mattress.

Revetment Mats provide protection to stable earth slopes and surfaces which are subject to erosion by water. They may be placed on a horizontal or on gradients up to 1:1. Steeper gradients have been successfully completed but are not recommended. Mats should not be used on unstable slopes or where conditions of excess deformation is anticipated.

Most installations have the mat toed in a minimum of 0.3 m. (1 ft.) into a trench at the crown and bottom of the slope to prevent underscour. In subaqueous installation the bottom trench may not be utilized if the mat is taken 1 m. (3 ft.) below known scour.

The cost of an installed mat is subject to many factors, some of which are, project size, water conditions, local labor conditions and concrete prices. At the current time a 10 cm. (4 in.) reventment mat installed will range from between $16.00 to $21.50 per sq. m. excluding site preparation.

1.3 Case Histories

Johnstown, Pennsylvania

The Johnstown area first obtained flood notoriety when the South Fork Dam broke on May 31, 1889, killing over 3,000 people. The next major flood to hit the Johnstown area was the one-in-two-hundred year storm in the winter of 1936 and the peak maximum discharge of the Conemaugh River at Pennsylvania Electric's Seward Station was estimated to be 2,500 cms. (90,000 cfs.). When Pennsylvania Electric Company's engineers were designing a new dike to surround Flyash Pond No. 2, they wanted to make sure that the dikes were not only strong enough to take the force of any future flood waters, but also higher than the two past major storms.

The river side of the dike was designed to have a 2 in 1 slope and the face of the dike was designed to be protected by a 20 cm. (8 in.) filter point Fabriform® revetment mat with the top of the revetment mat toed into the dike at the height of the 1936 flood at this location. Also, in order to prevent undercutting of the dike, the Fabriform was designed to have a 0.6 m. (2 ft.) toe into original ground at the bottom of the dike. The Fabriform revetment mat was approximately 305 m. (1,000 ft.) long and covered just under 3,530 m2 (38,000 s.f.) of the surface area of the dike.

After the mattress was installed, an additional 1.5 m. (5 ft.) was added to the dike as an additional safety factor.

On July 20, 1977, heavy rains inundated the Johnstown area causing floods of a greater magnitude than the 1889 and 1936 flood. At the Seward Station site, waters were 1.5 m. (5 ft.) higher than the 1936 storm and the peak discharge was over 2,750 cms. (97,000 cfs.) equivalent to a one in 250 year storm. The town of Robindale, situated between Flyash Disposal Pond No. 2 and the Seward Station, was completely wiped out; with the Seward Station inundated with over 6.1 m. (20 ft.) of water.

Although scour of the upper unprotected dike above the mattress took place, the mattress completely protected the dike and prevented the large quantities of flyash in the pond from polluting areas downstream. Except for some minor damage caused by heavy debris in the flood waters scraping the mattress, the mattress itself was unscathed.

New York, New York

LaGuardia International Airport is built largely upon landfill and is protected from the adjacent shoreline by a sheet pile bulkhead. During initial construction, the toe of the bulkhead was covered with an asphalt-topped earthen berm to prevent erosion and underscour.

In late 1980, after years of damage from heavy wave action, the entire shoreline was littered with large and heavy debris which had penetrated the asphalt-topped berm and exposed the unstable earthen substrate. This condition subjected the sheet pile bulkhead to rapid underscour and potential failure during storms, which would result in flooding and damage of adjacent runways.

As a means of permanently protecting the bulkhead from erosion and underscour, the Port Authority specified an 20 cm. (8 in.) thick revetment mat to be placed along 800 m. (2,600 ft.) of the bulkhead.

After removal of the large amount of accumulated debris, the erosion-damaged slope was backfilled to provide an even surface for the revetment mat. The Hydro-lining fabric formwork was shipped to the site in preassembled panels of 279 m2 (3,000 s.f.) in area. These panels were positioned on the prepared slope and joined together using a portable sewing machine. Using a 19 m3 (25 c.y.) per hour concrete pump, the grout slurry was pumped into the fabric until it had inflated to its full predetermined thickness.

In order to relieve groundwater pressure which could lift and crack the mat, weep holes were placed in the mat on 2 m. (6 ft.) centers. The project required in excess of 4,645 m2 (50,000 s.f.) of revetment mat and was completed in November, 1980. The revetment mat was installed without interfering with flight schedules, and the finished product provides a attractive, permanent protection for the bulkhead and adjacent runways.

2.0 JACKET PILING

Up until the 1960's the main method of repairing deterioration of wood, concrete or steel piles in a marine environment was to attach metal half-shells together around the section to be repaired and connecting this to a cumbersome bottom form prior to filling the annular space with concrete. The use of fabric forms offers a viable and economical alternative method.(4)

After the pile is cleaned, reinforcing steel installed, a pre-cut fabric sheet fitted with a heavy industrial zipper is wrapped around the pile by a diver. The fabric form is zippered in place and clamped below the bottom of the zone of the pile to be repaired and the top is supported off the marine structure or clamped above the splash zone. Through hoses installed to the bottom of the repair, fine grained concrete is injected thus filling the annular space with a minimum of 7.6 cm. (3 in.) of concrete. Due to the flexibility of the fabric forms and despite the internal spacers, care must be exercised in fast moving water so that the jacket of fluid concrete is kept uniform around the pile until initial set. Also, as sewing of the zipper to the fabric restricts the normal 10% fabric expansion at that location, stitching on the opposite side of the jacket is recommended in order to avoid a "banana" shaped jacket.

The light weight, ease of installation and low cost of the fabric forms resulting from this system nearly always are more economical than other forming systems. However, these repairs are still highly labor intensified, always made by divers and dockworkers whose rates will fluctuate by a factor of at least ten depending on whether the job is performed non-union or union, and the idiosyncrocies of the local labor agreement. This coupled with many other variables such as number of piles to be repaired, depth and velocity of the water at the site, diameter of jackets, amount of cleaning required, amount of reinforcing steel needed, etc. makes it almost impossible to give even a budget cost figure for pile jacketing except on a job by job basis. Experience has shown that minimum cost start at $150/m.

2.1 Case History

Since the mid-sixties many piles have been repaired by this system in water depths up to 18 m. (60 ft.). However the following case history is the largest diameter on record to the authors' knowledge. After approximately 50 years of exposure to the Pacific Ocean environment, the steel forms on the 2.5 m. (8 ft.) diameter caissons supporting a pier for the exportation of copper ingots had badly corroded and revealed deteriorating concrete at Andes Cooper Mining Company, Chanacal, Chile.

The rehabilitation design called for a 30 cm. (1 ft.) cover of reinforced concrete encasement utilizing a fabric form. The concrete encasement extended from the top of the caisson to the ocean floor, a length of between 7.7 m. and 8.6 m. (25 and 30 ft.). After removal of all unsuitable material and marine growth, reinforcing steel was attached to the caisson along with non-corrosive spacing devices. A fabric form 3.1 m. (10 ft.) in diameter was then placed around these elements and zipped closed, suspended from the pier and banded tightly to the caisson slightly below the ocean bottom. Concrete injection pipes were installed in the bottom of the form and were slowly withdrawn as the fabric form filled with high strength fine aggregate concrete. Steel bands were used to provide temporary support to the fabric during pumping and were subsequently removed.

3.0 TUBES/BAGS FOR UNDERWATER CONCRETE PLACEMENT

Since the availability of high strength synthetic fibers it has been possible to construct large bags and/or tubes to cast large concrete elements in place in a subaqueous environment. These concrete elements have been used to protect shorelines against heavy wave action, repair scour of bridge piers and abutments, structural repairs, construct breakwaters and groins, pipelines cradles/saddles, etc. (2,4,5,6,7,8). These fabric formed elements are an economical alternate to the conventional methods of large riprap and/or massive precast concrete elements.

3.1 Design and Installation Criteria

When utilizing large tubes/bags for shoreline protection, close attention must be given to protecting against toe scour and undermining of the underlying soil.

Until the concrete sets, the fluid mass of concrete could roll out of position endangering divers working with them. Therefore, the fabric tubes/bags may be anchored and held in proper alignment with steel stakes which may be removed after the concrete has set. To insure proper alignment when stacking the tubes/bags, the joints should be staggered. A temporary supporting device such as angles may be used to prevent the tubes/bags from rolling. Straps and/or grommets may also be used. The tubes and bags may be doweled together with reinforcing steel by inserting one end of a reinforcing steel bar through the fabric and into the fresh concrete, the other end is threaded into the next tube/bag prior to concrete placement. The maximum thickness a tube/bag may be filled is limited by its width. For all practical installations the height should not exceed 60% of the width. The length of tubes/bags have no maximum, however, those of 3.1 m. (10 ft.) are most popular, with tubes as long as 27.5 m. (90 ft.), to heights of 1 m. (3 ft.) and widths of 1.85 m. (6 ft.) being installed in over 12.4 m. (40 ft.) of water.

Placement of the concrete into the tubes/bags is accomplished by means of a concrete pump. The pump hose is "inserted into" the inlet valve (self sealing or manually tied) and filled with a sand/cement mortar. The mix design being within the same range as that utilized in the mattresses and described earlier. Pea gravel may be added to the mix to lower the material cost providing pumping conditions are tolerable.

3.2 Cost

The tubes/bags material will range in cost between $3.25 and $7.50/sq.m ($0.30 and $0.70/sq. ft.) depending on the material used which is dictated by the size of the tubes/bags.

4.0 COLUMNS THROUGH ABANDONED MINES AND LIMESTONE CAVITIES

Many parts of the earth are underlined by abandoned mines or limestone cavities whose subsistence could cause major failures to structures situated over them. The cost of grouting or backfilling these abnormalities is high and often indeterminate. One alternative is to form columns through the cavities on a grid pattern to support the roof of the cavity and prevent failure during the life of the structure. The technique involves drilling 12.6 cm. to 15 cm. (5 to 6 in.) diameter holes through the cavity and a minimum of 1 m. (3 ft.) below the floor, then to snake down the hole a fabric tube having an inflated diameter of between 0.6 and 1.2 m. (2 and 4 ft.). A steel pipe is used within the tube for stiffness and to inject the grout. When the fabric tube is into the drill hole below the floor, the grout pipe is slowly extracted as grout is injected. Care has to be exercised as not to rupture the tube with too high a hydraulic pressure and a multilift filling operation can be utilized.

4.2 Case History

A fossil electric generating plant in South Carolina is supported mainly on caissons extending through the overburden into limestone rock. The limestone is subject to cavitation and extensive grouting has been performed in the cavities beneath the caisson. A limestone reclaim pit was to have a deep foundation extending to the top of the rock. In order to prevent future settlement, while avoiding the large and indeterminate cost of cement grouting, it was elected to form 1 m. (3 ft.) diameter concrete columns through the cavity in the limestone. In the 6.1 m. by 10.4 m. (20 ft. by 34 ft.) area, 14 holes were drilled and the height of the cavity varied from zero to 4.3 m. (13.9 ft.) with the roof of the cavity at approximately 18.2 m. (60 ft.) from the surface. In order to test the continuity of the columns, the fabric was wrapped around a closed-end PVC pipe. After the fine-grained concrete inflated the tube to the 1 m. (3 ft.) diameter, geophysical sonic velocity probes were performed and verified that 1 m. (3 ft.) diameter tubes had been formed.

5.0 MISCELLANEOUS USES

5.1 Off Shore Pipeline Installation Cradles

In order to speed up installation of offshore pipelines, fabric tubes have been designed to be attached to the leading edge of the pipeline. After being inflated with fast-setting concrete, these serve as cradles for support of the pipe until the trench under the rest of the pipe is being backfilled thus freeing up the surface support equipment sooner and resulting in a more economical installation.

5.2 Cylinder Pile Closures

In order to form continuous walls in aqueous environments, large diameter cylinder piles have been installed, but the forming of the uneven vertical closures has been an expensive operation. Fabrics sewn on the job to accurately conform to the configuration developed in the pile placement operation and to meet the varying requirements of the project can be economically installed, reinforced and inflated with concrete.

5.3 "Safe" Riprap

An alternate design for an offshore circulating water system for a nuclear power plant called for the pipe to be buried in a trench in the ocean floor. The Nuclear Regulatory Agency was concerned that conventional riprap used as protective backfill would harbor marine life which could be exposed to contamination. Therefore, the design called for concrete inflated fabric tubes to be placed over the backfill and installed so that the convex-concave edges of the tubes would allow flexibility without allowing intrusion by marine life.

5.4 Scour Protection for Offshore Structures and Pipelines

The research platform "TVORDSEE" compared the advantages for scour protection of nylon woven mats filled with grout to artificial seaweed, sandbag clusters in nylon netting and pre-cast concrete slabs hinged to the edge of the platform. The mats weighing 3.5 kn/m2 proved relatively economical and offered a high degree of scour protection.

5.5 Raising of Earthen Dikes

When it is required to raise earthen dikes, one technique is to use fabric tubes inflated with concrete. A narrow trench can be dug in the top of the dike for positive cutoff and a single tube, or pyramid of tubes with vertical dowels, used to conform to the project requirements.

6. FUTURE DEVELOPMENTS

The use of synthetic fabrics as a concrete forming device has unlimited potential. One natural extension of its utilization is in the installation, repair and maintenance of offshore structures and related subsea installations. (8) With continued development of stronger synthetic fibers, special sewing/fabricating techniques and specialized installation contractors, some of the following applications become practical: a) Pipeline perimeter lining - continuous or rip joined; b) Protective domes for subsea installation; c) Repairs to pipeline protective coatings; d) Sleeve Grouting - electrical conduits/cables; e) Repairs to damaged structural nodes; and, f) Pipeline support and relining of large diameter corroded or damaged underground sewer lines.

7. DEVELOPMENT OF SYNTHETIC FABRIC AS A CONCRETE FORMING SYSTEM

Woven fabric of nylon, polyester and polypropylene have been used almost exclusively for the predescribed applications. Polypropylene being a byproduct from production of petroleum product is more available and economical and less subject to continuing cost escalation then nylon.

For economic reasons non-woven fabrics have been experimented with, but either poor aesthetic appearance, too low textile strength, non-uniform strength in different directions, inadequate connection capabilities, etc. have effected their more widespread use. Current weaving technology allows different yarns to be utilized in the warp and weft directions if so desired. Most forming fabric uses have been with yarns between 840 and 2,000 denier, however, deniers as low as 500 and as high as 3,200 have been used. The yarns utilized to produce fabric forming should be ultra violet stabilized to increase the longevity of the fabric form.

8. CONCLUSIONS:

This heavily patented field has resulted in only a small number of construction firms having installation experience; only a few suppliers of fabric materials and most regrettably little funding for basic research, thus the lack of interest in the academic community in this area. Consequently, to date there has been a limited number of technical publications and reports on using synthetic fabrics as a forming system with most emphasizing the case history approach. However, the flexibility of the fabric forming system will continue to inspire the imagination of the engineers and contractors to use fabrics as a tension form and solve many future problems economically.

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The Advancing Techniques in Flexible Forms

Les techniques progressives dans les coffrages flexibles

The term flexible formwork refers to the various applications of porous textiles in the containment of settable materials such as concrete and the like. In spite of well-documented evidence and the clear commercial success in the United States of flexible formwork in erosion control, little innovative field use has been made of the basic technique outside the interest of specialised concerns The purpose of this paper is to describe some of the more recent European instances which have highlighted the advances in the flexible formwork technique. Each of the case histories to be discussed includes some unique feature which builds upon the basic principle of the system They have been selected to illustrate the significance of the textile components in harmony with other engineering materials, thus providing a versatile solution to civil engineering problems. In common with many of the geotextile topics presented at this conference, the market must be reeducated if flexible formwork is to have optimum utilisation. It is hoped that this paper will play its part in stimulating progress towards the inclusion of geotextile technology in all engineering courses.

Le terme coffrage flexible fait référence aux diverses applications de textiles perméables utilisés pour la retenue de matières coagulables telles que le béton, le mortier et autres substances similaires. Malgré de nombreux résultats pratiques et le succès incontestable des coffrages flexibles aux États-Unis dans divers domaines tels que la prévention de l'érosion, il a été fait peu usage de cette technique en dehors des applications connues réalisées par certaines entreprises specialisées. Le but de cet article est de décrire quelques applications récemment faites en Europe qui mettent en évidence les progrès réalises dans les coffrages flexibles. Chacun des cas présentés comprend un élément original en complément de la technique déjà connue. Ils ont été sélectionnes afin de souligner l'importance du choix du composant textile en fonction des autres matériaux utilises. Le tout apportant une solution adaptée à différents problèmes du génie civil. Comme la plupart des autres sujets dans le domaine géotextile en discussion à cette conférence, il faut souligner le besoin d'information du marché dans le domaine des coffrages flexibles afin que cette technique puisse prouver tout son intérêt.

INTRODUCTION

Conventionally, the method of casting insitu structures and shapes in concrete involves the use of formwork fabricated from rigid timber and steel The essential requirement is that these forms provide an impermeable container holding the fluid concrete throughout the hydration period. The duration of this holding phase depends mainly on the desired concrete strength, the climatic conditions, safety of site personnel and the likely effect of the environmental conditions surrounding the finished structure

An alternative concreting method is to use flexible formwork. These forms are constructed from permeable woven textiles, having the feature of providing a porous wall to the injected concrete. Essentially, this allows the free passage of excess water to pass out from the filling material.

This porous wall feature is fundamental to the whole concept of flexible formwork, and it will be seen by illustration that, in itself, it provides the solution to many everyday engineering problems.

To examine in more detail its potential and the manner in which it functions, let us consider the first case study.

1 ENGLAND

The main contract was a flood alleviation scheme which involved the widening of a river beneath a railway bridge. The new line of the river's edge was to be sheet piled, and was to pass within 1 metre (3.28 ft) of the bridge piers. The design authority's main concern was that the bridge loadings should not create movement of the sheet piles, resulting in the collapse of the structure.

To overcome this problem, a concrete slab was proposed to be placed on the river bed to act as a strut between the two rows of steel piles

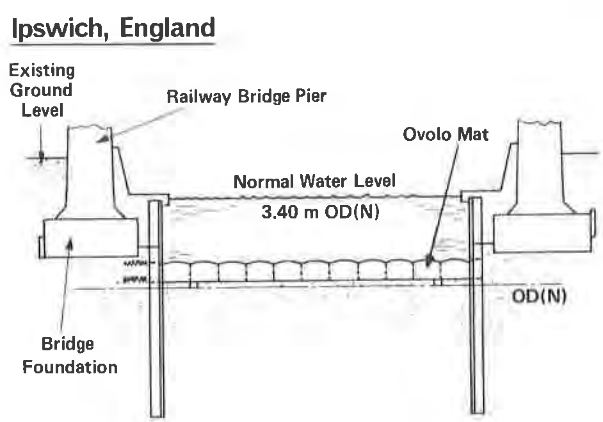

Fig 1: Cross section showing proposed strut

The slab was specified as having a minimum thickness of 0.5 m (1 64 ft), and a strength at 7 days of 21 N/mm² (3045 lbs/in²) It should be homogeneous and in full depth contact with the sheet piles both sides of the river.

Established practices required the construction of a cofferdam, de-watering and a bypass from the river flow.

To examine the suitability of a flexible formwork solution to this application, and to clarify the theory, let us delineate the following possible advantages.

a Porous wall

b Ease of handling

c Adaptability of shape and form

d Scope for alternative filling material

e Cost advantage

1a POROUS WALL

As previously stated, when used in concreting the flexible form is produced from permeable textiles. The fabric is constructed to allow a controlled bleeding of the excess water in the filling material, whilst restraining the loss of its solids.

The water loss produces rapid compaction and results in a dense high strength durable concrete. This can be achieved equally well under-water as in the dry.

It can be seen that there is an immediate advantage in this facility to the situation described, in that the need for a dry working area can be eliminated.

Although not relevant to the situation under discussion, adverse conditions can often affect the porous wall feature, for example, fabric placed in turbulent water will be subjected to buffeting and kneading. This will tend to squeeze out the fine binding particles through the filter wall of the form producing a sandy surface to the finished structure. In turn, this surface will wear and flake under the abrasive action of the water. Although it is interesting to note that, as soon as the finer fractions of the outer layer have passed through the fabric, the coarser part will tend to build up an internal secondary filter inhibiting the further outward migration of particle fines. In this situation, the resultant sandy appearance will only be superficial unless heavy kneading (and hence movement and agitation of the whole mix) has taken place within the fabric formwork.

In cases such as this, more attention must be paid to hydraulic bleeding properties in selecting a suitable fabric. It may also demand further consideration in mix design of the filling material.

1b EASE OF HANDLING

The length of slab section required was 40 m (131.2 ft). Had a cofferdam been used, it would only have been necessary to provide stop end shutters at various intervals lineally along the section, the sheet piles acting as side shutters and normal screeding as the top profile.

The total area of flexible formwork was 520 m² (5335.2 sq ft), and weighed 611 kgs (1345 lbs). It was placed in 10 pieces and joined underwater by the use of continuous chain zip. Mechanical equipment was not required and installation was performed by only a 4 man team. Clearly there was no advantage in the system's low weight to volume ratio over the small amount of rigid shuttering that would have been required by established methods. However, it will be appreciated from the evidence in case studies discussed that weight can play a significant part.

To maintain handling ease in some conditions special care must be taken in terms of size and method of installation. For instance, when sheet sizes reach 300 m² (3078 sq ft) in open exposed sites, high winds can be extremely dangerous. The form can fill with wind in the same way as a boat sail, and can easily lift 5 or 6 men off their feet, which could possibly result in a fatal accident.

Further, when placing large areas underwater, it is much safer for the formwork to be packed like a parachute, and similarly deployed when in the approximate position for its installation. Undercurrent and wave action resemble wind effect and can be more dangerous for the installation team. In one known application where an antiscour protection form was being positioned around an offshore structure, the form was wrenched from the divers and was lost for several days. Fortunately, the divers were not entangled in the sheet. The shutter turned up intact, but several miles along the coast. It was decided to proceed using flexible forms, but on smaller unit sizes.

1c ADAPTABILITY OF SHAPE AND FORM

The effectiveness of the strut in the case study under discussion clearly depended on it being in contact with the sheet piles both sides of the river to a minimum depth of 0.5 m (1.64 ft).

The design authority insisted that the whole profile of the pile pan should be filled with concrete that was integral to the slab formation. The sheet piles were Larson number 5, having a pan depth of 0.23 m (9 ins).

The adaptability of flexible forms is infinite, and in this case it was quite simple to tailor the side walls of the form to follow the pan profiles. The question of tolerance was overcome by using a unidirectional stretch woven fabric. This was an open construction in polyethylene, and provided a 20% elongation well below its ultimate tensile strength. The penetration of cement laden fines through this fabric facilitated a tight bond to the steel piles.

The form for the slab profile was fabricated to provide a dual layer envelope in a woven polypropylene having the following physical characteristics:

	Warp	Weft
Tensile Strength	60 kN/m	60 kN/m
Elongation	20%	20%
Permeability	545 L/m² /s	545 L/m² /s
Particle Retention	O_{90}	104 μm

To resist the ballooning effect of the internal hydraulic pressure, the top surface of the form was joined to the bottom surface by tubular columns in the same fabric. This provided an adequate uniform cross section within the tolerances of ± 75 mm (3 ins) imposed by the design authority.

1d SCOPE FOR ALTERNATIVE FILLING MATERIAL

The specification of the filling material will mainly be dictated by the structural qualities demanded in the finished construction.

In the situation described, the slab was required to have specific structural qualities, and fine aggregate concrete was preferred by the design authority. In terms of suitability to current formwork fabrics the filling material selection can be from the following range:

Material	Preferred Application
Neat cement grout	Structural repairs
Fine aggregate concrete (cement rich mortars)	Thin section slabs/shapes for severe exposure
Plasticized concrete	Thick section slabs/shapes, severe exposure
Resinous grouts	Thin sections in polluted environments
PFA cement mixes	Thin sections of high strength, light weight for severe exposure
Bentonite	Temporary constructions
Asphaltic grout	Thin sections in settling conditions of severe exposure

In selecting a suitable fabric to meet a particular filling material specification, it is important to balance the relationship between controlled bleeding/solids retention and internal hydraulic pressures.

From the earlier notes on the porous wall feature it will be readily understood that rapid expulsion of the free water in the mix will greatly reduce the hydraulic head on the fabric wall.

Flexible forms behave in much the same way as other liquid retaining membranes. Hydraulic pressures on the wall of the form act in an identical manner to those on a dam wall. The modulus of elasticity of the principal textiles used is around a 50th of that of steel. A curved surface will, therefore, always develop in the unrestrained areas.

The most drastic ballooning effects occur in trying to achieve a vertical plane. This can be expressed mathematically.

Assume the fabric between two anchor points behaves as a catenary under unfirom pressure. The tension in the fabric is obtained by matching the length and extension under tension to an assumed deformed shape, using an iterative process.

Assume value $a = {}^{f}/2L$ (1)

From relation $\frac{2aL}{h} + 1 = \cosh {}^{L}/h$

Obtain ${}^{L}/h$ where $h = {}^{H}/p$ (2)

Calculate the arc length

$s = \sqrt{f^2 + 2hf}$ (3)

Reaction $V = Ps$
$H = Ph$

and tension $T = P\,(f+h)$

Calculate extension of catenary

$\Delta s \quad \frac{(T+H)\,s}{2AE}$ (4)

where A = area
E = elastic modulus

then compare $L + \Delta s$ with s (5)

il compatible end of solutions,
otherwise back to step 1.

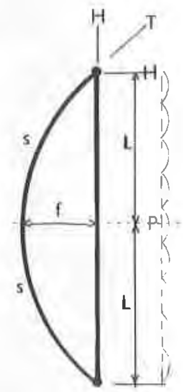

Fig 2:

It is essential therefore that if fabric stresses are to be minimised, the final shape of the filled form must be so designed as to distribute them evenly throughout the form. Furthermore, the shape must still impose an early restraint on the slump of the filling material to fully utilize the porous wall effect which is essential to the physical properties of the fill as well as reducing hydraulic pressures.

It is interesting to note high stretch fabrics may balloon to an unacceptable extent, but have the unique feature of increasing hydraulic bleeding as the pressure increases. As the bleeding increases, pressure reduces and a balancing effect, albeit indiscriminate, occurs during the filling operation.

In the writer's experience the most reliable way to determine the optimum container shape has been to construct a scale model using multidirectional lightweight fabric such as a jersey knit. The model is then injected with plaster and later used to plot dimensions for the fabrication of the form. The profile data obtained is then used to ascertain the fabric stresses.

1e COST ADVANTAGE

In the current economic climate, many prospective clients are adopting a "least cost" criterion in the selection of preferred methods. Field experience has shown that one or more of the technical advantages detailed above provide an indisputable cost benefit in favour of flexible formwork over established methods.

The reason for the economic attractiveness can be summarised for clarity as follows:

i It provides ease of access into otherwise difficult and, therefore, expensive locations.

ii It possesses the ability to produce improved qualities in the filling material, therefore widening the choice of materials to be used.

iii It provides the technical feasibility to produce consolidated masses underwater, negating the need for cofferdams and dewatering. Also, in tidal zones the working shift need not be affected by changing water levels.

In situations where underwater concreting prevents the movement of expensive shipping traffic, e.g.; in RO RO berths, the rapid consolidation of the concrete when placed in flexible formwork facilitates a return to normal working in only a few hours.

iv Time saving: The forms are easily installed and in many cases are left in place as part of the finished structure, therefore dismantling time does not apply. The low weight of the fabric often enables movement on site to be carried out by hand, eliminating the need to wait for availability of more specialised equipment, e.g.: high lift cranes.

v Its low weight offers cost saving in transport to sites.

In the situation described, the total cost of the strut construction was £25,000 (approx. $37,000).

The volume of fine aggregate concrete used was 260 cubic metres (340 cubic yards), costing a total of £9,100 (approx. $16,600).

The balance covered the cost of preparing the bed together with supplying, laying and filling of the form. It can be seen that this amount would have been considerably greater had a cofferdam been used.

The mass placement of underwater concrete is an aspect in the use of flexible forms which has an increasing opportunity for growth. The following case study is an example of one specialised application, where it has been successfully used.

2 BELGIUM

A precast caisson forming a tunnel segment was required to be anchored into a canal bed in some 20 m (66 ft) of water.

The design involved sheet piles driven in line on the bed being concreted to a semi-rigid neoprene mounting cast into the tunnel segment.

This was achieved by using a steel bell-shaped shutter 50 m (164 ft) long, incorporating fixings for the mounting. A flexible form was attached to hang below the housing to provide a bottom and side enclosure for the pumped concrete.

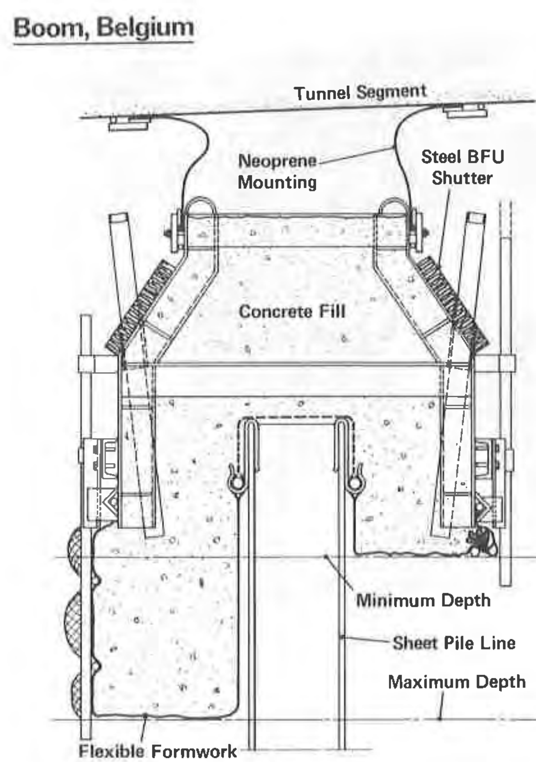

Fig 3: Typical cross section through formwork.

As we describe the site conditions in greater detail, it will be seen that the adaptability of shape and provided by this technique is an invaluable tool in the hands of the innovative engineer.

In spite of zero visibility below the water, it was ascertained that the bed level varied both sides of the driven steel piles. The bed level also varied along the length of the pile run and these variations were continually changeable.

The flexible form was provided with a fullness of fabric accomodating the maximum depth of bed variation. Experience has shownthat any excess material is merely trapped by the injected material, and this fact

precludes over stress.

The internal faces of the form following the line of sheet piles was manufactured from a very open woven mesh fabric. This ensures a good adhesion between the concrete and the piles.

Excessive balooning of the outside walls was restricted by steel bars fixed vertically to the steel shutter above, and by horizontal bars threaded into pockets in the side walls of the flexible form.

The above case study also introduces another aspect of the applications of flexible formwork where increasing opportunities exist. That is the combination of rigid and flexible components, thus providing a shutter which utilises the porous wall feature to obtain quality improvements to the injected concrete, together with the flatter surfaces provided by more rigid materials.

The following case study illustrates how simply this complementary aspect can be applied.

3 HOLLAND

The site was a general refuse tip. A considerable number of piers carrying a railway track were reported as being damaged by acid attack and machinery impact.

figure 4a the rigid components assembled

Figure 4b the flexible formwork being filled

The formwork proposed combined a permeable textile with steel wire mesh, and standard angle iron, thus providing the removeable shutter components. Preferred steel caps afforded permanent protection to the leading edges of the finished structure.

The main advantages of using flexible formwork in this case were:

i The speed with which each pier could be protected without involving a multiplicity of heavy weight shutters.

ii Little preparation work being required around each pier.

iii The need to provide access for cranes etc. being eliminated.

CONCLUSION

In conclusion it can be seen from the growing numbers attending conferences such as this, that textile engineering is becoming a more progressive science.

It is, therefore, difficult to avoid the conclusion that it will have to become a more important part of engineering educational courses.

The use of flexible formwork is typical of one small, but important, segment of textile engineering applications. These present new possibilities and design challenges to the mechanical and civil engineer of the future.

ACKNOWLEDGEMENTS

The writer wishes to express his gratitude to Henri F J M Hillen for his encouragement and inspiration.

The fabrics referred to are, in the main, from the range of industrial textiles produced by SA UCO NV of Belgium.

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Southern Pacific Transportation Co. Utilization of Geotextiles in Railroad Subgrade Stabilization

Les géotextiles dans les fondations de voies ferrées de la Southern Pacific Transportation Company

Southern Pacific Transportation Company field tests and applications have proved that geotextiles should be definitely considered in railroad subgrade stabilization. When an existing track structure is rehabilitated using track plows and track undercutters, installation of geotextile is an economical method of improving subgrade stabilization and reducing future maintenance costs.

New standardized tests for geotextiles need to be adopted which will conform more nearly to the actual application.

Tensile strength is important to any geotextile, but separation, filtration and particularly planar permeability for rapid release of pore pressures are equal or greater in importance to the stabilization of railroad subgrade.

Des essais pratiques et des applications réalisés par la Companie de Southern Pacific Transportation,ont démontré que les matières géotextiles sont définitivement à considérer dans les travaux de stabilisation de la fondation de la voie. Lors de la remise en état d'une structure de voie ferrée au moyen de pelleteuses et de dégarnisseuses, l'emploi de matières géotextiles représente une méthode économique pour améliorer la stabilisation de la fondation et pour réduire les frais d'entretien futurs.

De nouveaux essais standardisés de géotextiles devront éventuellement être adoptés pour les rendre plus conformes aux applications envisagées.

L'effort de traction est un facteur important pour tout géotextile; toutefois, la séparation, la filtration et en particulier la permeabilité plane pour assurer un rapide relâchement de la pression sur les pores, sont égales ou supérieures en importance pour stabiliser la fondation de la voie ferrée.

INTRODUCTION

Subgrade stabilization has always been a problem to the railroads, and this problem has been intensified with progressive increases in traffic speeds and heavier loads. Train delays due to poor track conditions, and the high costs of maintenance due to increase in cost of labor and materials, have given the railroads justification to invest in testing new methods to improve subgrade stabilization at an economical cost. After approximately 17 years of railroad subgrade stabilization by conventional costly methods, in 1965 I conceived the idea of installing a membrane between the ballast and the subgrade to prevent the ballast from penetrating into the subgrade and subgrade fines from protruding into the ballast. In 1975, geotextiles became known and we at Southern Pacific Transportation Company immediately started testing various geotextiles, and in 1977 our company participated with Monsanto Company in building a highly instrumented test site at Caldwell, Texas, using different types of geotextiles. As of this date, our company has installed over 1600 km (1,000 miles) of geotextiles under railroad track structures. This paper is being presented based mostly on my observations of the effects of geotextiles on railroad subgrade stabilization along with tests and inspection of geotextiles removed from under an operating track. In addition, I have tried to learn something about these geotextiles by observing their manufacture in France, Germany, and in the U.S.A., and in discussing their effects on railroad stabilization with other railroads in France and in the U.S.A.

TRACK SUBGRADE REACTION

To understand some conclusions stated in this paper the type of subgrade reaction should be understood. Railroad traffic produces both high magnitude cyclic and direct vibratory loads onto steel rails that rest on cross ties. These cross ties are then bedded in a layer of uniformly graded, angular, coarse, hard-broken rock known as ballast which produces an elastic foundation. Ballast is compacted only under each rail with no compaction under the center portion of the cross ties to prevent center-bounding, and this type of compaction along with the type of ballast materials results in nonuniform distribution of the load onto the subgrade with the load concentrated primarily under each rail. This type of loading causes a progressive permanent deformation. Characteristically, railroads consist of long, narrow corridors with many varying subgrade materials. This, along with heavy traffic tonnage, causes these deformations to vary in depth. Being open graded, railroad structures allow rain water to collect in the subgrade deformations and small ponds of water form in spots of the subgrade resulting in a completely saturated soil with free water. Dynamic, rapid, repeated traffic loads, transmitted to a saturated fine grain soil subgrade, create very high instantaneous pore pressures. If these pore pressures cannot be dissipated rapidly into a free draining material, the fine grain soil will go into a plastic state or even into a liquid state. This not only results in low shear strength, but allows the ballast to penetrate downward and the plastic, or liquid, soils to migrate upward into the voids of the coarse ballast. This phenomenon progressively becomes worse with time as the deformation becomes deeper, forming ballast pockets. This is commonly known by railroaders in the U.S.A. as "pumping track". The depth of the ballast pockets will depend on the type of subgrade materials, frequency and amount of rainfall, and total traffic tonnage. I have often encountered ballast pockets up to 4 m (15 feet), and some as deep as 8 m (30 feet). A deep ballast pocket can saturate the outer slope or base of an embankment resulting in a shear failure.

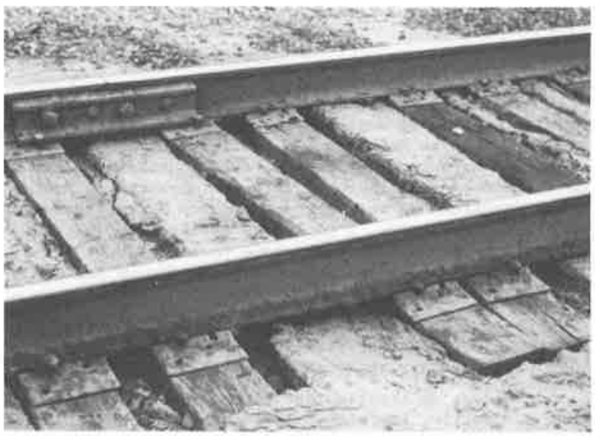

Figure 1. Typical Pumping Track Ballast Completely Fouled

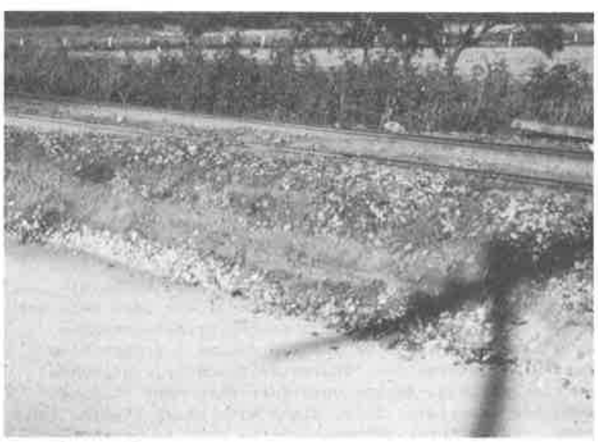

Figure 2. Side View of Mud Migrated Into Ballast

MEMBRANE TYPES

Before geotextiles were known, two types of membrane were tested. One, a 0.45 mm (18 mil) impervious PVC plastic sheet, and the other a 2.54 cm (1 inch) pervious fiberglass blanket compressed and semi-bonded to about 10 mm (3/8 inch) thickness. These membranes were installed in tunnels, under turnouts and in roadway crossings with the materials being overlayed or doubled in thickness under high pressure areas such as under track structures in tunnels and under the heel and the frog of switches. The results were that the PVC plastic sheet failed rapidly due to the fact that the impervious membrane prevented the escape of capillary water, and high pore pressures developed resulting in plastic flows in subgrade material. The pervious fiberglass blanket resulted in excellent track surface without maintenance because pore pressures were dissipated rapidly through the fiberglass blanket.

Figure 3. Fiberglass Mat in Tunnel With Double Thickness Under Track Structure

POLYMER GEOTEXTILES

Our company first started testing various types of non-woven geotextiles in south Texas where long periods of heavy rainfall occur and some of our lowest shear-strength, high plastic, expansive clay subgrades exist.

Geotextiles tested have consisted of either polyester or polypropylene polymer fibers with one having nylon added to polypropylene fibers. Short staple, long staple, and continuous filaments were tested, and the fineness of the fibers have ranged from 0.44 to 1.0 tex (4 to 9 denier). Geotextiles which have been bonded by needle punching, heat bonding, or combinations thereof, have been tested. Weights tested have ranged from 140 to 1400 g/m^2 (4 to 40 oz/yd^2). Woven geotextiles have not been used for reasons that will be explained later.

CALDWELL, TEXAS, TEST SITE

As previously mentioned, the test site at Caldwell, Texas was extensively instrumented. The subgrade consisted of expansive plastic clays and various geotextiles were installed under the ballast in the different sections. The instrumentation consisted of gauges, sensors and meters for measurement from the ball of the rail into the subgrade. As trains travelled through the various sections, these instruments measured rail bending, tie-plate load, cross tie strains, ballast deformation, earth pressure, subgrade deformation, soil moisture, pore pressure, soil expansion, and system dynamic responses. All of this data was fed into a computer where it was stored for analysis. In addition to the instrumentation, alignment and surface track surveys, soil sampling and visual inspection of geotextiles were made. A detailed report on the instrumentations and the test sections was published in American Railroad Engineering Associated Bulletin 678,June-

July 1980, page 361.

INSTALLATION

Geotextiles have been installed under high-speed main tracks, sidings, spurs, etc., and both in rehabilitation of existing track and in construction of new track. Depth of installations have varied from between the ballast and the subgrade with only 15 cms (6 inches) of ballast below base of cross ties to 45 cms (18 inches) below subballast in addition to the ballast. Needle punched geotextiles have been installed under main tracks and, in particular, under switches in two layers with double thickness located under the main track structure. Installations have been in all types of terrain including swamps and mountains, and subgrade soils have varied from very expansive plastic, low strength clays to high strength sand and gravels containing excessive fines.

SUBGRADE PREPARATION

Railroad subgrades were orginally constructed with local native materials without knowledge of soil compaction. These subgrades became compacted over the years by traffic tamping and became somewhat stabilized under light weights and low speed traffic. The maintenance of surfacing and alignment was accomplished by large gangs adding additional ballast. The increase in traffic loads and speeds in the United States during recent years have intensified subgrade stability problems. That is, these heavier loads and greater impacts have increased the stress in the subgrade to a greater depth than was previously stabilized.

When subgrade stabilization of existing, old track is to be performed, consideration is given to the prior stabilization and the addition of only a geotextile under rehabilitated ballast may suffice. If only geotextile is installed for stabilization, some spot failures can be expected due to local subgrade conditions, but these can be avoided by spot stabilization work before or during rehabilitation of track if the spots can be determined ahead of time.

A subballast should be installed on top of the geotextile when geotextile alone will not be ample to provide stabilization. The subballast should be high in shear strength and free draining, but does not necessarily need to be well graded. Depth of subballast required depends on subgrade shear strength, drainage, etc., but 15 to 20 cms (6 to 8 inches) is ample depth in most old existing roadbeds. Very often track structures cannot be raised in elevation to accommodate the addition of subballast or additional ballast due to overhead clearances, roadway crossings, switches., etc. Subgrades can now be lowered to a depth permitting these additions by using modern track plows or track undercutters. It is not possible to obtain a subgrade crown when using a track undercutter, and to eliminate a major negative crown it is necessary to keep the chain on the undercutter taut.

In new construction, the top 20 to 30 cms (8 to 12 inches) of the subgrade is compacted to 95 percent of optimum density. Often, in heavy rainfall areas, a drying agent such as lime or flyash is mixed with the subgrade soils to expedite construction in obtaining the required compaction. After compaction is completed the surface of the subgrade is made reasonably smooth by blade if necessary, and the geotextile is installed as taut as possible with center of geotextile on center line of subgrade. A selected subballast is then placed on top of the geotextile. If the subgrade materials have high shear strength, but contain excessive fine soils that allows 'pumping', only a geotextile providing filtration and separation is required without any subballast.

In all subgrade stabilization work, good drainage should always be provided. This drainage may require the installation of subdrains as well as surface drains.

METHOD OF INSTALLATION

In rehabilitation of track structures, installations have been made by removing the track, and after removal of the fouled ballast, the geotextile installed by unrolling by hand. When a ballast plow is used, it has been installed both directly behind the plow or by a sled after being plowed. The sled can be pulled by a track crane or other similar equipment at a very fast rate. When a ballast undercutter is used, the installation of geotextiles may be accomplished by using a sled, by unrolling geotextiles by hand parallel and near to the track and the track raised with jacks and the materialis then pulled side ways onto the subgrade. Another method used is to raise track with rail tongs attached to a spreader bar and lifted by track crane. In this case, a roll of geotextile can be placed under track structure and unrolled by hand as

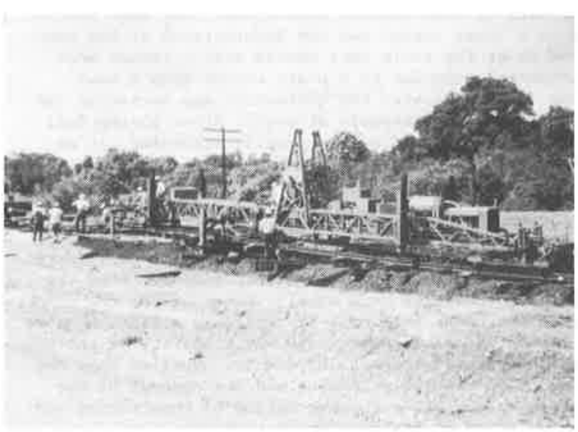

Figure 4. Track Plow Removing Fouled Ballast and Lowering Subgrade

Figure 5. Geotextile Inserted Behind Sled

the crane moves progressively along the track.

In new construction, geotextiles are either unrolled by hand or by equipment. When using the equipment method, a pipe is placed through the center of the roll and each end attached by a chain to mobile rubber-tired equipment. One end is anchored, and the geotextile unrolls as the equipment moves down center of subgrade. The equipment method is very fast and geotextiles can be kept in tension. In all installations, the ends of each roll should be overlapped at least two feet or should be sewed together.

Figure 6 Track Structure on Seotextile After Insertion Behind Track Sled

Subgrade stabilization under turnouts, tracks down streets, roadcrossings and unstable spots are usually performed by removal of the track, and geotextiles are installed in almost all of these locations by hand procedures. A double thickness or heavy weight geotextile is always installed under the high pressure areas even when a subballast is added.

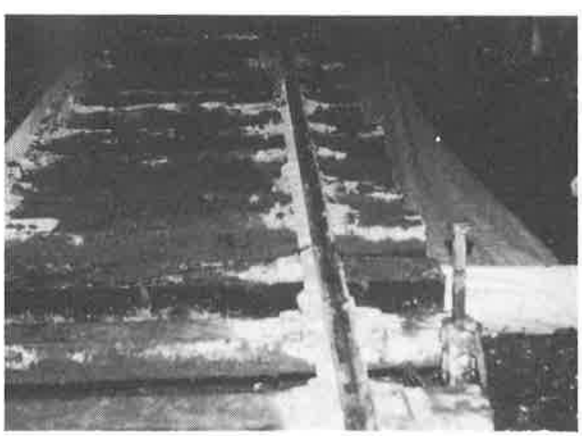

Figure 7. Geotextile Installed by Raising Track with Jacks.

If the geotextile is installed on subgrade without removal of track structure, subballast and ballast may be installed by common method from railroad ballast cars. Tamping of subballast or ballast should not be performed until at least 15 cms (6 inches) is under the base of the cross ties, and then only if the tamping feet are adjusted to prevent driving the ballast through the geotextiles. This is particularly true where lateral ballast resistance occurs such as the side next to another track, group tracks in yards,etc. Full depth tamping will not affect geotextiles if depth of cover below the cross ties is 30 cms (12 inches) or more.

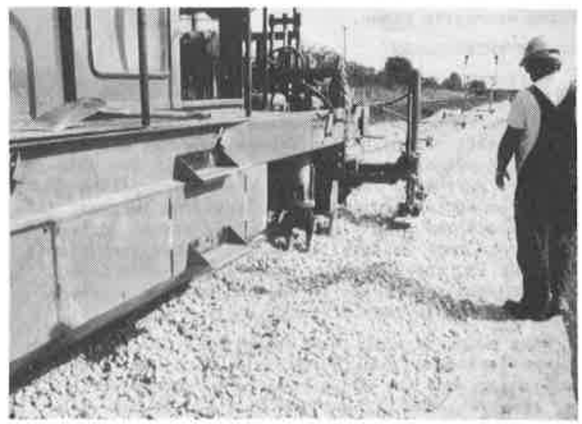

Figure 8. Ballast Tamper Compacting Ballast with Tamper Feet Being Closely Observed for Geotextile Damage.

STABILIZATION FUNCTIONS OF GEOTEXTILE

When I first started using geotextile membranes on railroad subgrade, it was believed that the membrane would only function as a filter and a separator between the ballast and the subgrade. It was soon realized that tensile strength of the geotextile was also contributing to the subgrade stabilization. Tensile stresses are well known to develop when a high modular ballast rests on a low modular subgrade. When ballast was removed from the test sections, the geotextiles were sometimes in high tension even though they were installed in a loose state, and the deformations of the subgrade under the rails were spread over a larger area than normal, similar to a plate rather than a bowl shape. This indicated the geotextile was spreading the load by providing tensile strength. After giving full consideration to geotextile tensile strengths, it was determined that additional stabilization benefits are being provided.

Subgrades were noted to be moist on the outer edge of the ballast where thick needle punched fabrics were installed and dry where heat bonded fabrics were installed. The conclusion reached at that time was that the thicker geotextiles were wicking water out of the deformations under each rail by capillary action. Removal of excess water would increase the stability of the subgrade, but there seems to be greater benefits than the increase in tensile strengths and the removal of excessive water. After further period of observation and study, I came to the conclusion that the rapid dissipation of high pore pressures in the subgrade through the geotextiles was one of the greatest benefits.

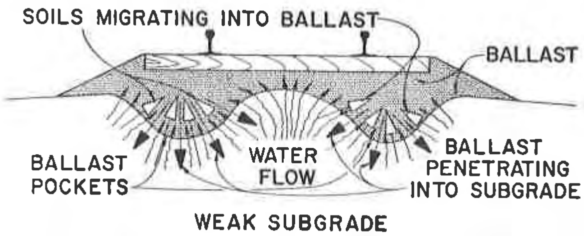

FIG. 9 TYPICAL SECTION WITHOUT GEOTEXTILE

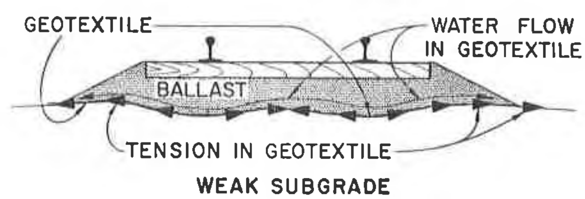

FIG.10 TYPICAL SECTION WITH GEOTEXTILE

Observations revealed cyclic train loads would further compress the geotextile beyond the compression caused by weight of track structure and squeeze out the excessive water toward the outer shoulder where little or no load is applied. This resulted in fast relief or dampening of the peak of high pore pressures in the subgrade. This function of geotextiles is,in my opinion, very important in addition to their filtration, separation, and tensile strengths.

Figure 11. Mud Swirls From Under Track Indicating High Pore Pressures

Woven or heat bonded geotextiles do not have planar permeability in the material and the top surface of this geotextile will become sealed in a relatively short time. This seal consists of crusher rock dust, locomotive sand, wind deposited dust, traffic car dust, dust from abrasion of rock ballast being washed downward through the ballast onto the geotextile by rainwater.

CRITERIA OF GEOTEXTILES IN RAILROAD SUBGRADE

In addition to their functions, geotextiles should meet certain criteria, as follows:

1. A minimum of 60 percent to a miximum of 100 percent of initial elongation to reduce the possibility of puncture by sharp ballast rock and tearing by ballast tampers.
2. Resistance to ultraviolet ray deterioration.
3. Resistance to clogging by fine soils.
4. Control of void openings under loaded conditions.
5. Minimum creep after initial elongation to provide continuous strength.
6. Provide friction between subgrade and ballast.
7. Resistance to abrasion and tear.
8. In some cases, resistance to oils, chemicals and to heat.
9. Flexibility for ease of installation.

SOUTHERN PACIFIC GEOTEXTILE SPECIFICATIONS AND REASONS

After several years of experience on the use of geotextiles, our company's present specifications for the material is "geotextile fabrics shall be engineering grade, non-woven needle punched comprised of gray-colored polyester fibers having a minimum denier of 1.0 mg/m (9) per filament, a minimum fiber length of 15 cm (6 inches), a minimum fiber tensile strength (tenacity) of 4 grams per denier, and shall be packaged in a minimum 200 um (8 mil) black polyethylene film". Weights and widths are specified for each application, but not less than 4 m (13½ feet) in width.

Some of the principal reasons for these specifications are as follows:

1. Non-woven: Because woven geotextiles do not have planar permeability, and it is anticipated that voids would open under tensile stress resulting in penetration of fine soils.
2. Needle punched: Because heat bonded geotextiles do not provide planar permeability and will trap water just under the geotextile resulting in less stability.
3. Gray colored: The darker shade reduces ultraviolet ray degradation effects and reduces the snow blinding effect of white geotextiles.
4. Polyester fibers: Polyester fibers have higher tensile strength, less creep, better diesel fuel resistance, and are less effected by ultraviolet rays than the polypropylene fibers.
5. Filament of 1.0 mg/m (9) denier: High denier filament will provide rapid planar permeability. This is very important in quickly relieving high pore water pressures in the subgrade when railroad loads are applied. An upper limit denier filament should be specified since a very heavy filament would result in loss of filtration and tensile strength, but the upper limit is unknown at this time. We do know that 1.0 mg/m (9) denier will provide filtration, but there may be a small loss in tensile strength compared to a lower denier. Another benefit is that it appears to be less likely to plug with fine soils.
6. Minimum fiber length of 15 cm (6 inches): Equivalent weight geotextiles made with short fibers have less strength and have excessively elongation.

7. Minimum tenacity of fiber to be 4 gms per denier: The high tenacity of the fibers provides more tensile strength at equivalent weight.
8. Packaged in black polyethylene: Aids in prevention of ultraviolet ray degradation during storage.

Weight of geotextile specified depends on the type of subgrade soil, frequency and duration of rainfall, and traffic tonnage. Width specified depends on the depth the geotextile is installed below base of cross ties. For example, if 30 cms (12 inches) of subballast is used plus 20 cms (8 inches) of ballast, we specify a width of 5.2 m (16 feet).

Woven and heat bonded geotextiles are often punctured by sharp ballast due to their stiffness and low elongation. When heavy denier needle punched geotextiles are infiltrated by very fine soils apparently they are being extruded by high pore water pressures. Tensile strength is very important, but of equal or greater importance is the filter, separation and planar permeability. All known needle punched materials provide ample friction between subgrade and ballast, but I have not been able to detect any abrasion of the fabric in our tests. Puncture during installation, or by sharp ballast, is a very serious problem and methods of prevention must always be considered. Geotextiles will aid in keeping subgrade moisture near constant which reduces the shrinkage and swelling of expansive clays.

TESTS OF GEOTEXTILES

It is to be especially noted that test data such as grab tensile, strip tensiles, etc., are not included in our specifications. I do not consider the present ASTM and other standard tests for fabrics to be applicable to geotextiles. New test standards should be developed which will simulate the field conditions in which they are currently being used. A few manufacturers and suppliers have developed some tests which more simulate field conditions, but they have not been adoped as a standard. In any type of test, the geotextile should be completely saturated with water since all geotextiles inspected under field applications were saturated, and it has been noted that water does effect the characteristics of some geotextiles.

MANUFACTURERS OF GEOTEXTILES

Most of the non-woven fabric manufacturers were producing fabrics for other purposes which, in my opinion, were not necessarily the optimum for use as an engineering geotextile. Even the method of needle punch bonding can effect the characteristics of the geotextile such as spacing, size, and number of barbs of the needles. Caution must be taken when any new type of geotextile is developed containing untested filaments, a mixture of filaments, or the addition of additives. Some functions of the geotextile may be improved, but other functions may be lost thereby making the geotextile unsuitable for certain applications. For example, an additive may reduce the planar permeability and certain filaments such as nylon, can have a major strength loss in the presence of water.

Only recently have a few manufacturers started to attempt to produce a geotextile to obtain better stabilization. The main problem was that the fabric expert did not know engineering applications, and the engineer did not know fabric characteristics. However, in working together better geotextiles are beginning to be produced. In fact, there are now companies producing only geotextile fabrics and they are making designs for specific applications. One such application is a package for different sizes of railroad turnouts, or switches, with the thicker geotextile under the high impact pressure points. These extra thicknesses are

Figure 12. Geotextile Turnout Package Being Installed under switch.

then needle punched together which no doubt is better than two separate thicknesses. In addition, the packages are being manufactured to fit the shape of various switches which eliminates waste of the geotextile. However, I must say that we have not detected any adverse effects of the two-layer system. Continual improvements in geotextiles can be expected as both the engineer and the fabric producers become more knowledgeable in geotextiles and their uses.

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Behavior of Woven Fabrics Under Simulated Railway Loading

Comportement d'un géotextile tissé sous l'action représentant une charge de poids de chemin de fer

SUMMARY

The effects of placing a geotextile between a railway base course and a fine-grain subgrade were evaluated by performing a series of cyclic triaxial tests. Each test specimen was subjected to 5000 cycles of simulated railway car wheel loading; resilient modulus and permanent strain were recorded per cycle of loading. Two geotextiles were used during the program: PPS45 and Propex 6062. Both are woven fabrics comprised of polypropylene oriented split film. Tests were also conducted in which a 3 cm layer of sand was placed between the geotextile and the fine-grain soil. Results of the test series showed that the resilient moduli of samples with the geotextile were higher by 10 to 50 percent than if the geotextile was not included. The accumulated permanent strain decreased by 30 to 60 percent when the geotextile was introduced. Finally the geotextile reduced the amount of fine-grain migration into the base course.

SOMMAIRE

Les effets du placement d'un géotextile entre un ballast de chemin de fer et un sol de fondation cohérent ont été evalués en faisant une série d'essais cycliques triaxiaux. Chaque échantillon a été soumis à 5000 cycles de chargement typique d'un roue de wagon de voie ferrée; le module de rebondissement et la deformation permanente ont été enregistré pour chaque cycle de chargement. Deux géotextiles ont été utilisés dans le programme: PPS45 et Propex 6062. Les deux tissus étaient fait de pellicule de polypropylene coupée et orientée. Plusieurs essais ont été fait avec une couche de sable de 3 cm intercalée entre le géotextile et le sol cohérent. Les résultats de la série d'essais ont montré que le module de robondissement du sol avec le géotextile était plus élévé (10 à 50 pourcent) que sans l'utilization du géotextile. La déformation permanente accumulée a baissé de 30 à 60 pourcent quand le géotextile a été utilisé. Enfin le géotextile a diminué la quantité de migration du sol dans le ballast.

INTRODUCTION

During the past 100 years, railways have been located in virtually every climatic condition and on virtually every type of soil. The rail-tie system was originally placed directly on native soil with little consideration given to soil behavior. Subsequently as rail loads and track speeds increased and as maintenance costs rose, more refined analytical procedures involving basic concepts in geotechnical engineering evolved. Today railways tracks are constructed on composite foundations which are dimensioned to satisfy various stress and deformation criteria (1, 4, 7, 8).

One of the foremost considerations in the current design of railway foundations, such as shown in Figure 1, is the migration of fine-grain native soils into the base course and ballast, a process which is commonly called fouling (7, 8). Fouling of the ballast and base course results as dynamic wheel loads "pump" the fine-grain material upward through the pore spaces in the coarser material. This pumping phenomenon is particularly prevalent in silt-size soils, which are fine enough to migrate through the base course and ballast and which have little or no unconfined shear strength. As the ballast and base course are fouled with fine-grain soils, the track undergoes larger deformations for given wheel loadings. Secondary considerations such as drainage and frost susceptibility also become more critical (7). Eventually significant maintenance costs may be incurred.

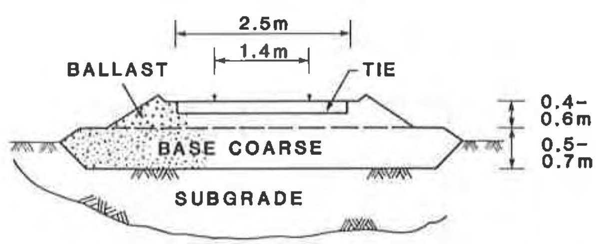

FIGURE 1 TYPICAL EUROPEAN TRACK SECTION

The most common method for preventing migration of fine-grain soils involves placement of a sand foundation layer between the fine-grain soil and the base course. The thickness and characteristics of the sand layer are selected to satisfy certain filtering criteria. This design method can involve significant construction costs, and therefore, an attempt is made to either minimize the thickness of the sand or even eliminate it.

Another potential approach to solving the migration problem involves the use of geotextiles (2, 5, 6). The geotextile offers two significant benefits: 1) it can be used as a filter between the fine-grain soil and the base course and 2) it provides some additional reinforcement to the base course during wheel loading. Furthermore the cost of the geotextile on a unit area basis can be competitive with a sand filter if labor, material and long-term maintenance costs are considered.

Although the potential benefits of using the geotextile are fairly evident, acceptance by the rail industry has been slow. This slowness can be attributed to the limited data base upon which to judge the benefits. In an effort to enhance this data base, a laboratory testing program was conducted for the purpose of investigating the effects of geotextiles on 1) fine-grain soil migration and 2) base course strength during dynamic wheel loading.

TESTING CONCEPT

A wheel-foundation interaction problem is very complex, by virtue of the foundation geometry and the transient wheel loading if nothing else. Consequently a number of simplifications were made during this study to investigate the effects of geotextiles on soil migration and base course strength. Although some of these assumptions may limit the applicability of the results, it is felt that the trends obtained from the testing program provide valuable support for the use of geotextiles as a means of preventing fine-grain soil migration and increasing base course strength.

The first simplification involved the use of a two-layer triaxial specimen to represent conditions at the base course-subgrade interface (Figure 2). Whereas the triaxial sample does not provide an exact replication of the field, particularly from a scaling standpoint, it offers a simple means for evaluating the effects of repeated loading cycles under known stress and drainage conditions. For this study the upper half of the specimen was the base course material; the lower half was the fine-grain subgrade. As noted previously, silts have been most susceptible to migration; hence a silt was selected to represent the subgrade.

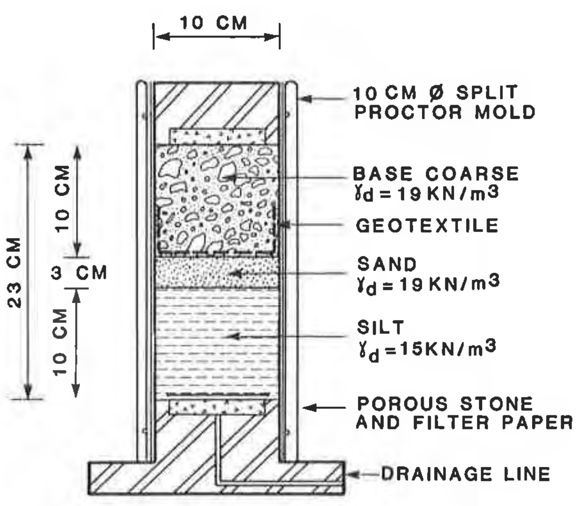

FIGURE 2 CROSS-SECTION OF TRIAXIAL TEST SPECIMEN

Another important consideration is the water conditions within the foundation system. The migration problem has been most noticeable when the subgrade is saturated. The ballast, in turn, is generally very porous and above the natural ground water level, and hence, is normally in a drained condition. For this study it was assumed that the silt would be soaked, with a phreatic head at the level of the base course-silt interface.

The next simplification involved the stress conditions under geostatic and transient wheel loading. The geostatic stress at the base course-silt interface was assumed to be 35 kN/m^2 based on the unit weight of the ballast and the dead-weight loading from the tie and rail. The transient loading was represented by a half-sine wave (Figure 3) with a maximum transient stress of 50 kN/m^2. This stress level corresponds to the passage of an axle load of approximately 90 kN at a depth of 50 cm below a concrete tie (7). The period for the transient loading was 0.8 seconds. This period would result from a railway car travelling at about 40 km/h for an average axle to axle spacing of approximately 8 m. This loading rate was less than might be expected for many situations; however, it was thought that it would maximize the opportunity for pumping (i.e., faster loading would be completely undrained).

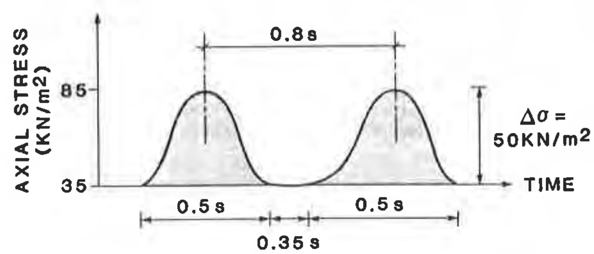

FIGURE 3 TYPICAL WAVE FORM FOR TRANSIENT LOADING

SOIL AND GEOTEXTILE PARAMETERS

Three soils were used during the course of the investigation: a coarse foundation material (base course), a fine to medium sand and a silt. Grain-size distribution curves for each soil are presented in Figure 4. The silt was slighty plastic; the liquid limit, plasticity index and natural water content (during testing) were 42, 15 and 20 percent, respectively. The sand was used during three tests as a supplemental filtering material. The grain-size distribution of the sand was typical of sands used during railway construction. To maintain a satisfactory ratio of particle size to sample diameter, the maximum particle size of the base course was limited to 15 mm.

Two geotextiles were used during the testing program: Propex 6062 and PPS45. These geotextiles are designated as Geotextile 1 and Geotextile 2, respectively. Both are woven fabrics comprised of polypropylene oriented split film. The characteristics of each geotextile are summarized in Table 1. The two geotextiles are generally similar in characteristics. Different tensile strengths and permeabilities result from the different weaving patterns.

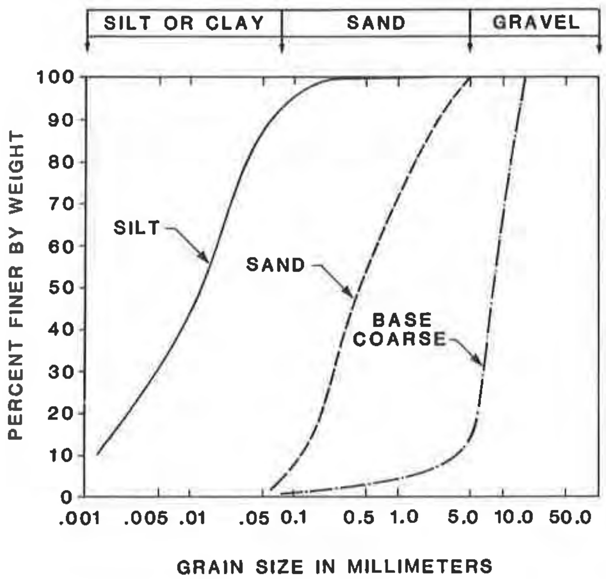

FIGURE 4 GRAIN-SIZE DISTRIBUTIONS OF TEST MATERIALS

TABLE 1. SUMMARY OF PROPERTIES FOR GEOTEXTILES

	Geotextile 1	Geotextile 2
Equivalent Opening Size (95)	0.11 mm	0.17 mm
Percentage Open Area (Est.)	7%	5%
Thickness at 20 kN/m^2	0.50 mm	0.37 mm
Weight	190 g/m^2	140 g/m^2
Tensile Strength	1700 N/5cm	1125 N/5cm
Failure Strain	16%	12%
Permeability	17 L/m^2s	10 L/m^2s

TESTING PROCEDURE

As shown in Figure 2, the top of each test specimen was a base course, and the bottom was a silt. Methods used in preparing, testing and evaluating samples are described below.

Sample Preparation. Each sample was prepared by compacting the silt in a 10 cm diameter by 23 cm high Proctor split mold using a 2 kg hammer. Approximately 25 blows per layer were applied; layer thicknesses were about 1.0 cm. Once the midheight of the specimen was reached, about 10 cm, one of the following four procedures was followed.

1) A 3 cm layer of sand and a layer of geotextile were placed on the fine material and then the coarse material was compacted using the same procedure (Tests 1 & 2).

2) A 3 cm layer of sand was placed on the fine material after which the coarse material was compacted (Tests 3).

3) A layer of geotextile was placed on the fine material and then the coarse material was compacted (Tests 4 & 5).

4) The coarse-grain material was compacted directly on the fine material (Test 6).

The unit weights of the silt, sand and base coarse after compaction were approximately 15, 19 and 19 kN/m^3, respectively. Figure 5 provides a summary of the test cases.

SETUP						
GEOTEXTILE	1	2		1	2	
TEST NO.	1	2	3	4	5	6
SYMBOL	○	□	⬡	◇	▽	△

FIGURE 5 TEST CASE SUMMARY

After the specimen was tamped to its 20 to 23 cm height, it was weighed and then transferred to a triaxial test cell. Each sample was "capped" with a plaster compound to assure a flat sample top. The test specimen was then consolidated overnight under a pressure of 35 kN/m^2. This duration of confinement was more than sufficient to allow complete consolidation of the fine-grain soil. A phreatic head was maintained at the top of the silt layer throughout consolidation and testing.

Sample Testing. Each sample was subjected to 5,000 cycles of loading. An MTS cyclic loading device was used to apply the loading sequence. The MTS device was operated in a controlled-stress mode to impose a constant cyclic axial stress. A load cell mounted just above the triaxial chamber was used to monitor cyclic loads. Sample deformations were measured with an LVDT (linear variable differential transformer). During cyclic loading sample drainage was closed at the bottom and open at the top of the test specimen.

Load and deformation measurements were recorded with an oscillograph (light-beam type) and an x-y recorder. From these measurements it was possible to define plastic creep and elastic rebound during each loading cycle. These data were used to define the resilient modulus for each cycle of loading. By definition (8) the resilient modulus is the ratio of the cyclic deviator stress divided by the recoverable portion of axial strain (Figure 6).

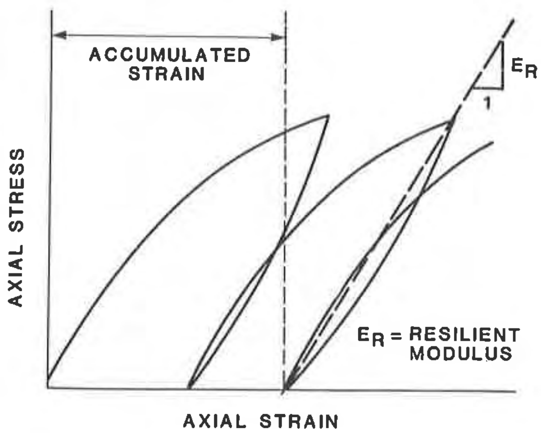

FIGURE 6 IDEALIZED SOIL RESPONSE

Post-Test Evaluation. At the conclusion of each test, the test setup was disassembled and the sample was carefully inspected. Grain-size analyses were performed on several samples to determine if particle breakdown or fine-grain soil migration had occurred. For Tests 1, 2, 4 and 5 the woven fabric was inspected with a microscope to determine if any evidence of fabric deterioration existed.

TEST RESULTS

Resilient modulus values (E_r) obtained during this testing program are plotted in Figure 7. This plot shows both the variation of E_r with cycles and with different interface conditions. It is clear from these results that resilient modulus generally decreased with cycles of repeated loading and that for any given number of load applications a consistently higher resilient modulus occurred when the geotextile was used. A noticeable difference in resilient modulus also resulted when a thin layer of sand was placed between the base course and the silt and between the geotextile and silt.

Accumulated plastic strains for the six cases are plotted as a function of loading cycle in Figure 8. This plot shows that plastic strain increased with the number of loading cycles. However, the plastic strain for a test specimen with a geotextile was always smaller for a given number of load applications than if the geotextile was excluded.

Visual inspection of the soil specimens following Tests 3 and 6 revealed that some fine-grain soil migration occurred. As would be expected, migration was reduced when a geotextile separated the two materials (Tests 1, 2, 4 and 5). There was no evidence of geotextile deterioration from the repeated cycles of loading.

DISCUSSION

Several trends can be noted from the data shown in Figures 7 and 8. First the rate of resilient modulus decrease appears to end after about 1000 to 5000 cycles of loading for samples containing the geotextile. No change in the rate of decrease seems to occur for samples without the geotextile. This response suggests that the geotextile ultimately may prevent further deformation. The plastic strain plots in Figure 8 tend to support this observation, i.e., accumulated strains for tests with the geotextile appear to stabilize (must be inferred for Tests 1, 2 and 4). This probably reflects an incremental stiffening of the sample by the geotextile wherein the sample can no longer undergo unrestricted lateral deformations (3). As deformations are restricted, the volumetric change per cycle of loading decreases, and hence, the asymptotic behavior.

The effects of geotextiles on resilient modulus and accumulated strain also form a consistent response. Resilient modulus for a given number of load applications is always higher when the geotextile is included. Likewise accumulated strains are smaller by up to 60 percent when the geotextile is used. The sand layer between the geotextile and the silt causes a noticeable increase of moduli. This latter result suggests that more resistance is developed on a sand-geotextile interface than on a silt-geotextile interface, which may in turn reflect a higher coefficient of friction for the sand-geotextile case.

Different characteristics of the geotextiles also led to different resilient moduli and accumulated strain. Moduli are lower, and accumulated strains are higher for the more flexible geotextile (PPS45). Again, the difference in specimen response is attributed to the different engineering properties of the geotextiles.

CONCLUDING REMARKS

The results of this testing program were by necessity limited. A number of questions still exist regarding the behavior of geotextiles during cyclic loading. For example the effects of larger stress pulses certainly merits additional consideration. The effects of loading rate and millions of cycles of loading would also be of interest. Despite these questions, the testing program successfully demonstrates that a geotextile can be used to increase the resilient modulus of a base course-subgrade system by as much as 10 to 50 percent for typical railway ballast loading conditions. Accumulated strains in turn could be 30 to 60 percent lower. By placing a geotextile between the subgrade and base course, some reduction of fine-grain soil migration also results.

These three observations support the use of geotextiles in railroad ballast design. A reduction in the accumulated strain is particularly important in railway track design in that the formation of ballast pockets in proximity to the rails would decrease. Water tends to accumulate in these ballast pockets, thereby reducing the supporting strength of the subgrade material and causing deterioration of track supporting capacity. The increase in foundation stiffness may also allow for a reduction of the thickness of the ballast or base course. Finally the reduction in fine-grain soil migration, particularly when combined with a thin sand blanket, should lead to better long-term behavior of the track-foundation system.

Although these results are promising, additional studies are still required to confirm trends noted herein and to investigate various uncertainties. The main uncertainty deals with the effects of millions of cycles of loading under a variety of environmental conditions, particularly temperature fluctuations. In

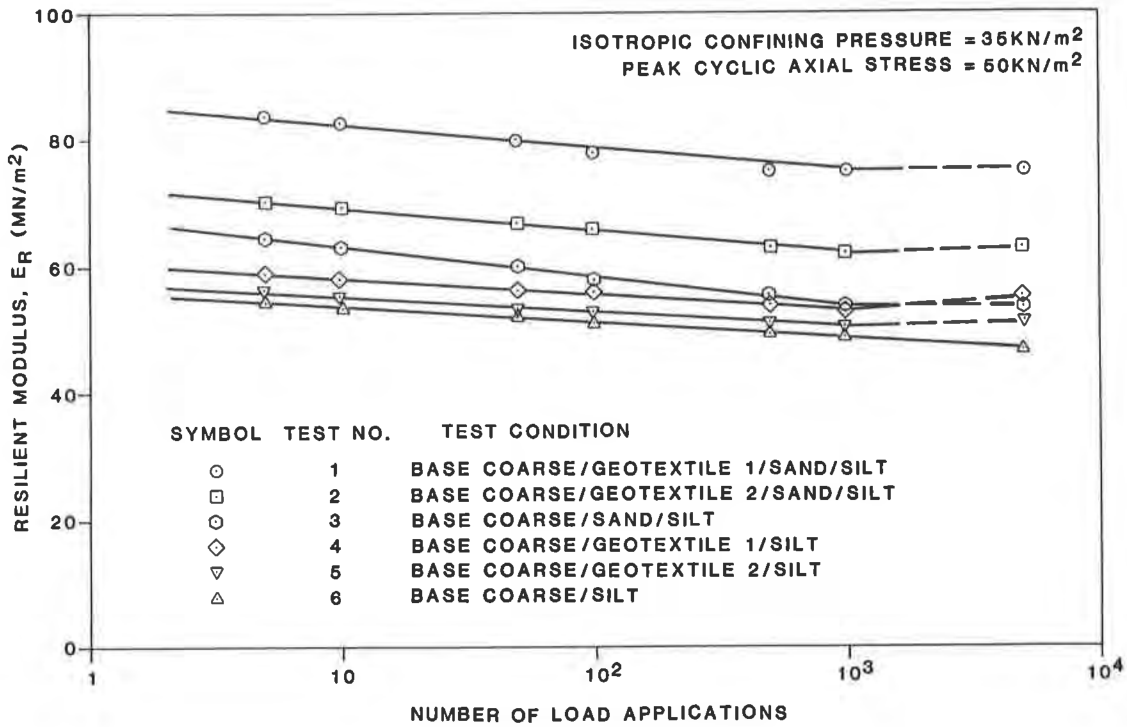

FIGURE 7 RESILIENT MODULUS VARIATION WITH NUMBER OF LOAD APPLICATIONS

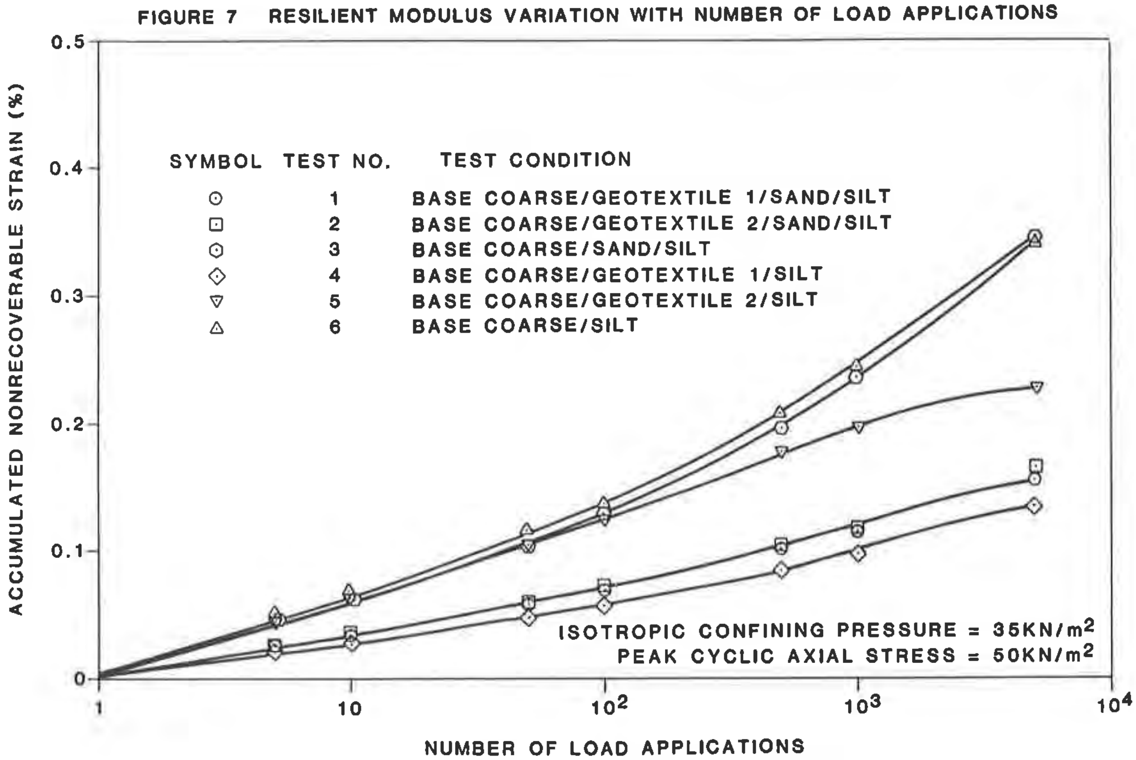

FIGURE 8 ACCUMULATED NONRECOVERABLE STRAIN VARIATION WITH NUMBER OF LOAD APPLICATIONS

all likelihood this uncertainty will not be completely resolved in the laboratory but will require long-term field experimentation. Inasmuch as the benefits of the geotextile are potentially significant, such field experiments are highly recommended.

ACKNOWLEDGEMENTS

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Geotextiles for Railroad Bed Rehabilitation

Les géotextiles pour la rehabilitation de base du chemin de fer

A wide range of geotextiles provide significant short term improvements to track support problems, however; for long term applications a very much greater discrimination is required. Discussed is the importance of geotextile manufacture, fiber characteristics and polymer type along with correct construction or installation procedures and the matching of the soils engineering properties with the geotextiles characteristics as essential for geotextile longevity in rehabilitated track.

In dealing with long term performance of the geotextile-soil system the geotextile must maintain its ability to separate, filter and drain laterally. These properties must be maintained despite an extremely harsh environment of abrasion from sharp cornered crushed stone.

Une large variation du géotextile fourni de significante courte terme perfectionnement aux problèmes de pisté, quand même, pour la longue terme application, une trés grande discrimination est demandée. Discussion est le plus important de fabrication gèotextile, les caractés des fibres et le genre polymé avec de construction correct au marche d'installation et le matche des proprietés de gèotecnique avec les caracteristics est essentiel pour le longtivité du géotextile dans le piste rehabilitè.

En amenagent avec le rendement de longue terme du terre gèotextile système, le géotextile doit maintenu son abilité de separer l'ecran et le chain lateralement. Ces propretés doivent être maintenu même si un extrement rude environs l'abrasion du rocher craché aiguiser.

INTRODUCTION

In the last two decades there has been a rapid employment of geotextiles in civil engineering. More recently considerable interest has been shown by the railway industry in the use of geotextiles during track rehabilitation. Continuous increases in maintenance costs and in operating costs through train delays have given impetus to research on improving track support stabilization of which the use of geotextiles is but one option. This option has, unfortunately, not met with universal acceptance; both good and bad examples have been cited by various track engineers.

Because of inconsistent performance ratings from regional engineers a research project was sponsored at Queen's University under the direction of the Author by CN Rail, CP Rail and TDC of Transport Canada through the Canadian Institute of Guided Ground Transport.

PURPOSE OF GEOTEXTILES

Geotextiles used in track rehabilitation, as opposed to new construction, are primarily used to prevent piping of fines (from the subgrade, subballast or dirty ballast below the undercutting elevation) from fouling the cleaned replaced or clean make up ballast. The phenomenon of piping fines infers the presence of excess water which, when subject to repetitive wheel loads, pumps upwards through the track structure under high hydraulic gradients. The more free water there is the faster, in general, the upward migration of fines. Thus drainage improvement (whether the improvement of side ditch drainage dealing with surface water, or the lowering of the ground water to increase the subgrade strength, or drainage of the track area to prevent water seepage into the subgrade bearing area) is always the first and most essential item in any subgrade stabilization work. Thus a geotextile as well as preventing migration of fines should also facilitate the drainage of excess water from the track area to the side ditches.

Geotextiles have found their greatest use in poorly drained regions of the terrain which are flat and possibly marshy. These areas are generally beyond the economic ability of the railways to drain because of the general regional high water table. Geotextiles are also being used at locations which are difficult to maintain such as grade crossings, diamonds, switches and like track structures by facilitating track drainage at these locations.

SITE EVALUATION OF GEOTEXTILES

A typical excavation made to assess geotextile performance at a grade crossing in a flat low lying flood plan, is shown in Fig 1. The excavation is shown extending down to the water table at about 950 mm (37 inches) below the ties. Some sand subballast penetration through the geotextile may be seen below the rail seat where the geotextile was found to be worn by abrasion. The ballast at the center of track, where the geotextile is intact and performing to the best of its ability, is relatively clean. The poor drainage of the terrain with high water table makes this locality

FIGURE 1 TYPICAL GEOTEXTILE EVALUATION EXCAVATION

an excellent example of good site selection for geotextile usage.

One of the main observations for good long term performance of an installation was the geotextile's ability to conduct water laterally, particularly when the ballast immediately above the geotextile begins to be partially fouled. In general woven and in particular monofilament woven geotextiles do not exhibit this ability to hydraulically conduct water within the thickness of the geotextile. Woven geotextiles, with time, become ineffective and act almost as a plastic sheet to prevent water from draining out of the subgrade. This is seen in Fig 2 for a woven geotextile after 1-1/2 years service life. The trapped water weakens the subgrade soil causing it to squeeze laterally from the area beneath the cross-ties. The track gradually subsides. Thus after 2 years service life the geotextile area shown in Fig 2, on excavation, had become that shown in Fig 3. This subsidence is accompanied with a rise in the shoulder material, as seen in Fig 4 which was photographed on the same day as Fig 3, further aggravating drainage from the track to the side ditches. Thus, with time, water becomes trapped in the track area, facilitating the weakening of the track subgrade bearing area.

Based on graded granular filter requirements the geotextile should have an EOS smaller than the size of the largest fifteen percent by weight of the particles

FIGURE 2 SLURRY BUILD UP BELOW WOVEN AT 18 MONTHS

FIGURE 3 SUBSIDENCE ABOVE WOVEN AT 23 MONTHS

FIGURE 4 TRACK AT FIG. 3 GEOTEXTILE LEVEL

of the soil to be separated. In practice it is found that the soil particles will push their way into a geotextile dramatically increasing its in-track EOS. This is illustrated in Fig 5 which shows a geotextile, taken from track, ultrasonically cleaned, and photographed in front of a bright light to show the particle penetration and abrasion holes beside an unused sample. Geotextiles for track rehabilitation should therefore have an EOS at least as small as the lower limit of fine sand (74 microns or a No. 200 sieve) and preferably an EOS less than 38 microns (No. 400 sieve). Particles less than the EOS generally intrude freely and particles of larger size are forced by the repeated loading of passing axles, into the geotextile. These particles lower the permeability of the geotextile and also act to abrade the individual fibers internally.

Clearly if particles are being forced into the geotextile opening up the random fibre structure any system of fiber bonding, provided it does not foul the opening size, should be beneficial to the geotextile performance. It is not surprising then that those geotextiles that had been resin dipped and forced air dried (to maintain their porosity and permeability) were observed to out-perform, both in the field and the laboratory, geotextiles of similar weight and construction which had no bonding.

The single most important conclusion regarding geotextile manufacture would be, in the writer's opinion, that geotextiles for use in abrasive environments should be resin dipped.

It might be thought that if resin bonding was found to be of such value then heat bonded geotextiles would also perform well. Unfortunately this was not found to be the case. The heat bonded geotextiles were found to very easily foul and became impermeable exhibiting, as seen in Fig 6, characteristics similar to woven geotextiles (compare Fig 6 with Fig 4). The heat treatment appears to flatten the fiber matrix, as seen in Fig 7, making the performance much like that of woven fibers. With time the geotextile loses its most important long term requirement, that of lateral hydraulic conductivity.

FUNCTIONAL REQUIREMENTS OF GEOTEXTILE

Based on the field study it may be stated that the basic functional requirements of geotextiles for railway track rehabilitation are

(a) To drain water from the roadbed, by gravitational forces, along the plane of the geotextile without build up of excess hydrostatic pressures.

(b) To withstand the abrasive forcing of moving aggregate caused by the passage of trains on a frequent basis and also caused by the tamping compaction during installation and cyclic maintenance.

(c) To filter or hold back soil particles while allowing the passage of water.

(d) To separate soils of different sizes and gradings which would readily mix under the loading and drainage environment.

(e) To resist the chemical environment of a railway track bed subject to locomotive diesel fuel and herbicides for vegetation control.

Since drainage of excess water from the roadbed is so important to roadbed stability sufficient granular depth and subgrade drainage must be provided to ensure that rutting does not occur, thus tensile reinforcement is not a basic functional requirement of the geotextile in track rehabilitation.

FIGURE 5 COMPARISON OF NEW AND USED GEOTEXTILES

FIGURE 6 HEAT BONDED 280 g/m^2 (8 oz/yd^2) AT 23 MONTHS

FIGURE 7 MICROSCOPIC VIEW OF HEAT BONDED GEOTEXTILE

REQUIRED PROPERTIES OF GEOTEXTILES

In order to perform its functional requirements a geotextile used in track rehabilitation will require certain measurable properties which may be used as a selection criteria. The most important of these are
(a) The hydraulic conductivity in the plane of the geotextile; generally met by all thick non-woven geotextiles.
(b) The equivalent opening size to prevent or limit fouling; generally less than 40 microns or No 400 sieve size.
(c) The ability to resist abrasion from moving ballast particles; generally helped by resin-dipping.
(d) The ability to resist impact damage due to ballast placement; generally helped by resin dipping.
(e) The ability to resist chemical deterioration due to the track environment; generally helped by resin dipping and by selecting Polyester as polymer type.

GEOTEXTILE TYPES

There are two major divisions of geotextile manufacture namely woven and non-woven. Because of the importance of the in-plane hydraulic conductivity of a geotextile only non-woven types are considered suitable for track rehabilitation. The main non-woven geotextiles in use and their performance is summarized as
(a) Heat bonded geotextiles which are observed to foul easily on a long term basis. They were also found to have low impact resistance and cut easily.
(b) Needle punched geotextiles without resin dipping which were observed to have low abrasion resistance in comparison with needle punched resin dipped geotextiles of the same mass per unit area and fiber type.
(c) Needle punched geotextiles which are resin dipped and force air dried to ensure good porosity. These geotextiles have proved to be the most resistant to abrasion, impact and the track environment.

POLYMER TYPE

While there are many fiber types available three are in common use in North America for railway application. These are (a) Polypropelene, (b) Polyamide, (c) Polyester. Note - Polyethelene is commonly used outside North America.

The main factor which should be considered in the selection of fiber type is its ability to remain stable under the environment created by railway operations. Little work appears to have been done on the long term effect of locomotive diesel fuel or vegetation control herbicides such as 2-4-D. Evidence from the field, which was by no means extensive, and in the laboratory suggests that Polyesters remain more stable than the other two polymers, although no discernable environmental deterioration was observed in the field for any polymers that had been coated through a resin-dipping process. The beneficial effect of resin-dipping to Polypropelene and Polyamide was not as evident in the laboratory work where the geotextiles were allowed to repeatedly soak and air dry using various fluids. Polyester clearly performed best in the laboratory.

FIBER DENIER

In order to obtain a small geotextile pore size or EOS a low denier fiber is required. On the other hand the larger the denier of the fiber the more resistant the fiber will be to abrasion and cutting. The best denier of fibres for their best collective behaviour is still unknown. There appears to have been a tendency for the fiber denier of non-woven geotextiles to have decreased with time and presumably with observed performance with 6 denier fibers being the most common usage today (1982). With increased knowledge it may be possible to make multilayered units using different denier fibers in each unit of the geotextile. Low denier fibers being used on the outer layers to prevent particle intrusion and large denier fibers on the inner layer to give high porosity and thus high permeability. Under no circumstances should two unattached layers of geotextile be used adjacent to each other since this would allow fines to accumulate between the two geotextiles. On wetting these fines become very weak. At least one derailment has been related to this condition where a geotextile was laid as a double thickness over a considerable distance.

BURIED DEPTH EFFECT ON GEOTEXTILES

One of the contributing factors to a geotextile's performance in track is the depth at which it is placed below the base of the ties. While it is well known that the intensity of pressure generated from a standard wheel load is distributed through the track structure decreasing with depth, it is also known that abrasion depends on the deformations occurring at the geotextile level and thus subgrade modulus. This is to a large extent controlled by track drainage since soil behaviour is very much dependent on soil suction and moisture content. Thus as a first comparison six sites installed with needle punched resin dipped geotextiles all having unit area mass (weight) between 450-510 g/m^2 (13 - 15 oz/yd^2) were examined. The estimated percentage of geotextile damage for each sample of geotextile was obtained by measuring the area of the enlarged holes in the worst 300 mm (ft) square section (generally close to the rail seat) to obtain the percentage worn through area of the geotextile. The results are shown in Fig 8. The values range from 0.3 percent at a depth of 350 mm (14 inches) to 4.1 percent at a depth of 175 mm (7 inches), clearly indicating that as the depth of ballast between the ties and the geotextile is reduced the amount of damage that the geotextile receives is increased.

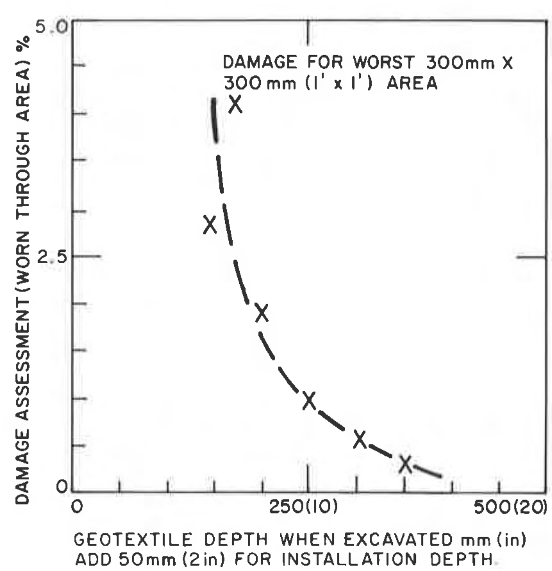

FIGURE 8 OBSERVED GEOTEXTILE DAMAGE VS BURIED DEPTH

The majority of the plotted values indicate a linear drop in percent damage with respect to depth down to a depth of 250 mm (10 inches). Below 250 mm, the rate of change in percent damage is considerably decreased which suggests that a large portion of the damage occurring to these types of manufactured geotextiles excavated from 250 mm (10 inches) and greater below the base of the ties originated during the installation period of repeated tamping and raising track in small lifts and not during the in-service life.

Note that some of these geotextiles have been in service for up to 5 years so that 50 mm (2 inches) of track settlement may have already occurred, particularly as all the ballast above the geotextile must have been freshly placed or replaced. A comparable installation depth should be taken as 300 mm (12 inches) or greater.

INSTALLATION OF GEOTEXTILES

In the planning of any geotextile installation the execution of good drainage practices should be considered of paramount importance. Whether the surface below the track is sledged, ploughed or undercut it is desirable to have the freshly prepared track bed surface with a slope of at least 48:1. Figure 9 illustrates some of the points requiring attention during installation which are commented on below.

If a track gopher undercutter with trencher is used to prepare the track roadbed surface the drainage should be towards the trench side of the track which in double track territory should be on the ditch side. The prepared trench should be kept 150 mm (6 inches) or more deeper than the prepared track bed surface and the fouled shoulder material removed to below the bottom of the trench.

At grade crossings or other areas where drainage must be along the trench parallel to the track, at least for some short distance, then a small perforated pipe is recommended to facilitate drainage. The pipe should be laid directly on the geotextile which itself will be laid on the prepared roadbed and trench bottom. The perforated pipe should be taken to the ends of the grade crossing and then bent perpendicular to the track direction and extended to drain directly into the surface side ditches or drainage facilities. Geotextiles are often bent upwards through grade crossings to prevent lateral migration of particles from the highway pavement support. Once the geotextile has cleared the pavement area it should be turned down, as shown in Fig 9, to allow gravitational drainage of water through the shoulders of the track immediately each side of the crossing. Again a small perforated pipe, properly installed, will facilitate drainage and add to the geotextile's usefulness.

Geotextiles, once in place, must be covered with 200 mm (8 inches) or more of ballast prior to first tamping otherwise the tamper tines, which penetrate about 75 mm (3 inches) below the ties and cause excessive movement of ballast to about 150 mm (6 inches) below the ties, will cause the vibrating ballast to abrade holes in the geotextile.

If the geotextile is of sufficient quality to withstand the harsh track environment and has been correctly installed then ballast fouling occurs either by ballast degradation or through the ballast surface from foreign sources. Clearly good quality ballast aggregate is a desirable asset for a geotextile installation even if only applied over the short lengths of track where the geotextile is installed.

In order to allow for future undercutting of a track and also to reduce the abrasive forces on a geotextile it is highly beneficial to the longevity of the geotextile to be placed as deep as possible. On the other hand, track undercutting costs increase rapidly with depth. As a compromise, a 300 mm (12 inch) depth below the ties appears to be reasonable and acceptable based on observed installation and damage depths. This depth would be deep enough to allow future shallow undercutting without having the geotextile cut into small pieces. These small pieces of geotextile can become caught in the undercutter's screens turning the geotextile into a detriment rather than asset.

COST-BENEFIT OF GEOTEXTILES

Any cost-benefit of a geotextile installation will need a site specific analysis however where they are used to protect the ballast below switches and

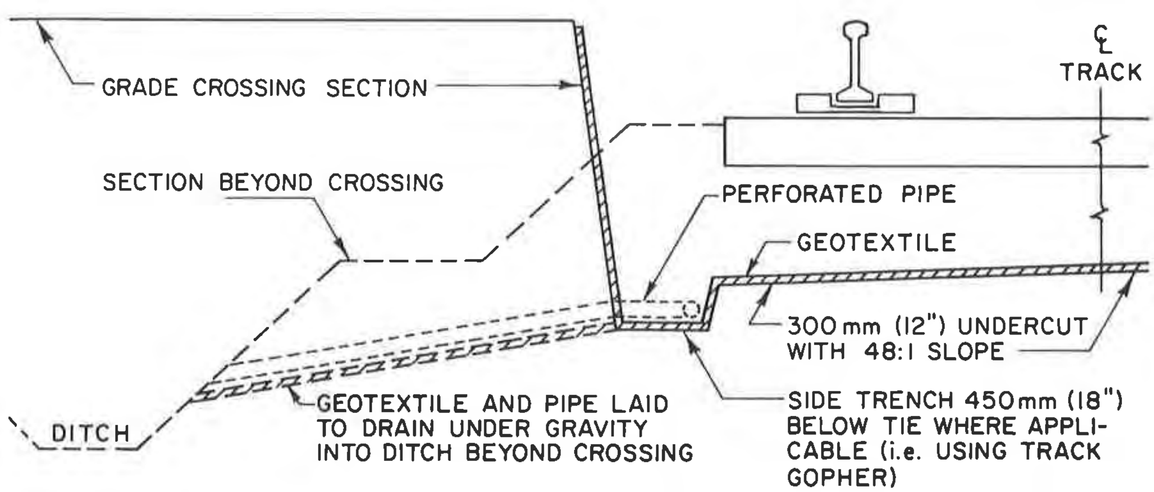

FIGURE 9 GRADE CROSSING GEOTEXTILE INSTALLATION

diamonds some general statements are possible. Switches, diamonds and like track structures cost today (1982) in the order of \$25,000 to \$100,000 each depending on size and any underlying geotextile would cover 25 to 100 m^2 (250 to 1000 ft^2). Even if a geotextile as expensive as \$10 a square metre (\$1/ft^2) were to be selected it should only add about one percent to the cost of the installation. In poorly drained terrain such a percentage cost is likely to be negligible compared to the savings from the reduced maintenance and added life of the specialized track structure. Geotextiles properly selected and installed are therefore likely to be of immense beneficial cost savings.

It is worth recording that on every site where the writers extracted a geotextile, the track foremen responsible for the territory stated, without exception, that they had observed less tie maintenance in track having underlying geotextiles as opposed to track without geotextiles. While a cost cannot be assigned to such statements it is clear that in the poorly drained terrain of this study the reduced tie maintenance was sufficiently significant to be observed by the day to day maintenance personnel.

SUMMARY AND CONCLUSIONS

While a wide range of geotextiles perform satisfactorily over a short period of time, much greater discrimination is required for good geotextile performance over longer periods. Geotextiles should never be used as a substitute for good or improved drainage practice. Rather they have their greatest potential for long term benefits in facilitating drainage from the track area to the side ditches by lateral flow along the geotextile under gravitational flow conditions. Installation procedures to ensure such conditions are achieved are of immense importance for satisfactory long term performance of the installation. Because drainage, or the lack of it, is such an important factor geotextiles are found most widely used in poor draining, flat and marshy terrain. Their low incremental cost in the installation of switches, diamonds and the like results in high benefits when correctly used below such track structures in poorly drained terrain.

Of the three major types of non-woven geotextiles studied, needle punched geotextiles which have been resin dipped followed by forced air drying to ensure retention of porosity were concluded to be considerably better than either heat bonded or needle punched with no resin dipping. Woven geotextiles were found to perform detrimentally on a long term basis as were heat bonded geotextiles. Needle punch resin dipped geotextiles with an EOS less than 40 microns (No 400 sieve size) are recommended as the small pore sizes resist clogging better and prevent the intrusion of larger particles (relative to a geotextile with a larger EOS) which were observed to cause greater internal fiber abrasion and reduced lateral permeability.

A trade-off between geotextile unit area mass (weight) and buried depth is apparent but a 700 g/m^2 (20 oz/yd^2) of needle punched resin dipped geotextile at a buried depth below tie base of 300 mm (12 inches) is recommended as a minimum for continuous welded rail and 1000 g/m^2 (29 oz/yd^2) as a minimum below discontinuities. These masses (weights) should be doubled if the geotextile is not resin dipped and increased 50 percent if the buried depth is decreased to 200 mm (8 inches). If less than 200 mm (8 inches) ballast depth is used damage may well occur from moving ballast during initial tamping reducing considerably the benefit from the geotextile.

The above recommendations leave substantial leeway for the development of new geotextiles involving multi-layer manufacture with low denier fibers being used on the outer layers and large denier fibers in the centre. Under no circumstances should two unattached layers of geotextile be used since fines could well accumulate between the layers.

ACKNOWLEDGEMENT

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Evaluation of Fabric Performance in a Rail-Road System

Evaluation du comportement d'un géotextile dans une voie ferrée

ABSTRACT

This paper reports the results of a study to evaluate the additional rigidity imparted by geotextile to the overall soil-ballast system. The triaxial tests indicate that the shear strength of the overall soil-fabric-ballast system has a significant increase, approximately 2.7 times compared to that of a soil-ballast sample. The rebounds and resilient moduli measured in the dynamic K_0-tests, indicate that the resilient behavior and deformation characteristics are improved in the soil-fabric-ballast system due to the contribution of the fabric. No punctures were noted on the geotextile after 300,000 cycles of repeated load nor was any significant abrasion observed.

Ce present article rapporte les resultats d'une etude sur l'evaluation de la rigidité additionnelle transmise par le géotextile à l'ensemble du systéme sol-ballast. Les essais triaxiaux móntrent que la resistance au cisaillement de l'ensemble sol-textile-ballast augmente d'une maniére considérable, à peu près 2.7 fois comparée à celle donnée par un échantillon de sol-ballast simple. Les rebondissements et les modules d'élasticité mesurés aux essais K_0 dynamiques, indiquent que le comportement élastique et les charactéristiques de déformation sont améliorés dans l'ensemble sol-géotextile-ballast. Ceci est dû à la contribution du géotextile. Ni les poinçonnements sur le géotextile, ni une abrasion significative n'ont été observés après 300,000 cycles de charge repétitive.

INTRODUCTION

Currently a wide variety of geotextiles are being used in highway and railroad construction. It is often suggested that these geotextiles may have a considerable potential for use in increasing the strength-deformation characteristics of the overall soil-ballast system both in new construction and in remedial maintenance applications. However, so far a quantitative evaluation of a geotextile in terms of additional strength characteristics imparted to the system has not been conclusively demonstrated. The objective of this investigation was to quantitatively evaluate the additional shear strength of the system when reinforced with a geotextile and the survival of the fabric under a number of repeated loadings. The former was evaluated by triaxial compression tests and later by performing dynamic K_0-test. Both investigations shall be reported in this paper.

TRIAXIAL TESTS

In order to investigate the stress-strain behavior of the soil-fabric-ballast system, triaxial tests on 2.36 cm x 4.72 cm (6 in x 12 in) composite samples of soil-ballast with and without fabrics were performed. The fabric between the soil and ballast was placed initially under some tension. This was accomplished by allowing the fabric to hang over the prepared soil sample and then applying two "O" rings over the fabric and soil as shown in Figure 1. Triaxial tests on (same size) soil and ballast specimen were performed to compare the results with that of the composite soil-ballast and soil-fabric ballast. For each of tests, three consolidation stresses were used: 68.9, 103.35 and 137.8 kpa (10, 15 and 20 psi). Additionally, triaxial tests on a 2.36 x 2.36 cm soil smaple and on a 2.36 x 3.15 cm (6 in x 8 in) soil-fabric-ballast sample with 137.8 kpa consolidation stress were performed to study the effect of end restraints and the height to diameter ratio of the samples. The fabric used was woven Celanese Mirafi 600X.

Soil: The soil used in the experimental program was a sandy clay which can be synthesized in the laboratory by the following constituents by weight: kaoline-50%, ottawa sand-45%, and bentonite-5%. The soil had a specific gravity of 2.66, Liquid Limit of 40 and Plasticity Index of 28. The optimum moisture content and maximum dry unit weight were found to be 16% and 17.6 kN/m^3 (112.3 pcf) respectively.

Ballast: The ballast selected was from AAR Track laboratory and consists of lowgrade limestone. The grain size distribution of soil and ballast is shown in Figure 2.

Preparation of Samples Without and With Fabric: The triaxial samples were prepared in a split cylindrical mold lined with a rubber membrane. In preparing composite samples of ballast and soil without fabric, the mixed soil was placed and compacted on the bottom of the split mold in four equal layers; then the ballast was placed and compacted on the soil in three layers. The compaction energy for both the soil and ballast was by means of standard Proctor hammer to compact 25 blows per layer. The dry unit weight of the soil was 108 pcf and the moisture content was

18.5%. The compacted unit weight of the ballast was 157 kN/m^3 (100 pcf). When the preparation of samples was complete, the top of the ballast was levelled and the top platten placed in position. To minimize air leakages a second layer of two thinner membranes were placed over the first. The membranes were then sealed to both the top and bottom plattens by means of clamps. In preparing the samples with fabric (refer to Figure 3), the fabric sheet was placed on top of the compacted soil. The fabric was stretched slightly and held in placed by two "O" rings which provided some tension to the fabric. In order to set up the fabric sheet in the sample, the split mold and the rubber membrane had to be removed after compacting the soil sample. When the fabric placement was complete, another rubber membrane was reset on the sample. The split cylindrical mold was then put back on the sample and the ballast was compacted as the same of that of the samples without fabric.

ANALYSIS OF TRIAXIAL TEST RESULTS

All the triaxial tests conducted in a saturated condition were isotropically consolidated undrained tests with pore pressure measurements. The deviator stress versus axial strain curves under three consolidation pressures are shown in Figures 4 and 5. A comparison of the maximum deviator stress of the soil-fabric-ballast samples with the soil-ballast samples, clearly indicates that the strength of the soil-fabric-ballast sample is singnificantly more. The maximum deviator stress of the soil-fabric-ballast sample was about 2.7 times of that of the soil-ballast sample under 68.9 kpa (10 psi) consolidation stress. A comparison of maximum deviator stress for soil-ballast and soil-fabric-ballast samples is presented in Table 1.

Table 1. Maximum Deviator Stress

Confining Pressure	Maximum Deviator Stress	
	Soil-Ballast	Soil-Fabric-Ballast
68.9 kpa	45.0 kpa (6.54 psi)	122.0 kpa (17.7 psi)
103.4 kpa	68.9 kpa (10.0 psi)	197.0 kpa (28.6 psi)
137.8 kpa	87.5 kpa (12.7 psi)	237.7 kpa (34.5 psi)

The maximum deviator stresses of 2.36 x 2.36 cm soil and 2.36 x 3.15 cm soil-fabric-ballast samples tested under confining pressure of 137.8 kpa were 88.5 kpa (12.8 psi) and 232.9 kpa (33.8 psi) respectively. The stress paths for different kinds of samples are shown in Figure 6 where the change of stresses are shown by dashed lines.

The pore pressure increase and decrease during the test for ballast (only) samples indicated the typical behavior of dense materials. For all other samples, the pore pressure increased after the tests started until the samples failed. This indicated that these samples failed due to failure in soil portion. Also it shows that the tested fabric did not cause any deviation in pore pressure behavior, thus providing free movement of water.

The tests conducted on 2.36 x 2.36 cm soil samples and 2.36 x 3.15 cm soil-fabric-ballast samples (see Figure 6) indicate that the increase of the deviator stresses of the soil-fabric-ballast samples was due to the contribution of the geotextile sheet and not due to the effect of end restraints. Additionally, the maximum deviator stress developed in 2.36 x 3.15 cm (6 in x 8 in) soil-fabric-ballast sample was 232.9 kpa (33.8 psi); which is very close to the maximum deviator stress developed in 2.36 cm x 4.72 cm (6 in x 12 in) soil-fabric-ballast sample - 237.7 kpa (34.5 psi). The deviator stress versus axial strain curve of these samples are plotted in Figure 6. The figure indicates that the contribution of the geotextile to the soil-fabric-ballast system investigated from the tests was not restricted to the 2.36 cm x 4.72 cm (6 in x 12 in) samples. The generality of the contribution of the geotextile in the soil-fabric-ballast system is thus demonstrated.

Figures 7 and 8 show the transformed hyperbolic representation of the stress-strain relationship for soil-ballast and soil-fabric-ballast samples. The initial tangent modulus versus consolidation stress plots are presented in Figure 9. These figures clearly indicate that the geotextile improves the stress-strain behavior of the soil-ballast system by providing more rigidity to the system.

It was observed that after the triaxial tests the fabric sheets were concave irregularly because of the compression of the ballast, but no punctures occurred. Some fine materials from the soil samples were observed on the upper portion of the fabric. In the samples without the fabric, the ballast intruded into the soil portions as observed after the triaxial tests.

DYNAMIC K_0-TESTS

The K_0-test cell consists of a steel cylinder, and a plexiglass cylinder which is the container for preparing the sample. In order to place the fabric between soil and ballast, the plexiglass cylinder was separated into two parts. A grove was cut into the plexiglass to interlock the fabric in the sample. Figure 10 shows the details of K_0-cell. The dynamic K_0-tests were conducted with soil-ballast and soil-fabric-ballast samples using Celanese 600X and Monsanto C-34 fabrics. The intent was to investigate the puncture resistance, separation characteristics of the fabric and the overall response of the soil-fabric-ballast system. The loading apparatus for the cyclic loadings for the K_0-tests was a Servo-Controlled hydraulic power system which applied loads with a frequency of 2HZ per second. Model 9300 of Monitor Laboratory was used for data recording.

Rebounds and Permanent Deformations: The rebounds of the sample after subjecting to 20,000, 40,000, 60,000, 100,000, 150,000, 200,000, 250,000 and 300,000 cyclic loadings of vertical stress σ_v=276.6 kpa (40 psi); were measured. It was observed that the rebounds of the soil-fabric-ballast samples decreased as the number of repetitive loadings increased. The rebounds of the soil-ballast sample also show the same behavior (Figure 11). However, by comparing the rebounds of the soil-ballast and soil-fabric-ballast samples, it was found that there was significant difference especially after 20,000 cycles of repeated loadings. From Figure 11 it may be noted that the permanent deformation for the soil-ballast sample was significantly larger than that of the soil-fabric-ballast samples. It clearly demonstrates that the deformation characteristics of the soil-fabric-ballast system was improved by the fabric layer.

Resilient Behavior: The semilog plots of the resilient moduli versus numbers of cyclic loadings are shown in Figures 12 and 13. Significant difference of the resilient modulus between soil-ballast and soil-fabric-ballast samples were observed on and after 200,000 cycles of repeated loadings. The two dynamic K_0-tests performed with Celanese 600X and Monsanto C-34 showed the same behavior. Based on the above analyses, it may be concluded that the resilient behavior and deformation characteristics of the soil-fabric-ballast system were improved by the geotechnical fabric.

Separation Function of Fabric: In the dynamic K_0 tests, the ballast was oven dried and weighed after 300,000 cycles of repeated loadings, and then it was washed on

No. 140 sieve. After the materials passing through sieve were washed through the sieve, the ballast was then oven dried and weighed. A comparison of the dry weight of the ballast before and after washing (on No. 140 sieve) indicated a difference of 1.6N (0.35 lb) for Celanese 600X and 1.4N (0.30 lb) for Monsanto C-34. For the soil ballast sample, the difference was found to be 6.1N (1.34 lbs). It may be pointed out that the differences in dry weights before and after washing on No. 140 sieve, for the soil-ballast sample were predominantly from pumping (movement into ballast voids) of the soil and very little portion was from abrasion of the ballast. Contrary to above, in the soil-fabric-ballast samples, the differences in dry weights can be predominantly attributed to abrasion of ballast due to the presence of fabric layer. Therefore, the fine materials pumped into the ballast from the soil portion of the soil-ballast sample were significantly larger than those of the soil-fabric-ballast samples. It indicates that the fabric sheet has the function of reducing the cohesionless materials pumped into the ballast from subgrade.

ACKNOWLEDGEMENTS

The authors are grateful for the financial support provided by the Association of American Railroads (AAR). Technical discussions with and help in the design of experiments of Mr. Dov Leshchinsky of AAR are gratefully acknowledged. The authors thank Mr. Donald Spreadbury, Laboratory Technician of Civil Engineering Department for his help during the project.

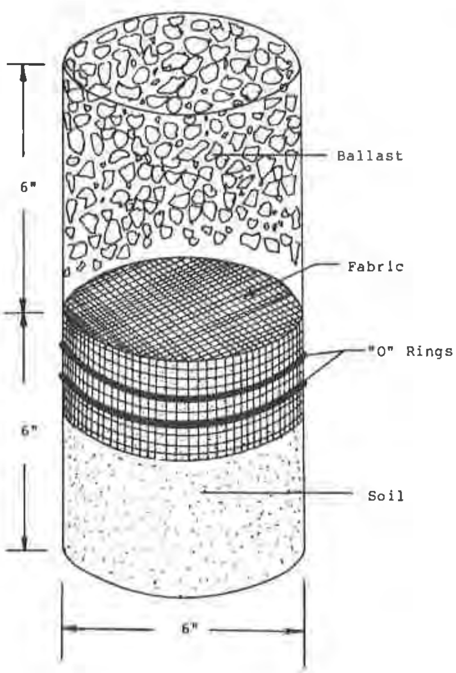

Figure 1. Typical Soil-Fabric-Ballast Specimen

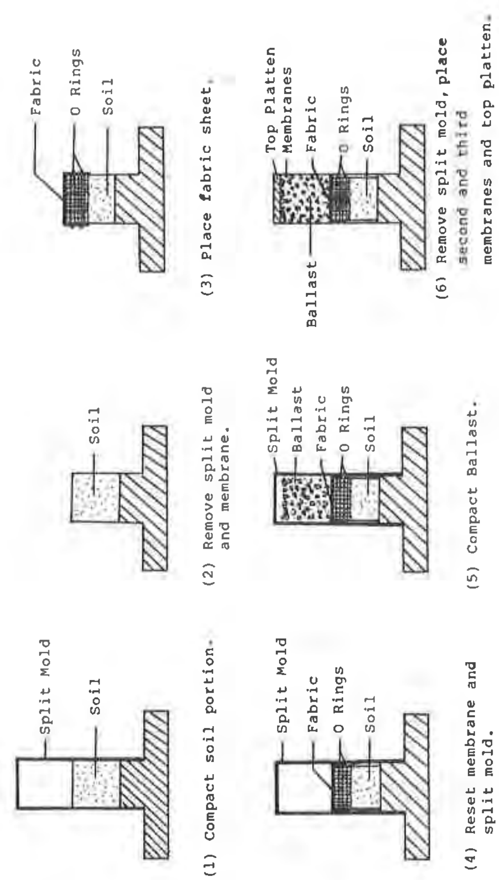

Figure 3. Procedure of Preparing Triaxial Soil-Fabric-Ballast Samples

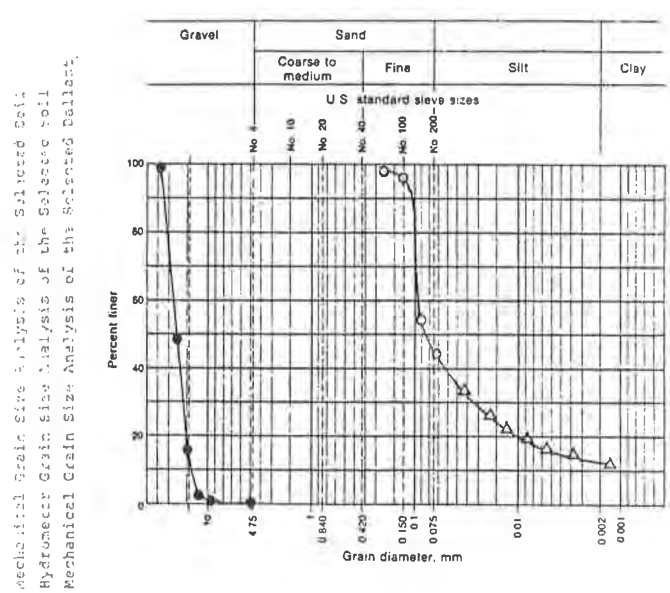

Figure 2. Grain Size Distribution Curve for the Soil and Ballast.

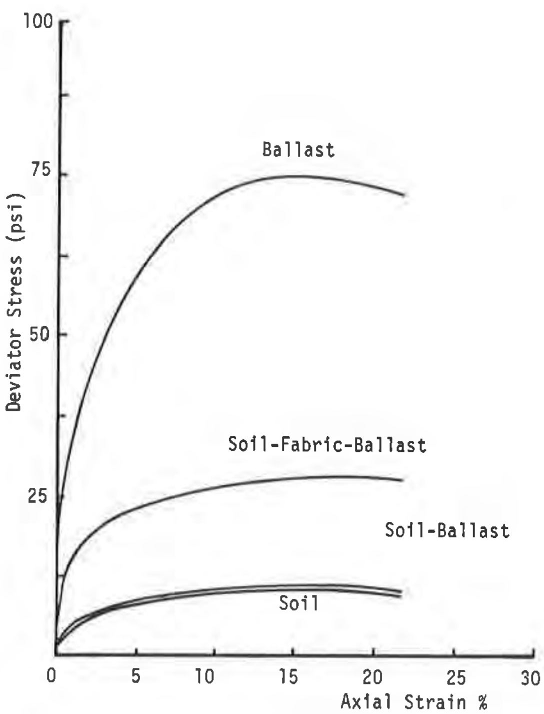

Figure 4. Deviator Stress versus Axial Strain (σ_c = 10 psi) (1 psi = 6.89 kPa)

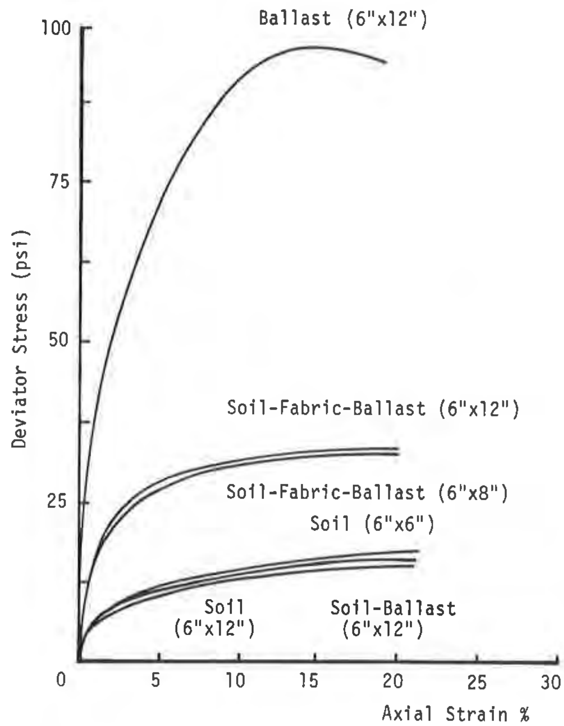

Figure 5. Deviator Stress versus Axial Strain (σ_c = 20 psi) (1 psi = 6.89 kPa)

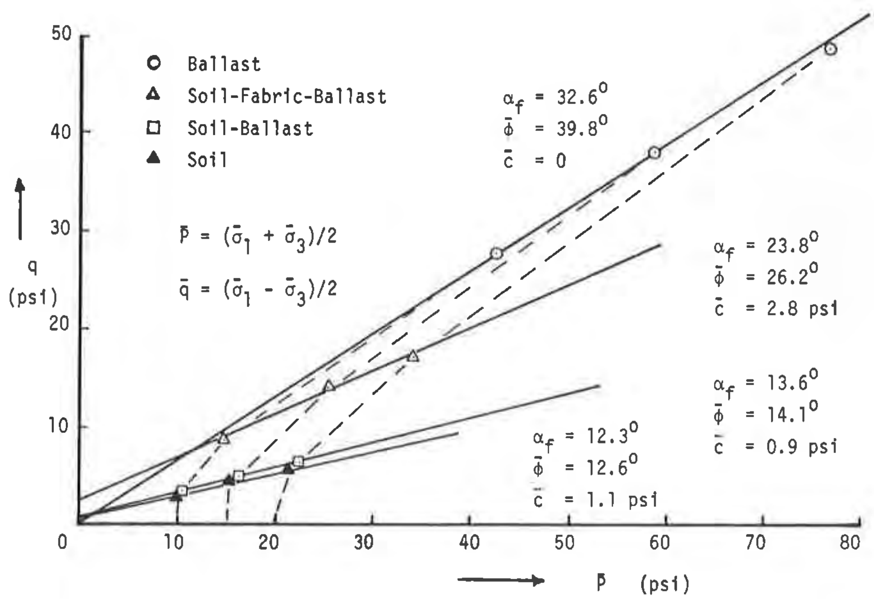

Figure 6. Stress Paths for Triaxial Tests. (1 psi = 6.89 kPa)

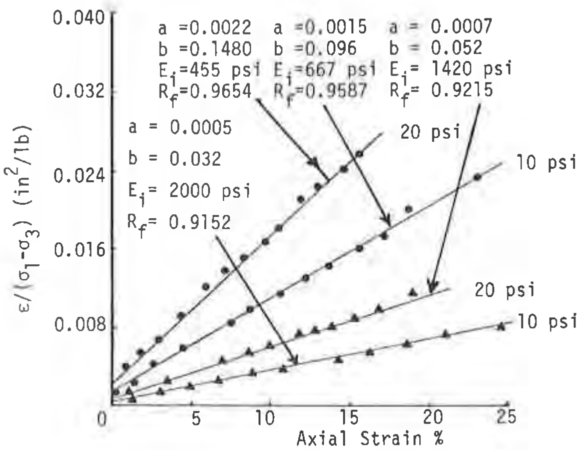

Figure 7. Transformed Hyperbolic Stress-Strain Relationship for Different Confining Pressure

1 in = 2.54 cm, 1 lb = 4.45N

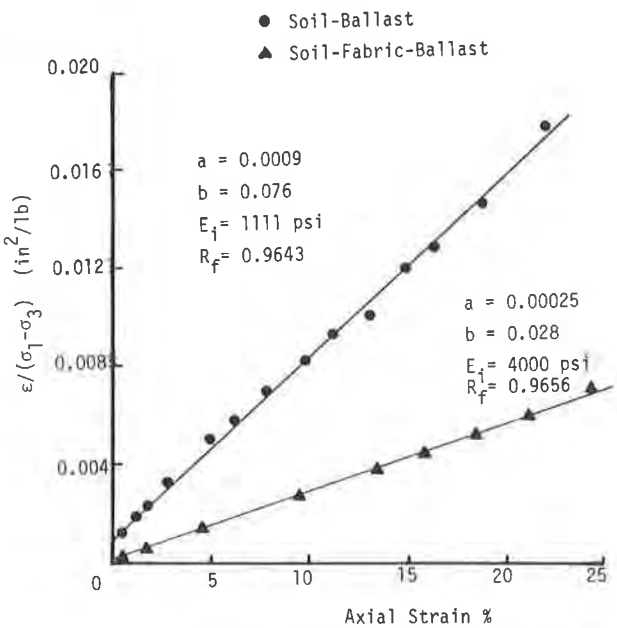

Figure 8. Transformed Hyperbolic Stress-Strain Relationship (σ_c = 20 psi)

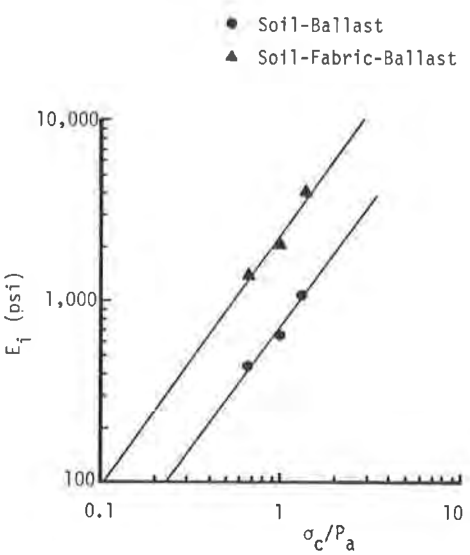

Figure 9. Relationship between Initial Tangent Modulus E_i and σ_c/P_a

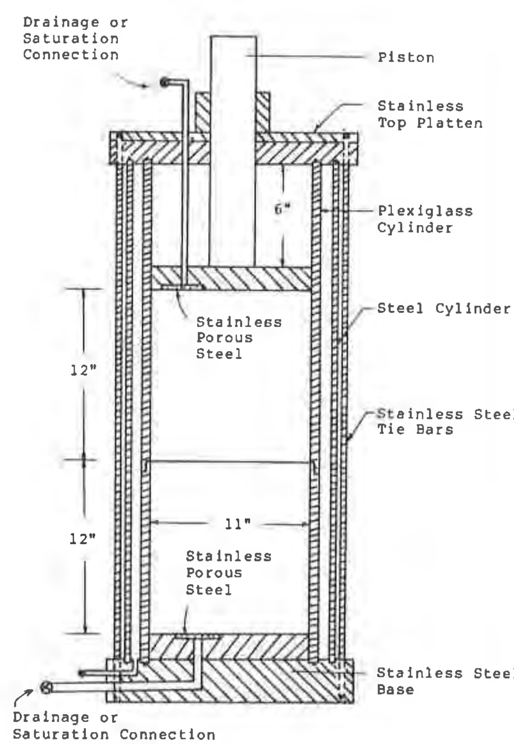

Figure 10. Details of K_o Cell for Dynamic Tests

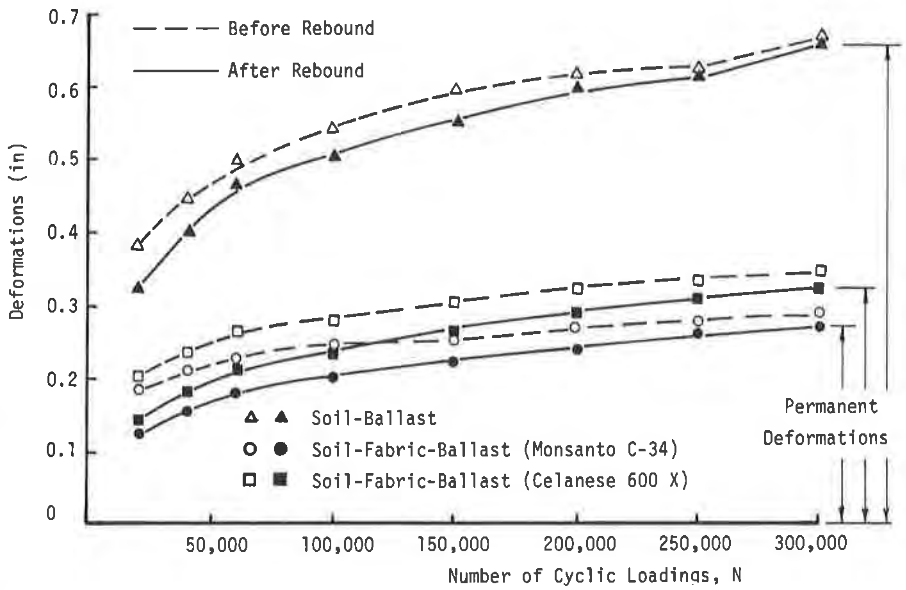

Figure 11. Rebounds and Permanent Deformations of Soil-Ballast and Soil-Fabric-Ballast Samples after 300,000 Cyclic Loadings (t = 100 min)

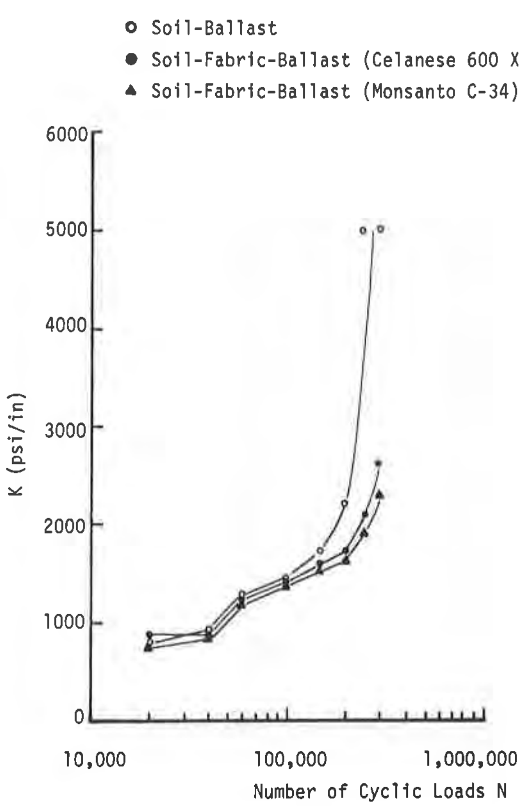

Figure 12. Number of Cyclic Loads versus K (t = 10 sec)

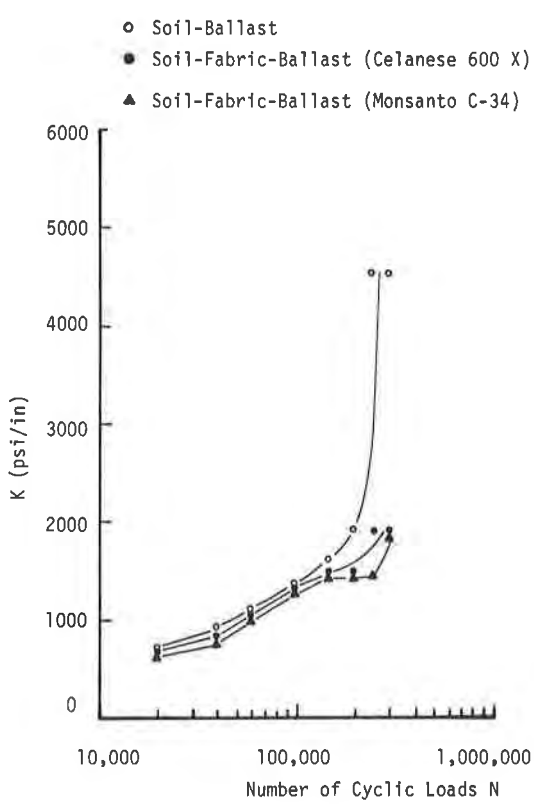

Figure 13. Number of Cyclic Loads versus K (t = 1 min)

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Full Scale Railroad Geotextile Testing Procedures
Examens completes de géotextiles-chemin de fer

During the past several years, many railroad lines have conducted full scale in-service tests of various geotextiles. Unfortunately, some of the results of these tests are difficult to correlate, and are often fragmented, invalid, or even unrecorded. This paper describes testing procedures and methods which have been shown to provide clear, useful, analytical results. Accordingly, the paper should provide railroads with a checklist, if not a guideline, for conducting future geotextile tests.

Pendant plusieurs annees passees, beaucoup de lignes de chemin de fer ont donne les examens completes de geotextiles. Malheureusement, beaucoup de resultats de ces examens sont difficiles de mettre en correlation et sont souvent fragmentes, invalides, ou meme pas enregistres. Ce papier decrit les procedes des examens et les methodes qu'on montre pour donner les resultats clairs, utiles et analytiques. En consequence, le papier doit donner aux chemins de fer une liste de controle si non la norme pour conduire les examens geotextiles.

INTRODUCTION

Throughout this paper, the assumption has been made that geotextile test sites shall involve existing track as opposed to new construction, simply because existing sites typically are exposed to much more severe conditions. Specifically, an existing site is likely to have been chosen because it already is troublesome. Additionally, the geotextile is abraded much more severely when installed following an undercutting operation.

Types of Geotextiles Available

References 1 and 2 will provide the reader with a detailed description of the various types of geotextiles as well as their manufacturers and distributors.

It should be noted, however, that railroad tracks are usually considered the most severe application for geotextiles. Notably, the abrasion resistance required is particularly critical (elsewhere in these proceedings, Dr. G. Raymond discusses the value of resin bonding in abrasion resistance); lateral drainage may be required in which case a thick, needlepunched nonwoven or composite fabric will work best; and tensile strength is more critical in the cross-track or transverse direction of the fabric, than in the trackwise, machine direction of the fabric (see reference 3). Other considerations such as burst strength and puncture resistance may also be critical.

In any case, it is to the best interest of all concerned to test as many products as practical. Accordingly, at least one manufacturer of each of the following types of geotextiles should participate:

- Wovens
- Heat-bonded nonwovens
- Needle-punched nonwovens
- Composites

These manufacturers should be especially encouraged to donate new or improved products for testing and evaluation.

Test Site Selection

If at all possible, the test site should be a straight section of normally high speed, high volume, high tonnage double track. Both tracks should be undercut and rehabilitated. The geotextiles should be installed on one side, with the other side serving as a "control" section. The section of track should be troublesome, i.e. one which has required frequent re-surfacing or undercutting.

The test section should be sufficiently long to allow the installation of at least one hundred feet of each fabric to be tested; and the adjacent terrain and subgrade conditions should be consistent in order not to prejudice any fabric results. Finally, shoulder drainage should be provided (clean swales will suffice) in order to check the lateral drainage capabilities of the geotextiles.

Geotextile Installation Techniques

The two normal methods are:

(a) Lay the fabric out alongside the track, then manually pull it under the track, or
(b) attach the fabric to a roller on the side or beneath the undercutter.

Both methods should be tried and evaluated.

Installation Records

The following data should be collected at the time of installation:

(1) Precise stations which locate the beginning and end of each type of fabric.
(2) The width of each type of fabric.
(3) The planned depth of each fabric below the rail - record data beneath each rail and at centerline.
(4) Type of ballast, i.e., granite, limestone, Open hearth slag, etc.
(5) Condition of ballast, i.e., new, cleaned, partially cleaned, etc.
(6) Take soil sample of fines in ballast if it is not clean. (Use airtight bag.)
(7) Take soil sample beneath the fabric. (Use airtight bag.)
(8) Analyze moisture content and grain size distribution for (6) and (7) above.

Photographic Documentation

Because of the large number of photographs likely to be taken at each site "opening", it is critical that the photographs be adequately documented. To do otherwise is to invite mass confusion because the photographs will be impossible to identify.

Accordingly, the following information should be clearly printed on a slate board:

Fabric Type
Date
Station

The slate board should then be placed such that its information will be seen in each photo. Note that because there may be a large number of manufacturer's representatives wishing to take their own photos, the slate must be carefully placed, all the photos taken for that particular situation, then the slate must be moved for the next shot, etc. However tedious, this procedure should be considered imperative.

Frequency of Inspection

The actual frequency of inspections may vary from test site to test site, depending on the severity of the problems and the effectiveness of the geotextiles. Nonetheless, the minimum recommended frequency is semi-annually for the first two years, annually for the next three years, and bi-annually thereafter.

Site Inspections - "Openings"

The first item to note prior to opening the track to expose the fabric, is whether there has been any track mis-alignment or deformation - if so, these should be carefully documented.

Each type of fabric should then be exposed, at the same relative location (e.g., 20 feet from the leading edge). Each of the following procedures should be photographed, being careful to document as described above. Additionally, each of the airtight bags referred to below should be similarly documented, i.e., fabric type, date, and station.

(1) Remove one tie.
(2) Remove the ballast above the fabric. Within four inches of the fabric, this procedure must be done manually, with great care, in order to preclude damage to the fabric.
(3) Obtain a sample of the "fines" above the fabric, and place in an airtight bag. Then remove all of the ballast above the fabric.
(4) Clearly mark the fabric such that the region one foot each side of each rail can be identified, then cut out a piece of fabric 18 inches wide across the width of the track. Place the fabric in an airtight bag.
(5) Obtain a soil sample from beneath the fabric and place in an airtight bag.
(6) Measure the distance beneath each rail to the fabric, as well as the centerline distance to the fabric.
(7) Replace the removed section of fabric with new fabric, being careful to overlap at least 12 inches.
(8) Replace the ballast and tie.
(9) If the site is a double track, take the appropriate samples from the "control" track.

Laboratory Testing

The following laboratory tests should be performed on the fabric and soil samples:

(1) Geotextile Tests
    (a) White light penetration test for punctures (sample under rail)
    (b) Observation for "accomodation",i.e., "bumps" but no punctures (see reference 3) (Sample under rail)
    (c) Tensile tests in both cross-track and track-wise directions (Sample under rail)
    (d) Permeability
    (e) Mullen Burst
    (f) Other possible tests are
        - dry weight change after washing
        - moisture content
        - puncture resistance
        - lateral permeability
        - trapezoidal tear
        - etc.
(2) Soils - Ballast
    - moisture content
    - grain size distribution
    - soils analysis

Laboratory Selection

Several manufacturers have excellent laboratory facilities which can be used, provided other manufacturers do not object. In any case, a number of independent soils testing laboratories now have geotextile testing facilities.

Presentation of Results

Particularly if the tests were conducted by a manufacturer's laboratory, results should be presented as determined, with no editing or comment. For example, as the soils are analyzed, no comment should be made regarding the geotextile propensity for trapping or draining moisture. Such interpretation should be reserved for those whose task it is to interpret, as discussed in the nest section.

Interpretation of Results

As soon as the testing results are completed, copies should be sent to each manufacturer as well as a minimum of two independent geotextile consulting engineers for comment and interpretation. All of these comments may then be compiled, either by another geotextile consultant or by railroad personnel, into a final report.

SUMMARY

Perhaps the most critical step of all in such full scale railroad testing is the publication of the results as soon as possible, in order that the level of knowledge may be improved for the industry at large. There may be objections by some manufacturers to such publication due to the relatively poor performance of their products. The obvious response to such objections is that manufacturers must be willing to take the risk of failure in such testing in order to properly evaluate and improve their own products. Indeed, both the railroads and the geotextile industry can only benefit through the publication of such results.

Acknowledgements

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Model Test of a Rail-Ballast-Fabric-Soil System

Essai d'un modèle rail-ballast-textile-sol

ABSTRACT

The paper presents the results of a laboratory model tests to study the reinforcing functions of a geotextile in a rail-ballast-fabric-soil system. The geotechnical fabric (geotextile) caused the entire system to behave as a relatively elastic foundation, and therefore the response to loading can be predicted. It was also found that the reinforcing effect of the geotextile causes reduced deflections and strains within the whole system. Such laboratory model tests can serve as a steping stone for evaluation of input parameter for a numerical analysis scheme.

Ce présent article présente des essais modèles de laboratoire à l'étude du rôle de renforcement d'un géotextile dans un système de rail-ballast-géotextile-sol. Le géotextile donne à l'ensemble du système un comportement comparable à celui d'un fondement relativement élastique. Donc, la réponse au chargement puet être estimée. Aussi, il a été observé que l'effet de renforcement du géotextile créait des flexions et déformation réduites dans l'ensemble du systeme. Tels essais peuvent servir comme le fondement d'urve évaluation des paramètres d'entrée d'un algorithme d'analyse numérique.

INTRODUCTION

Model testing under controlled condition has been a forceful tool for studying mechanism in geotechnical engineering. This paper presents the results of a study to evaluate the performance of a selected geotextile in a rail-ballast-fabric-soil system under controlled laboratory model testing. The model test to be presented can be used as a qualitative basis to define the so-called input parameters for numerical analysis of the system using available models. The major objective of this study was to evaluate the reinforcing function of a selected geotextile. The investigation included the model testing of the system and also laboratory testing for evaluation of material characteristics.

DESCRIPTION OF MODEL TEST

The model consisted of:

a. A light crane rail with five wood ties.
b. A subgrade with known properties.
c. A selected geotechnical fabric.
d. A container tank for the soil and ballast; and for conducting the experiment.
e. A loading frame and equipment.

Rail and Ties. A.S.C.E. light 178 N (40 lb) crane rail 241 cm (95 in.) long with five oak wood ties 94 cm (37 in.) long, 5 cm (2 in.) wide and 4.57 cm (1.8 in.) thick, was used (Fig. 1). The ties were attached to the botton of rail by screws on both sides of the rail with steel tie plates (Fig. 2).

Subgrade. The soil used for subgrade consisted of 50% of Kaolin clay, 45% of Ottawa sand, 5% of Bentonite clay and 18.5% of water (by weight). The soil can be classified as a sandy clay with a specific gravity of 2.67, liquid limit of 44 and Plasticity Index of 22. The standard compaction test data provided an optimum moisture content of 16% and maximum dry density of 17.3 kN/m^3 (110 pcf). The soil used for the tests had a water content of 18.5%; slightly over the optimum. The ballast selected was from AREA No 4 of AAR Track Laboratory and consisted of low grade limestone. The maximum dry density of the ballast was 20 kN/m^3 (128 pcf). The grain size distribution for soil and ballast is shown in Figure 3.

Geotextile. AMOCO non-woven fabric STYLE EPRH-336 was used for this test.

Container. A steel tank 241 cm (95 in) long, 94 cm (37 in) wide and 105 cm (41.5 in) deep was used.

A soil mixer and a compactor was used during the preparation of subgrade. The soil was deposited in the tank and compacted in 10 cm (4 in) thick layers to a total thickness of 76 cm (30 in). About 10.67 cm (4.2 in) of ballast was then placed on top of the clay and then levelled. The soil and tie were placed on the ballast, and in the area where there were no ties additional ballast was placed to bring the total thickness to 15.25 cm (6 in). For the second test where the geotextile was used, the fabric was installed prior to placing the ballast. The fabric was placed with an initial tension by hanging a weight at the ends.

Loading Frame and Equipment. The loading frame (Fig. 4) was designed to apply a moving vertical load up to 45 kN (10,000 lbs) maximum. The moving load consisted of a single wheel 15.25 cm (6 in) dia and 9.62 cm (3 in) thick, was attached to hydraulic cylinder and electrical system (Fig. 5). The apparatus included a 1 HP electric three way motor, reversing control system, two limit switch, stainless chain and sprockets. The motor gave the power for the wheel to move at a constant speed of 25 secs. per trip, after touching the limit switch, the rotation of the motor reversed automatically and caused the wheel to move back to touch the other limit switch and vise versa to make it move back and forth.

Measuring Equipment. The data measuring equipment in the test included embedded soil stress and strain gages, steel and fabric foil strain gages and mechanical deflection gages. The soil stress gages were piezo-resistive gages. The soil strain gage consist of a pair of sensors 2.54 cm (1 in) in diameter. The average distance between the two sensors was about 2.54 cm (1 in). The soil stress and strain gages were placed in the soil at a depth of 9 cm (3.5 in). Prior to placing in soil standard calibration procedure was utilized. Figure 6 provides the plan view of the location of soil stress and strain gages.

A strain indicator unit were used to measure the strains of foil gages fixed at various locations on the rail. Six foil gages were fixed on the flanges of the rail to measure the bending of the rail (Fig. 6). Two foil gages were placed on each side of steel tank to measure any longitudinal or transverse deflection of the tank. Six foil gages were also fixed on the fabric to measure its expansion, however, the gages were damaged by ballast and did not provide after first loading.

One mechanical dial gage was placed on each tie to measure the deflection at the center of the ties. The gages were capable of measuring deflections to 0.0025 cm (0.001 in) sensitivity.

TESTING PROGRAM

One test was conducted without fabric and the other with fabric. Each test following a loading and unloading procedure required a total of 6 days. At various times during the testing, the loading wheel was stopped at each tie and the data from the dial and strain gages was recorded. The test procedure was as follows:

1. The first day of the test, 22.3 kN (5,000 lb) of vertical load was applied for 500 cycles. The load was then completely removed. A cycle is defined as one complete movement from one edge of tank to the other end and back.
2. The second day of the test, 22.3 kN (5,000 lb) of vertical load was applied for 275 cycles. The load was then completely removed.
3. The third day of the test was utilized for observing and recording the rebound of the system.
4. The fourth day of the test, 26.7 kN (6,666 lb) of vertical load was applied for 400 cycles. The load was then completely removed.
5. The fifth day of the test, 26.7 kN (6,666 lb) of vertical load was applied for 350 cycles. The load was then completely removed.
6. The sixth day of the test was utilized for observing and recording the rebound of the system.

After finishing the first test without the fabric, the ballast and the top 10.15 cm (4 in) of the soil were removed and replaced by new soil, placed and compacted as previously described. The fabric was then placed on top of soil. An initial tension was applied in the fabric by hanging weight. The fabric was covered with ballst and second test was performed.

TEST RESULTS

The results obtained from tests with and without the geotextile are now presented. The vertical moving loads for two series of tests were 22.3 kN (5,000 lbs) and 26.7 kN (6,666 lbs). These were chosen because they produce pressures in the soil layer close to that of real life (i.e., under railroad loading). The comparison from the later loading (that is from 26.7 kN) only will be presented in the paper.

The ties were numbered from 1 to 5 as shown in Figure 1 and Figure 6. The measurements were recorded at each tie immediately after the loading started, after loading to 250 cycles and 400 cycles. The load was then removed and the system allowed to rebound for twelve hours. The system was then reloaded and measurements recorded immediately after reloading, after loading to 500 cycles and 750 cycles. The loading wheel was stopped at each tie after each of above mentioned period, and readings from all gages were noted. Selected data for 26.7 KN load after 400 cycles and rebound readings after removal of the load for a period of twelve hours are presented in Figures 7 and 8. In Figure 8 the deflection readings presented are the maximum deflections and represent the deflection under the load (i.e., the deflection under tie 1 is the deflection when load was at tie 1 and deflection under tie 2 is deflection when load was at tie 2 and so on). Similarly Figure 9 presents the maximum deflection after 750 cycles and rebound readings after removal of the load for a period of twenty-four hours. Strains recorded from soil strain gage No. 2 during 26.7 KN loading period can be read from Table 1.

Table 1. Strains Obtained from Soil Strain Gage No. 2 During 6,666 lb (26.69 kN) Loading Period

Loading Period	Test without Fabric	Test with Fabric
Loading Started	0.0024	0.0029
After 250 Cycles	0.0048	0.0037
After 400 Cycles	0.0096	0.0042
Loading Restarted	0.0034	0.0024
After 500 Cycles	0.0045	0.0024
After 750 Cycles	0.0052	0.0045

The deflections obtained with and without the fabric of course were found to have similar patterns. A plot of load versus total deflections at the point where the load was acting upon, at different cycles of load is shown in Figures 10 and 11 for positions 1 and 3. Plots of the strain (total deflection at the point where the load was acting upon, divided by total length of rail 241.3 cm) is plotted against the number of cycles on a semi-logarithmic scale in Figures 12, 13 and 14 for positions 1 and 3. The results indicate the similar patterns and similar slopes of relation between strain and number of cycles (especially after 400 cycles). It is clear that larger strain occur in the subgrade without the geotextile than in the subgrade reinforced with geotextile.

CONCLUSIONS

A comparison of results from with and without geotextile indicate the following:

Total deflection of ties was reduced by about 30% when fabric was used.

From the tie deflection patterns, it can be seen that the subgrade behaves as an elastic foundation when reinforced with fabric. It takes about 450 cycles of loading for the soil unreinforced by fabric to exhibit the same behavior, i.e., to cause to elastic behavior. In other words the 450 cycles of loading cause a densification in the ballast-soil system which was equal to ballast-fabric-soil at the initial stage.

The investigation therefore proved that the geotextile due to its membrane action, caused the entire system to behave closer to an elastic foundation. Its response to loading therefore can be predicted. However it must be borne in mind that an investigation of a scaled structure (rail and ties) on a prototype subgrade cannot be directly applied to the field. However such an investigation increases the understanding of the mechanisms involved and can serve as a check to a numerical analysis scheme of the input parameters of the ballast-fabric-soil system are known.

ACKNOWLEDGEMENT

The financial support provided by the AMOCO Fabric Company for the investigation is deeply appreciated. Thanks are due to Anton Ketterer, technician in C.E. Department for assistance in building the test equipment.

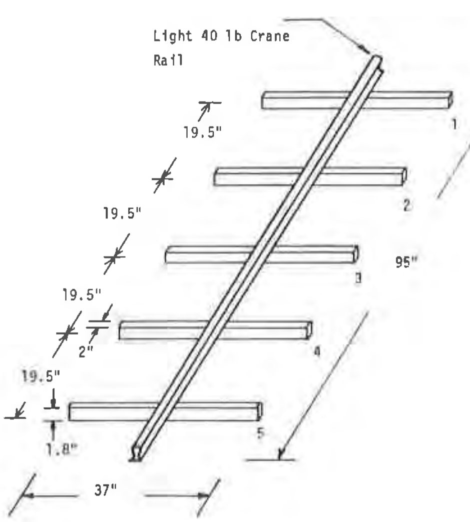

Figure 1. Light 40 lb Crane Rail with Five Oak Wood Ties. (1 in.=2.54cm, 1 lb=4.45N)

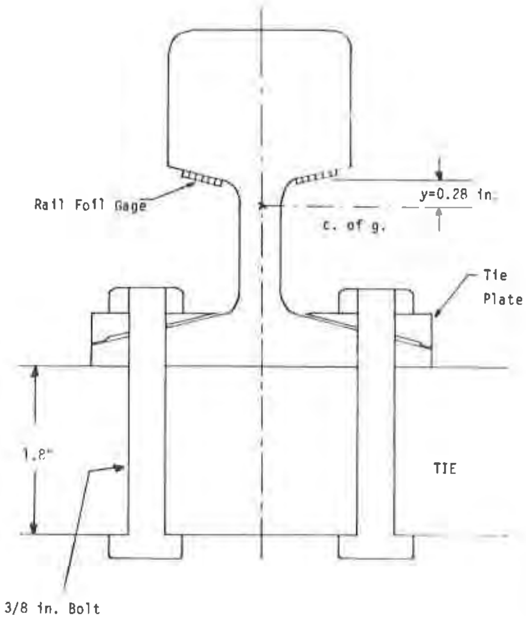

Figure 2. Light Crane Rail with Tie Plate and Rail Gages. (1 in.=2.54cm, 1 lb=4.45N)

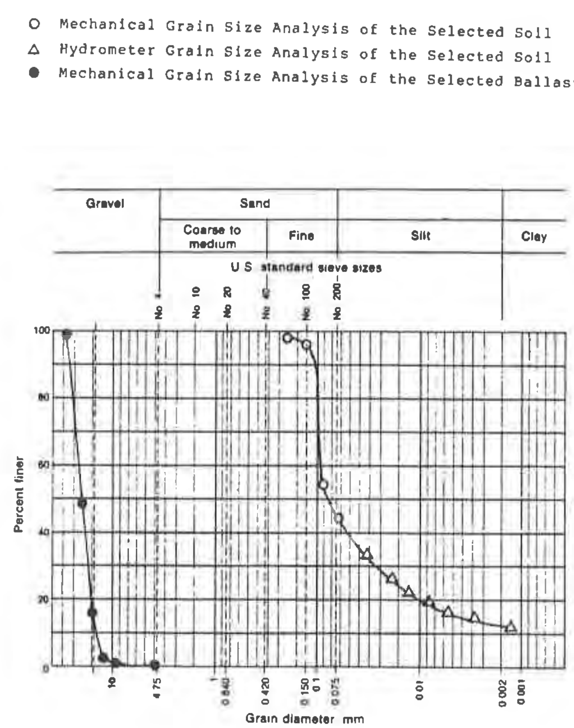

Figure 3. Grain Size Distribution Curve for the Soil and Ballast.

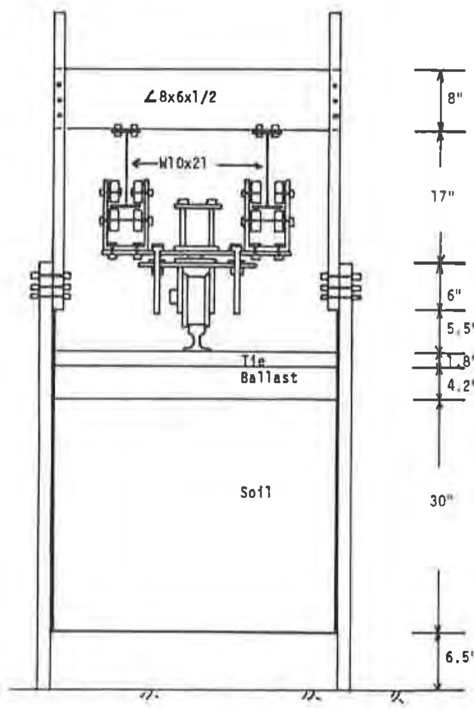

Figure 4. Loading Frame Front Profile.
(1 in.=2.54cm)

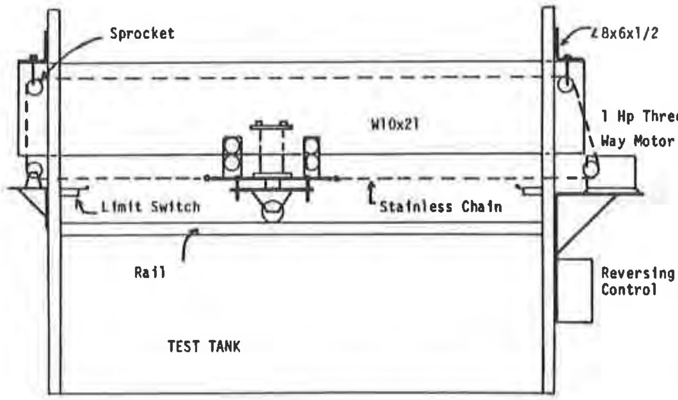

Figure 5. General View of the Electrical System of the Model Test Tank.

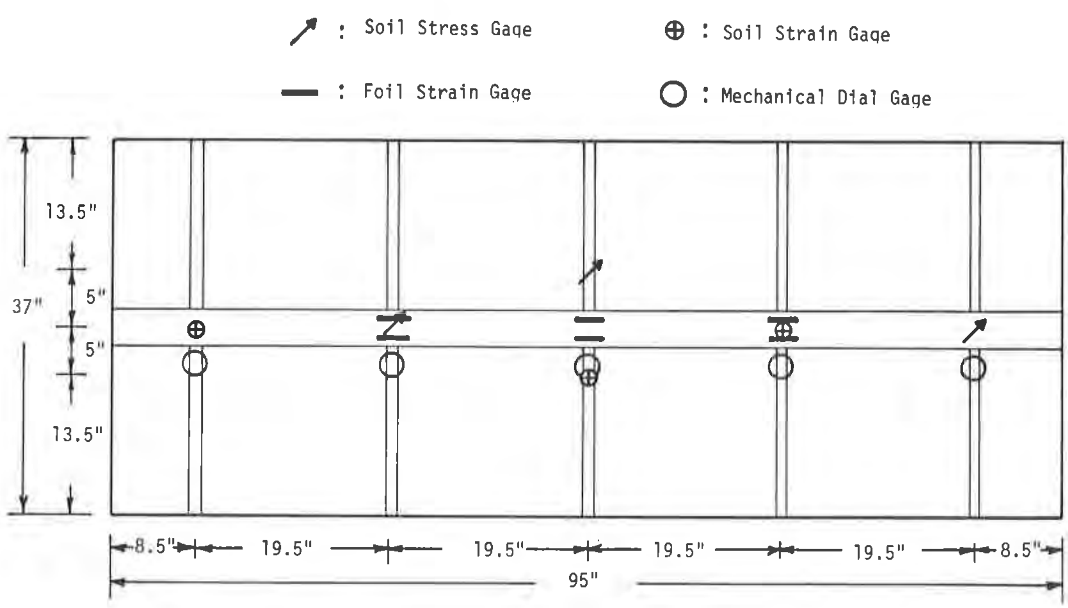

Figure 6. General View of Placement of Stress Strain Gages and Dial Gages.
(1 in = 2.54 cm)

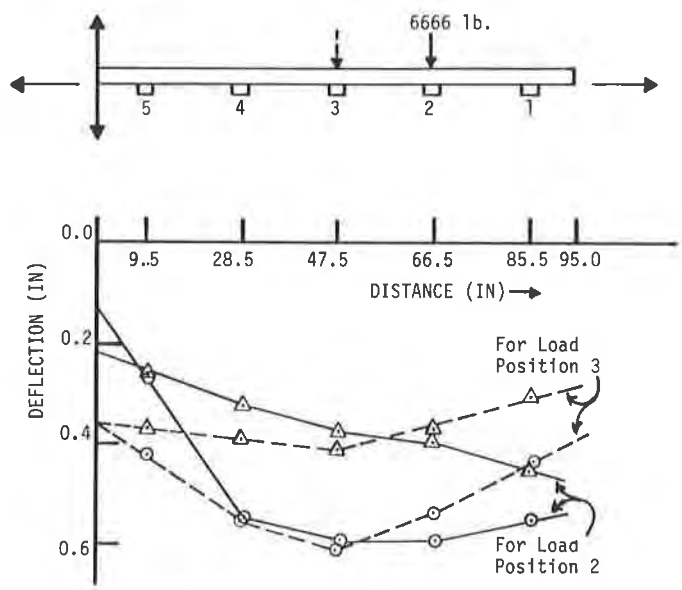

Figure 7. Tie Deflections for Load at Position 2 and 3 after 400 Cycles of Loading.

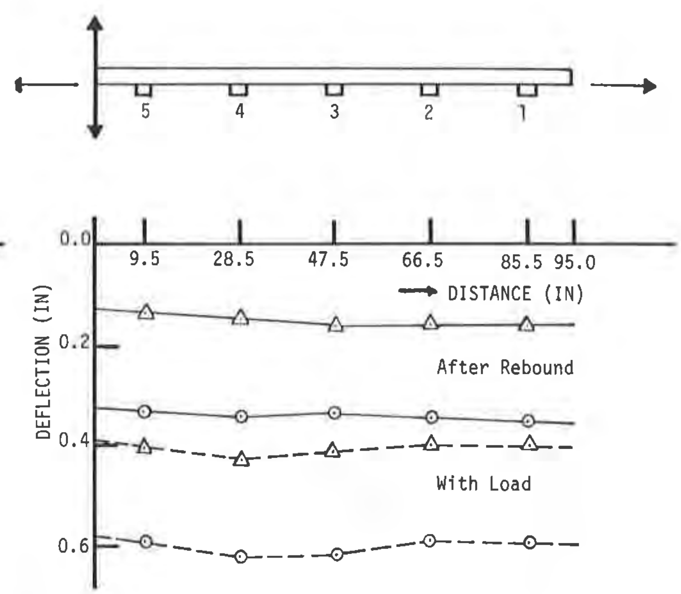

Figure 8. Maximum Deflection and Rebound After Twelve Hours

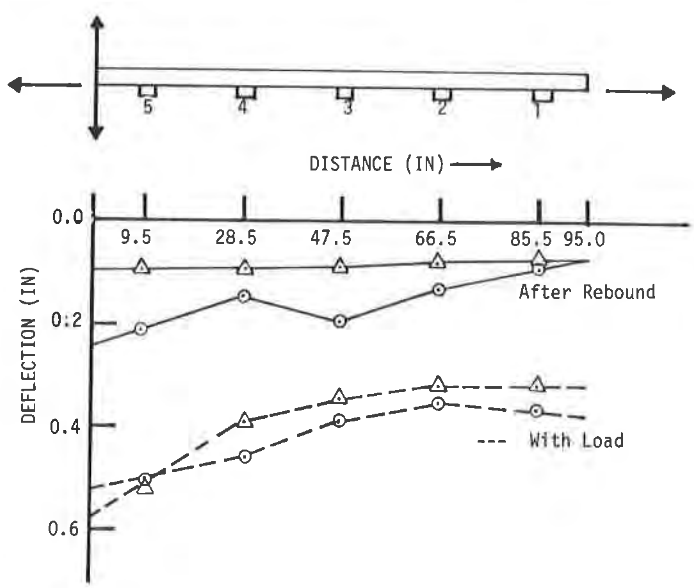

Figure 9. Maximum Deflection and Rebound After Twenty Four Hours.

PLEASE NOTE FOR ALL FIGURES ON THIS PAGE

△ Tests With Fabric

⊙ Tests Without Fabric

1 in. = 2.54 cm

1 lb. = 4.45N

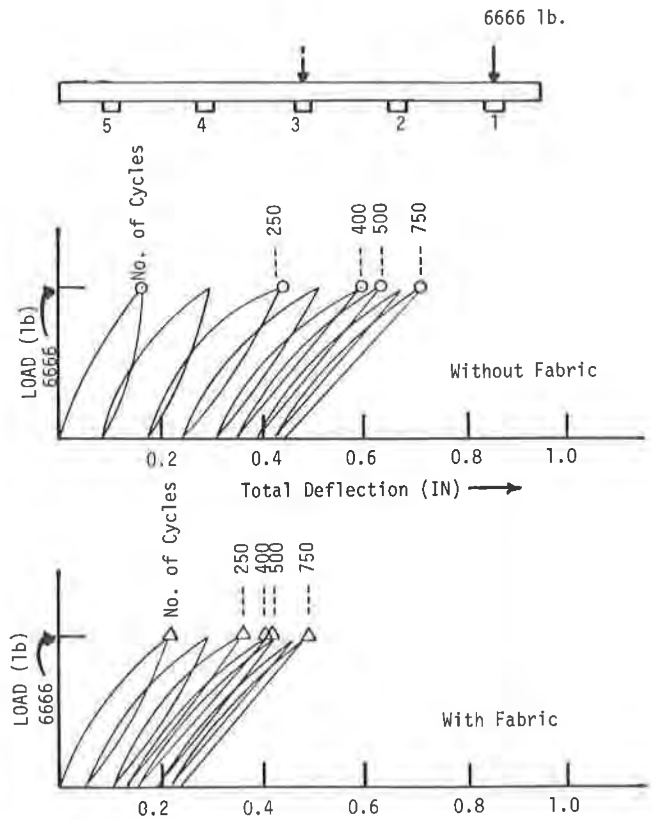

Figure 10. Total Deflection at Position 1 at Different Cycles

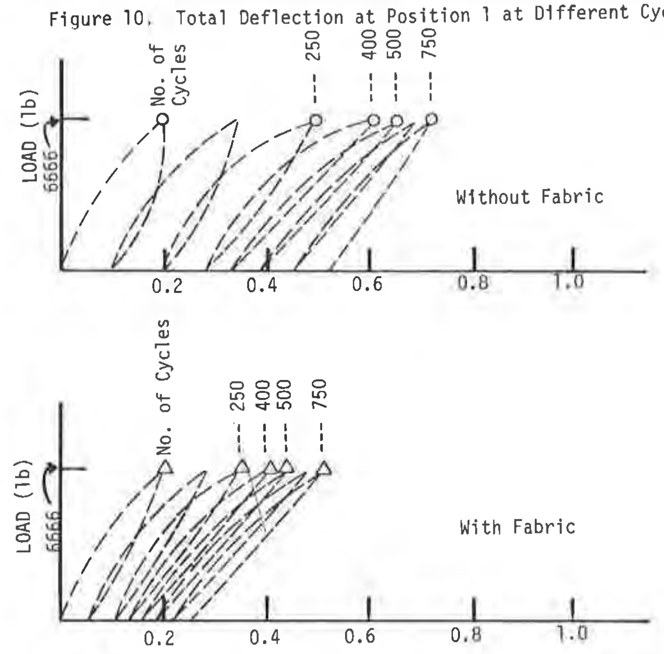

Figure 11. Total Deflection at Position 3 at Different Cycles.

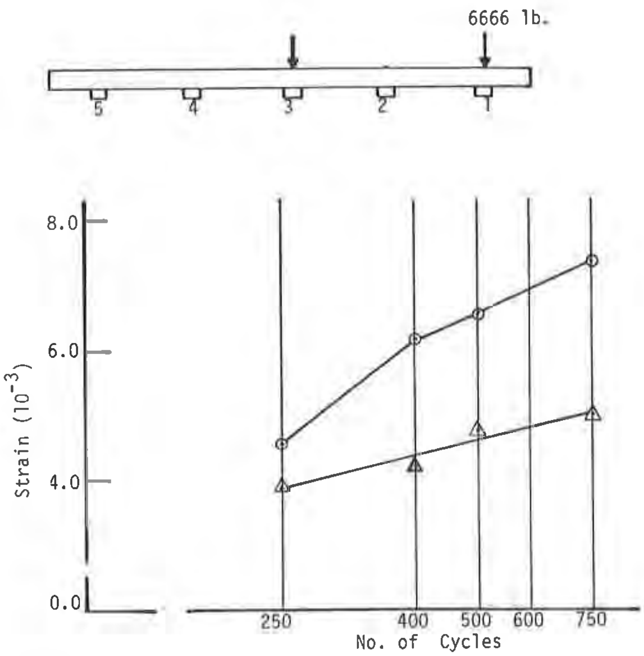

Figure 12. Strain at Position 1 Versus No. of Cycles

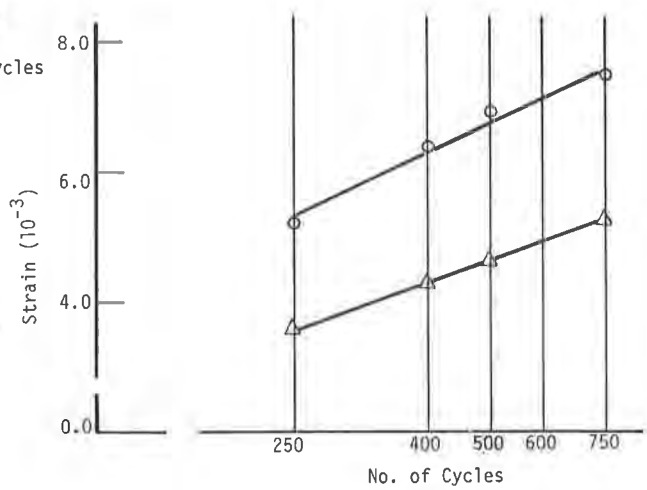

Figure 13. Strain at Position 3, Versus No. of Cycles

PLEASE NOTE FOR ALL FIGURES ON THIS PAGE

△ Tests With Fabric

⊙ Tests Without Fabric

1 in = 2.54 cm

1 lb = 4.45N

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Using a Geotextile to Prevent Shrinkage Crack of Rigid Pavements

Utilisation de géotextiles pour éviter la remontée des fissures des chaussées à assises rigides

ABSTRACT

In order to prevent the propagation of the shrinkage cracks of the road courses treated with hydraulic binders in the wearing courses made in bituminous concrete, a geotextile impregnated with bitumen was inserted between the cracked course and wearing course.

Five experimental fields corresponding to various climates and traffics were attended since 1977. The results seen on the fields were completed by laboratory trials.

The conclusion at the present time is that the crack propagation is in fact at least one year belated. Inversely if the geotextile does not suit perfectly well, because too compressible, the wearing course is abnormally acted upon then fatigue accelerates the damages. Additional studies are going to be carried out in order to develop a geotextile corresponding to the problem to be solved and to define nature and proportions of the impregnation bituminous binder

RESUME

Pour éviter la remontée de fissures de retrait thermique d'assises de chaussées traitées par des liants hydrauliques dans les couches de roulement en béton bitumineux, on a interposé entre la couche fissurée et la couche de roulement un géotextile imprégné de bitume.

Cinq chantiers expérimentaux correspondant à des climats et des trafics différents ont été suivis depuis 1977. Les constatations faites sur chantier ont été complétées par des essais en laboratoire.

La conclusion actuelle est que cette méthode retarde effectivement la remontée des fissures au minimum de 1 an. Par contre si le géotextile est mal choisi (trop compressible) la couche de roulement est anormalement sollicitée et sa dégradation par fatigue est accélérée. Des études complémentaires sont à faire pour mettre au point un géotextile adapté au problème à résoudre et pour définir la nature et le dosage du liant bitumineux d'imprégnation.

1 - INTRODUCTION

La France utilise dans une proportion importante pour la construction et le renforcement de ses routes les plus circulées, des assises en granulats traités par des liants hydrauliques ou pouzzolaniques recouvertes par une couche de roulement en béton bitumineux. Ces assises rigides à modules d'élasticité élevés sont le siège d'une fissuration de retrait thermique systématique et inéluctable pour le climat français.

Sur un plan mécanique l'effet nocif de cette fissuration est pris en compte dans le dimensionnement.

Ces fissures se transmettent à la surface de la chaussée d'autant plus vite que l'épaisseur de la couche de roulement est plus faible. Leur apparition à la surface de la chaussée est une cause de la pénétration d'eau dans le corps de la chaussée; elle entraine quelquefois des dégradations du béton bitumineux.

Pour les routes importantes, l'entretien consiste soit à boucher les fissures par un produit adapté soit à réaliser une nouvelle couche de roulement.

Compte tenu de l'importance du problème pour la France, la mise au point de techniques permettant d'éviter ou de retarder l'apparition en surface de ces fissures de retrait a fait l'objet de nombreuses recherches. Les études ont porté sur la modification de la composition du béton bitumineux, sur l'interposition de membranes, sur la désolidarisation de la couche de roulement au droit de la fissure etc..Parmi toutes ces techniques, l'interposition entre la couche fissurée et la couche de roulement d'un géotextile imprégné de bitume a fait l'objet des essais les plus importants.

2 - FONCTIONNEMENT DES STRUCTURES PARTIELLEMENT FISSUREES

2.1. PROCESSUS DE REMONTEE DES FISSURES

Le mécanisme de propagation des fissures transversales de retrait des chaussées à assises traitées aux liants hydrauliques est fortement régi par la nature des liaisons à l'interface couche fissurée couche supérieure.

Le schéma de la figure 1 présente les divers cheminements rencontrés.

Le point de départ est la fissuration de la couche de base. Ensuite sous l'action conjuguée des sollicitations d'origine thermique et du trafic, la fissuration peut se développer soit directement vers la surface et aboutir à une structure fissurée avec interface collée, soit au niveau de l'interface. Il y a alors décollement des couches et un processus de fatigue par flexion de la couche de roulement au cours duquel prend naissance une nouvelle phase de propagation verticale.

Un tel scénario conduit à une structure fissurée avec destruction des liaisons à l'interface.

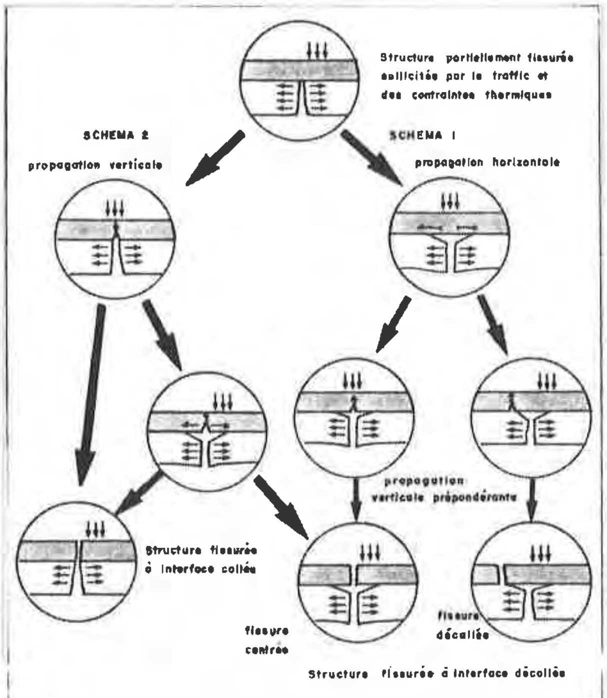


Fig. 1 Mécanisme de fissuration d'une structure de chaussée : cheminements types.

2.2. MODELISATION DU MECANISME DE PROPAGATION

Les concepts de la mécnique de la rupture permettent de décrire ce mécanisme de propagation et d'évaluer l'influence des paramètres qui gouvernent le processus. La notion de force d'extension de fissure permet de quantifier les efforts qui tendent à propager une fissure dans une direction donnée.

Une fissure ayant atteint l'interface couche de base - couche de roulement peut se propager verticalement ou horizontalement sous l'effet de sollicitations d'origine thermique - G et R étant respectivement la force d'extension de fissure et la résistance à la fissuration dans une direction donnée, la direction de propagation privilègiée sera celle qui présente le rapport G/R le plus élevé.

Le temps de remontée de la fissure en surface est fonction de la longueur du cheminement de cette fissure.

2.3. INCIDENCE D'UNE MODIFICATION D'INTERFACE

Plusieurs voies permettent un accroissement du temps de remontée de la fissuration :

. viser le cheminement de la fissure le plus long possible en privilègiant une propagation horizontale à l'interface.

. freiner la propagation verticale en interposant un matériau, soit très difficilement fissurable, soit qui découple les deux couches de manière à stopper la fissure en cours.

Dans presque tous les cas une phase d'initialisation de fissure, en principe relativement longue, est introduite dans le processus.

La technique d'interposition d'un complexe (géotextile + liant) dont le module d'élasticité est inférieur à celui des couches de la chaussée répond assez bien à ces critères. La propagation horizontale est privilègiée dans la mesure où l'anisotropie du matériau conduit à un rapport G_H / $R_H > Gv/Rv$ dû à l'amélioration de la résistance à la propagation verticale dans le géotextile alors que les liaisons interfaciales sont inchangées. La petitesse relative du module d'élasticité assure bien une désolidarisation puisque la force Gv tendant à propager la fissure verticale dans la couche de roulement est d'autant plus faible que le complexe est souple.

Quel que soit le procédé utilisé, il ne doit pas mettre en cause la perennité de la structure sous les sollicitations du trafic. C'est ainsi que l'efficacité d'un découplage par l'introduction d'un matériau de faible module doit être limitée pour éviter une trop grande fatigue par flexion de la couche de roulement.

3 - LES EXPERIMENTATIONS SUR CHAUSSEES

Des expérimentations d'interposition de géotextile entre la couche fissurée ou susceptible de se fissurer et la couche de roulement ont été entreprises dès 1977.

Les chantiers réalisés ont précédé les études en laboratoire car :

. Le problème important pour la France devait être résolu rapidement et la réalisation d'expérimentations précoces permettait de gagner plusieurs années pour le suivi du comportement sur site.

. Les expériences permettaient de mieux cerner les paramètres à étudier en laboratoire.

3.1. DESCRIPTION DES EXPERIMENTATIONS

Parmi tous les chantiers réalisés, cinq ont fait l'objet d'expérimentations précises. Les chantiers expérimentaux ont été réalisés en deux temps :

. des chantiers préliminaires (1977) ont permis de définir les problèmes de mise en oeuvre des géotextiles sur chaussées (chantiers RN 79 et RN 80)

. les expérimentations (1978) sur trois sites choisis en fonction de leur diversité (trafic moyen et hiver rigoureux, trafic lourd et hiver rigoureux, trafic lourd et hiver moyen) : ce sont les chantiers CD 978, RN 4, RN 10 .

Tous ces chantiers sont définis dans le tableau 1

Tableau 1 - Caractéristiques des chantiers expérimentaux

Nature et date du chantier	Longueur de l'essai	Géotextiles utilisés	Dosage en émulsion de bitume à 60 %	Couche de roulement sur géotextile
RN 79 Chaussée neuve Juillet 1977	480 m	Bidim U 24 Bidim U 34	0,750 Kg/m2 1 kg/m2	Béton bitumineux 0/10 mm épaisseur 6 cm
RN 80 Renforcement Septembre 1977	350 m	Bidim U 34 (270g/m2)	0,440 Kg/m2	Béton bitumineux 0/10 ou 0/14 mm épaisseur 6cm et 9cm
CD 978 Renforcement Eté 1978	300 m	Bidim U 24 Bidim U 44 (340g/m2)	1,250 Kg/m2 2 Kg/m2	Béton bitumineux 0/10 mm épaisseur 6 cm
RN 4 Renforcement Novembre 1978	600 m sur 1/2 chaussée	Bidim U 24 (210g/m2) Terram (210g/m2)	variable 1,2 à 1,8 Kg/m2	Béton bitumineux 0/14 mm épaisseur 7 cm
RN 10 Renforcement Novembre 1978	550 m	Bidim U 24 (210g/m2) Bidim U 34	1,2 Kg/m2 1,2 Kg/m2	Béton bitumineux 0/14 mm épaisseur 8 cm

3.2. CONSTATATIONS A LA MISE EN OEUVRE DU GEOTEXTILE

La mise en oeuvre manuelle du géotextile (les expérimentations étant trop restreintes pour justifier un matériel spécialisé) a quelquefois provoqué une insuffisance de la mise en tension de la nappe de géotextile : difficultés de guidage, tendance à la formation de plis entrainant la réalisation de joints avec recouvrement après découpe (figure 2).Il faut rappeler que les chantiers réalisés sont des chantiers de renforcement sans interruption de la circulation.

Le géotextile a été mis en place après répandage sur la chaussée d'une couche d'émulsion (de l'ordre de 1,2 à 1,4 kg/m2 d'émulsion à 60 % de bitume pour un géotextile de 210 g/m2. Pour une partie du chantier de la RN 4, le dosage a été porté à 1, 8 kg/m2 d'émulsion. Dans ce cas le liant a traversé le matériau sous la pression des pneumatiques des camions approvisionnant le chantier. Ceci a provoqué un collage de la nappe aux pneumatiques qui a été réduit par le jet de pelletées d'enrobé dans les traces du véhicule.

Dans tous les autres cas la réalisation du chantier n'a pas posé de problèmes particuliers et la circulation s'est faite sans incident sur le géotextile.(Fig. 3)

4 - BILAN DES EXPERIMENTATIONS

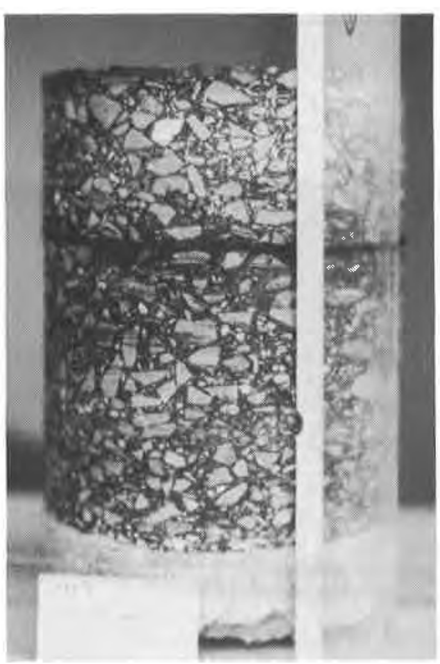
4.1. CONSTATATIONS IN SITU

Préalablement à la réalisation des expérimentations d'interposition de géotextile, un relevé précis des caractéristiques mécaniques des zones a été entrepris: position et forme des fissures, mesures de battements des lèvres des fissures, mesures de déflexion.

Ce point zéro a permis de constater que l'interposition de géotextile de type Terram ou Bidim (210 g/m2) ne majore pas significativement les déflexions pour des épaisseurs de béton bitumineux de 6 à 8cm Seuls des essais en laboratoire peuvent permettre d'évaluer les différences apportées par cette interposition.

La réalisation de carottages a permis de constater que le géotextile était presque toujours imprégné par le bitume : ramollissement du bitume de l'émulsion par la chaleur à la pose de l'enrobé et sans doute diffusion du bitume de ce dernier dans le géotextile.

En général, le géotextile adhère bien à la nouvelle couche d'enrobé. Le collage avec l'ancienne chaussée est plus difficile à évaluer; l'opération de carottage introduisant des efforts de cisaillement non négligeables.



Sur le chantier de la RN 4, il apparait que des dosages en bitume supérieurs ou égaux à 0,8 Kg/m2 conduisent à une liaison résistant au carottage (Fig. 4) Dans ce cas, le géotextile a été placé entre la couche de roulement d'origine dans laquelle la fissuration de la couche de base était remontée et la couche de roulement d'entretien.

Sur le chantier du CD 978, un bon collage est obtenu par un dosage en bitume de 1,2 Kg/m2. Dans ce cas, le géotextile a été placé entre la couche de base et la couche de roulement. (Fig.5)

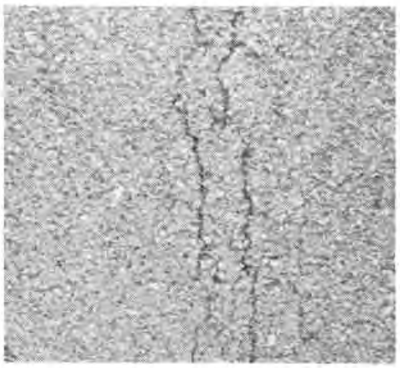
Le suivi dans le temps des diverses expérimentations, à l'exception des deux premières réalisées en 1977 et ayant servi de mise au point, permet de dégager les tendances provisoires suivantes 3 ans après la mise en oeuvre.

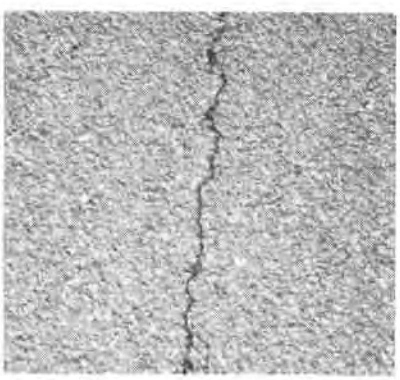
L'interposition de géotextile conduit à différer systématiquement l'apparition des fissures en surface au minimum de 6 mois à 1 an par rapport aux zones non traitées.

Les fissures réapparaissant à travers le géotextile intéressent rarement toute la largeur de la chaussée, elles se produisent le plus souvent uniquement sous les bandes de roulement de la circulation.

Les fissures réapparaissent d'autant plus facilement que le géotextile utilisé est épais et compressible et que la couche de béton bitumineux est mince. Sur la RN4, aucune fissure n'est apparue après 3 ans et demi sur la section traitée au Terram alors que les zones non traitées sont fissurées. Les fissures apparaissent plus vite à travers le Bidim U 44 (340 g/m2) que sur les zones traitées au Bidim U 24 (210 g/m2).

Les fissures réapparaissanten surface sont diffuses et ramifiées dans les zones où un géotextile a été interposé sous la couche de béton bitumineux (Fig. 6) Elles sont nettes et franches dans les zones témoins non traitées (Fig. 7)





4.2. ESSAIS REALISES EN LABORATOIRE

Compte tenu des constatations faites in situ sur les chantiers expérimentaux, des carottages ont été réalisés sur le CD 978 afin d'effectuer des essais complémentaires. Les essais ont été faits sur trois types d'échantillon :

- Carottage réalisé dans une zone témoin sans géotextile
- Carottage réalisé dans une zone traitée avec bidim U 24
- Carottage réalisé dans une zone traitée avec bidim U 34

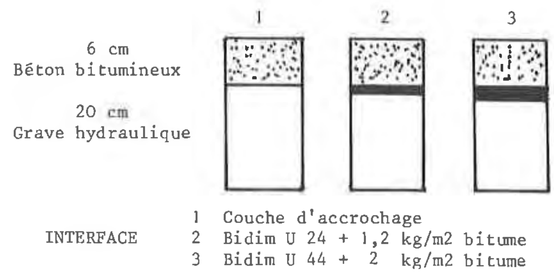


Fig. 8 : Carottages Ø 150 mm réalisés sur le CD 978

Ces échantillons ont été soumis à un essai de compression axiale à une température de 20 °C. Pour une pression de 100 Pa, les déformations mesurées sont les suivantes :

7 µm pour l'échantillon témoin N° 1
40 µm " " N° 2 (bidim U 24)
70 µm " " N° 3 (bidim U 44)

Les modules de déformations correspondants sont :

700 M Pa pour le témoin
80 M Pa pour l'échantillon avec bidim U 24
50 M Pa " " " " U 44

L'interposition sous la couche de roulement d'un géotextile imprégné de bitume revient donc à ajouter une couche à très faible module de déformation en compression. Ce module est d'autant plus faible que le géotextile est compressible. Il dépend aussi du pourcentage de remplissage des vides du géotextile par le bitume d'imprégnation et des caractéristiques du bitume.

La déformation en compression du géotextile se traduit par une augmentation des contraintes d'élongation par flexion à la base de la couche de roulement, ce qui a un effet négatif sur la tenue en fatigue de cette couche de roulement.

5 - CONCLUSIONS - PERSPECTIVES

L'utilisation de plus en plus importante d'assises de chaussées en matériaux traités aux liants hydrauliques sur les routes françaises les plus circulées a conduit à de nombreuses expérimentations pour éviter que se propagent en surface les fissures de retrait thermique inévitables avec de tels matériaux.

L'interposition entre la couche fissurée et la couche de roulement d'un géotextile imprégné de bitume a donné lieu à de nombreuses expérimentations car cette solution trouve des justifications théoriques.

Les chantiers réalisés ont montré qu'un bon collage du géotextile à la fois à la couche inférieure et à la couche de roulement pouvait être obtenu par un dosage en bitume de 1 kg/m2 avec un géotextile d'un grammage voisin de 200 g/m2.

Les quelques difficultés de mise en oeuvre rencontrées devraient être résolues par la mécanisation de la pose du géotextile, ce qui par ailleurs réduira les coûts.

L'efficacité de l'interposition d'un géotextile sur la vitesse de remontée des fissures est prouvée. Le recul insuffisant que nous avons actuellement sur les chantiers expérimentaux ne permet pas de dire de combien pourrait être retardée cette remontée de fissure dans le cas le plus favorable.

Il est indispensable d'utiliser un géotextile peu compressible sous peine de voir apparaitre à la base de la couche de béton bitumineux des déformations anormales qui diminuent fortement la tenue en fatigue du béton bitumineux et rendent le traitement plus nocif qu'utile.

Il est donc maintenant nécessaire de poursuivre les essais et expérimentations pour :

. Mettre au point un géotextile bien adapté au problème à règler. Ce produit devra être peu compressible sans être trop rigide pour ne pas rendre sa mise en oeuvre difficile. Il devra en outre avoir une bonne compatibilité avec le bitume et résister aux températures de mise en oeuvre des enrobés bitumineux.

. Mieux définir les dosages et la nature des liants bitumineux à utiliser pour le collage.

Une méthode d'étude par simulation du trafic en laboratoire sur des plaques de faible dimension a été mise au point. Elle permettra d'étudier la variation de nombreux paramètres dans un délai suffisamment court.

Les constatations faites à plus long terme sur des chantiers expérimentaux qui seront réalisés avec des produits bien adaptés, devraient montrer si le retard obtenu dans la remontée en surface des fissures justifie le coût de l'interposition d'un géotextile entre les couches de roulement et les assises traitées aux liants hydrauliques.

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The Use of Geotextiles in Flexible Pavement Surface Dressing

L'emploi des géotextiles dans les enduits superficiels sur les chaussées souples

RESUME

La communication traite de travaux de recherches sur l'amélioration par géotextiles de la technique des enduits superficiels sur chaussées souples à faible ou moyen trafic. La technique normale est de fixer une mosaïque de granulats sur la surface de la couche granulaire au moyen d'un liant bitumineux. Cependant ces traitements se détériorent souvent rapidement par l'apparition de fissures et de nids de poules. On peut agir sur les mécanismes de dégradations en incorporant un géotextile non-tissé dans l'enduit superficiel pour lui donner une résistance à la traction et améliorer son adhérence à la couche sous-jacente.

La communication présente les résultats de la recherche effectuée par les Ponts et Chaussées avec la collaboration d'un producteur de géotextile (Sommer) dans le cadre d'un programme de la Division des Affaires Economiques et Internationales visant à l'amélioration des techniques routières pour les pays en voie de développement.

ABSTRACT

The paper reports on research and development work on the improvement with geotextiles of the surface dressing technique used on flexible pavements for low and medium traffic roads. The standard technique is to set on the untreated gravel pavement layer a mosaïc of stone chippings bound with a bituminous binder. However such surface treatments often deteriorate rapidly due to the development of cracks and potholes. Deterioration mechanisms may be acted upon through the incorporation of a non-woven geotextile in the surface dressing to give it traction resistance and to improve its adhesion on the gravel layer.

The paper gives the results of research conducted by the "Ponts et Chaussées" with the cooperation of a geotextile producer (Sommer) under a program of the "Division des Affaires Economiques et Internationales" aimed at improving road construction techniques for developing countries.

1 - INTRODUCTION

Le développement des géotextiles s'est fait initialement et continue surtout à se faire dans le domaine des terrassements, du drainage et des ouvrages hydrauliques. On trouve en effet dans ces applications l'occasion de mettre à profit à la fois les avantages mécaniques et hydrauliques de l'association des sols et des textiles. De plus, sur le plan mécanique on se trouve souvent, comme dans le cas des pistes de chantier, dans des conditions où les déformations appliquées au système sol-géotextile sont suffisamment grandes pour faire jouer pleinement le rôle de renforcement du textile.

Les applications possibles dans le domaine des chaussées routières proprement dites à moyen ou fort trafic ne peuvent pas faire appel aux mêmes mécanismes. En effet les couches de fondation, de base et de roulement, pour les différentes techniques de chaussées couramment utilisées, travaillent avec des déformations trop faibles pour que le rôle direct d'armature du géotextile puisse vraiment être mis à profit. Par ailleurs si les capacités filtrantes et drainantes des géotextiles sont fort utiles dans les systèmes de drainage situés le long des chaussées, les géotextiles eux-mêmes n'ont pas leur place en tant que drain dans les chaussées elles-mêmes où les mouvements d'eau éventuels mettent en jeu des débits très faibles (sauf peut-être en cas de revêtement très fissuré) et où un géotextile poreux introduirait un élément de faiblesse mécanique inacceptable.

Le fait que les terrassements routiers aient été un domaine privilégié de développement des géotextiles, au moins dans certains pays et notamment en Europe, a conduit cependant les ingénieurs routiers à s'interroger sur leurs utilisations possibles en chaussée.

Une première application est la lutte contre la fissuration des couches de roulement, qui a été expérimentée depuis longtemps aux Etats-Unis avec des géotextiles non-tissés et en Europe depuis longtemps aussi avec une grille très rigide à laquelle on pensait donner un rôle de renforcement autant que d'interface et plus récemment avec des non-tissés. Cette application est l'objet d'une communication présentée à ce congrès par MM. Colombier, Astesan et Goacolou et l'on ne s'y étendra donc pas ici.

2 - LES ENDUITS SUPERFICIELS

L'application dont traite la présente communication est le renforcement des enduits superficiels des chaussées souples. On entend par chaussées souples les chaussées construites en grave non traitée par opposition aux chaussées constituées de matériaux liés au ciment, au laitier, au bitume ou tout autre liant donnant à la stucture une forte rigidité. Les chaussées souples se distinguent des chaussées dites semi-rigides et des chaussées en béton par une plus forte déflexion sous la charge des véhicules ; elles sont essentiellement utilisées pour les trafics moyens ou faibles. Pour ces trafics une couche de roulement en enrobés bitumineux n'est souvent pas justifiée économiquement et la solution utilisée pour imperméabiliser la surface et obtenir des conditions de roulement et d'adhérence satisfaisantes est l'enduit superficiel.

La technique de l'enduit superficiel consiste à fixer par un liant bitumineux, à la surface de la couche granulaire compactée, une mosaïque de granulats d'un calibre de l'ordre de 10 mm. L'enduit est monocouche ou bicouche selon que le bitume et les granulats sont répandus en une fois ou en deux fois. La quantité de bitume est de l'ordre de 1 kg/m² pour les monocouches et 2 kg/m² pour les bicouches.

L'épaisseur moyenne de liant correspondante est donc d'environ 1 ou 2 mm. Le bitume peut être mis en oeuvre à chaud ou à froid (émulsion). De bonnes conditions d'adhésivité entre le liant et les granulats sont un élément fondamental de la tenue d'un enduit superficiel.

Cette technique est très largement utilisée et l'on peut dire qu'elle est le revêtement routier le plus répandu dans un grand nombre de pays, aussi bien développés qu'en voie de développement. En plus de son caractère économique et de ses performances, elle demande des moyens de mise en oeuvre représentant un investissement moins élevé que les enrobés bitumineux.

Cependant cette technique demande beaucoup de soin à l'exécution et sa réussite dépend de nombreux facteurs liés aux caractéristiques de la couche support, au liant, aux granulats de l'enduit lui-même, aux conditions météorologiques d'exécution et à la qualité de la mise en oeuvre elle-même. La durée de vie des enduits superficiels est donc inégale. Parmi les dégradations que l'on peut souvent constater on peut citer les fissures et les "nids de poules". Les fissures résultent de la fatigue de la chaussée et des déformations trop importantes que subit l'enduit au passage du trafic ; un enduit fissuré ne joue plus son rôle de protection contre l'eau et les bords de fissures sont l'amorce de dégradations plus étendues. Les nids de poules ont pour origine le départ localisé des constituants de l'enduit ayant une adhérence insuffisante, suivi par une érosion des matériaux sous-jacents due au trafic. Un autre phénomène de vieillissement est l'enfoncement des granulats dans le support, ce qui dépend beaucoup de la nature de ce dernier.

Compte tenu de ces différents modes de dégradation, il apparaît que l'amélioration de la tenue dans le temps d'un enduit superficiel est à rechercher :

- dans une meilleure résistance de l'enduit dans son plan pour résister à l'apparition et à la propagation des fissures ;
- dans un meilleur accrochage sur le support ;
- dans l'emploi de dispositions s'opposant à l'enfoncement des granulats.

On peut penser, notamment à partir de certaines constatations déjà faites, que ces trois résultats puissent être atteints simultanément en incorporant à l'enduit superficiel un non-tissé approprié.

3 - LE RENFORCEMENT CONTRE LA FISSURATION SUR SUPPORT DEFORMABLE

Il est d'abord logique que la présence d'une nappe de fibres au sein de l'enduit lui donne une grande résistance à la fissuration par suite de la résistance à la traction des fibres. Ceci a été vérifié par des essais réalisés en 1980 au Centre d'Expérimentations Routières de Rouen des Laboratoires des Ponts et Chaussées.

On a réalisé un massif de limon peu plastique mis en place à une teneur en eau conduisant à un indice C.B.R. immédiat de 10. Sur ce massif a été placée une couche de 35 cm de grave naturelle 0/15 mm d'E.S. = 20 compactée à 98 % de l'O.P.M. Un enduit superficiel bicouche à l'émulsion de bitume (2,5 kg/m² d'émulsion à 60 % de bitume 180/220) a été réalisé, la moitié de la longueur (8 m) étant non renforcée et l'autre moitié renforcée par un non-tissé aiguilleté de filaments continus polyester de 210 g/m². Le module à la plaque EV_2 après réalisation de l'enduit était d'environ 20 MPa, le coefficient de restitution à la Dynaplaque (Fig. 1) étant compris entre 0,20 et 0,25.

Figure 1 - Mesure de la déformabilité du sol-support avant mise en oeuvre de la couche granulaire et de l'enduit superficiel. Mesures à la Dynaplaque (chargement dynamique sur plaque 600 mm de diamètre).

Measurement of the deformability of the subgrade before construction of the granular layer and of the surface dressing. Measurements with the Dynaplaque (dynamic loading on a 600 mm diameter plate).

Cette planche d'essai a été soumise à des passages de camion circulant à 20 km/h environ, l'essieu arrière étant chargé à 13 tonnes.

On a constaté l'apparition de fissures sur l'enduit non renforcé dès le début du trafic (à cause du faible module de la couche) alors qu'il n'y avait pas de fissure sur l'enduit renforcé (Fig. 2, 3 et 4).

Figure 2 - Enduit superficiel bicouche sur une couche de 30 cm de grave argileuse peu plastique (IP = 8) placée sur un sol support déformable. Fissures formées après huit passages d'un camion chargé à 13 tonnes par essieu.

Double layer surface dressing over a 30 cm layer of low plasticity (IP = 8) clayey gravel, laid over a deformable subgrade. Cracks formed after eight passes of a truck with a 13 tonnes axle load.

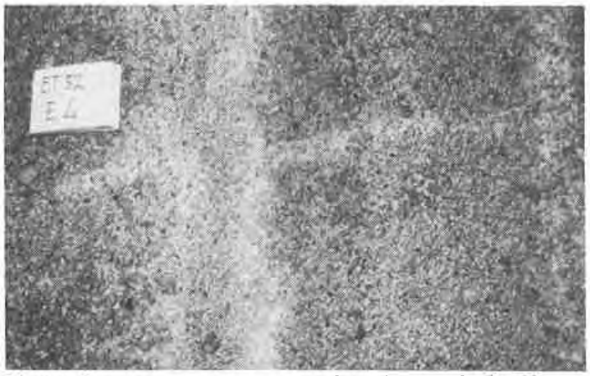

Figure 3 - Mêmes matériaux que dans le cas de la figure 2 mais un géotextile (non-tissé aiguilleté de filaments continus de polyester) a été incorporé à l'enduit superficiel. Aucune fissure n'est apparue après 52 passages de camion.

Same materials as in fig. 2, except that a geotextile (spunbonded needlepunched polyester non-woven) has been incorporated in the surface dressing. No cracks after 52 passes of truck.

Figure 4 - Vue des déformations de surface correspondant au cas de la figure 3, à 52 passages de camion également.

View of the surface deformations for the same situation as shown in fig. 3, also after 52 passes of truck.

Cet enduit renforcé à fini par périr sous l'effet de l'orniérage de plus en plus important de la couche testée, par une déchirure brutale d'une longueur de 2 m environ, alors que l'enduit non renforcé était déjà complètement détruit. Un aspect important de cette différence de mode de rupture est que, tant que l'enduit renforcé n'était pas rompu, il assurait toujours son rôle de protection contre l'eau.

L'emploi d'une nappe textile a donc, sur la cohésion de l'enduit, un effet considérable obtenu probablement de façon plus rationnelle et moins coûteuse que pas l'augmentation de l'épaisseur de l'enduit (enduit multicouche), dont cette meilleure cohésion est un des effets.

Pour obtenir ce résultat une nappe non-tissée est préférable à une nappe tissée car elle contient des fibres orientées dans toutes les directions et donne donc une résistance à l'enduit dans toutes les directions. Le module souvent faible de ces nappes n'est pas un inconvénient car c'est le fonctionnement du composite fibre-bitume qui entre en jeu. En revanche, l'emploi de tissés risque de conduire à un résultat plus anisotrope en déformabilité, ce qui peut être un inconvénient.

Il faut cependant noter que si l'incorporation d'un géotextile à l'enduit a accru sa résistance à la fissuration, elle a augmenté aussi la nécessité d'un bon accrochage pour que les contraintes de traction qui apparaissent dans l'enduit ne provoquent pas son décollement. L'étape suivante de la recherche concerne donc l'accrochage.

4 - AMELIORATION DE L'ACCROCHAGE DE L'ENDUIT

L'accrochage d'un enduit renforcé ayant une certaine résistance à la traction est un phénomène complexe car il s'agit de donner à la liaison support-enduit à la fois une résistance à l'arrachement et une résistance au cisaillement. L'incorporation d'un géotextile à l'enduit modifie son comportement en traction (résistance, module, isotropie) et modifie par conséquent la transmission par l'enduit lui-même au support des efforts tangentiels imposés par les véhicules. Cette transmission sa faisant par l'intermédiaire de la liaison enduit-support, celle-ci doit être adaptée à ses nouvelles fonctions. Si l'on compare l'enduit renforcé au cas limite de l'enduit dans lequel chaque granulat serait fixé au support indépendamment de ses voisins, la différence est que dans le cas de l'enduit renforcé des efforts tangentiels sont transmis à une certaine distance de la zone de contact roue-enduit, alors qu'il n'y a pas de transfert de contrainte à l'extérieur de cette zone dans le cas simplifié des granulats indépendants. Cette différence est importante car en dehors de la zone chargée il doit pouvoir y avoir une certaine résistance au cisaillement alors même qu'il n'y a pas de charge normale appliquée directement.

Les mécanismes évoqués ci-dessus ne sont pas encore très bien connus en ce sens qu'ils n'ont été l'objet que de fort peu de mesures. Ils suggèrent néanmoins trois conclusions :

a) la résistance à l'arrachement est toujours un élément essentiel, qu'il y ait géotextile ou non ;

b) la résistance au cisaillement de la liaison enduit-support doit être particulièrement forte dans le cas de l'enduit renforcé pour résister aux efforts de cisaillement à l'extérieur de la zone chargée ;

c) le bon comportement de l'ensemble du système exige qu'un bon rapport existe entre le module de l'enduit renforcé, les caractéristiques de la liaison et les propriétés mécaniques du support.

L'étude expérimentale conduite au Centre d'Expérimentations Routières de Rouen a donc été poursuivie pour rechercher des produits tels que leur liaison avec le support présente une bonne résistance au cisaillement. Cette recherche a été faite dans le cadre d'un programme associant les Laboratoires des Ponts et Chaussées, un producteur de géotextile et les services du Ministère (Service d'Etudes Techniques des Routes et Autoroutes et Direction des Affaires Economiques et Internationales) ayant vocation à susciter et financer des actions de recherche et de développement portant sur les problèmes routiers des pays en voie de développement, en vue d'apporter à ces problèmes des solutions techniques nouvelles adaptées à leurs besoins et aux conditions d'exécution des travaux que l'on y rencontre.

L'étude a consisté à tester au cisaillement sur un même support toute une série d'échantillons prototypes comportant différents dispositifs d'accrochage au support. Elle a permis de voir à la fois les problèmes de fabrication que poserait leur production et de comparer leur comportement dans des essais de cisaillement de petite dimension.

La résistance à l'arrachement n'a pas été étudiée directement car la présence du géotextile ne doit pas modifier les conditions d'adhérence obtenues sans renforcement ; par ailleurs les procédés d'amélioration du comportement au cisaillement ne peuvent qu'améliorer la résistance à l'arrachement.

Dans ces essais un échantillon d'enduit superficiel renforcé de dimension 0,60 m x 0,35 m est soumis à un effort normal connu et sollicité en traction (Fig. 5). L'enregistrement de la courbe effort-déplacement permet l'analyse du mécanisme de rupture du système enduit renforcé-sol support.

Le sol-support est un sable 0/10 mm contenant 8 à 9 % de fines peu plastiques (IP = 10).

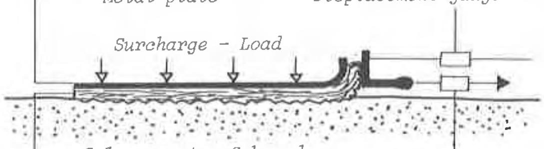

Figure 5 - Essais sur échantillon
Sample tests

La mise en oeuvre de l'enduit superficiel comporte les opérations suivantes :

- scarification du sol support ;
- répandage d'une première couche de liant (émulsion cationique à 65 % de bitume dosée à 1 kg/m² d'émulsion) ;
- pose du géotextile ;
- deuxième couche de liant semblable à la première ;
- répandage de granulats 10/14 mm au dosage de 10 l/m² ;
- troisième couche de liant semblable ;
- répandage de granulats 4/6 mm au dosage de 7 l/m² ;
- compactage de l'ensemble au marteau vibrant.

Un assez grand nombre de solutions d'accrochage ont été testées au moyen de ce dispositif et on a examiné en particulier la valeur de l'effort tangentiel lorsque le mouvement relatif support-enduit commence à être mesurable et la valeur maximale de cet effort associée au déplacement relatif correspondant.

Il résulte de ces essais que l'on peut améliorer de 40 à 70 % l'effort tangentiel transmissible du support par un enduit renforcé.

Le but de ces essais était cependant surtout de faire un choix pour fabriquer des géotextiles renfort d'enduit d'une dimension suffisante pour faire des essais en vraie grandeur avec circulation de véhicules. Ces essais sont actuellement en cours.

5 - APPLICATIONS

La recherche étant en cours, le procédé d'association d'un géotextile à un enduit superficiel n'est pas encore entré dans la pratique. On doit cependant s'interroger sur les conditions pratiques de mise en oeuvre et sur les conditions économiques.

Du point de vue de la mise en oeuvre on dispose de l'expérience acquise dans la réalisation d'interfaces antifissures sous enrobés, qui ont été faits sur un assez grand nombre de sections expérimentales. On sait à la suite de ces travaux que le déroulement d'un non-tissé aiguilleté ne pose pas de problème ; que sa perméabilité au bitume ou à l'émulsion est suffisante pour que le bitume traverse parfaitement un non-tissé de 2 à 300 g/m² ; que le mariage bitume-fibres est excellent (résultat connu aussi en étanchéité). On connaît également la quantité de bitume nécessaire pour l'imprégnation du géotextile. On sait enfin que pour des travaux à chaud il est prudent d'expérimenter avec des fibres de polyester.

Sur le plan économique, la justification du procédé dépend d'abord du bilan qui s'établira entre l'augmentation du coût de l'enduit et l'accroissement de sa durée de vie.

La différence de coût de l'enduit dépend essentiellement de deux facteurs :

- le coût du géotextile : celui-ci dépend du type retenu et notamment du dispositif d'accrochage utilisé : il peut être important par rapport à celui d'un enduit non renforcé et même le dépasser ; le faible coût de transport est cependant à rappeler, compte tenu du faible poids du matériau (entre 1 et 2 t pour 1 km de chaussée de 5 à 7 m de large) ; le coût de mise en oeuvre du géotextile est faible ;

- les modifications apportées à la formule d'enduit qui aurait dû être retenue sans la présence du géotextile. Par rapport à un enduit monocouche la quantité de bitume sera éventuellement un peu plus élevée, mais un enduit monocouche pourra peut-être suffire là où un bicouche aurait été nécessaire. Une économie est alors possible.

La justification du procédé pourra provenir ensuite d'une amélioration éventuelle de la fiabilité de la technique de l'enduit superficiel, dans la mesure où l'incorporation d'un élément produit industriellement et par conséquent lui-même très fiable peut rendre plus sûr le comportement de l'ensemble. Il est certain en effet que, dans d'autres domaines d'emploi, les géotextiles sont un élément régulateur de la qualité des travaux.

Enfin les essais relatés dans le chapitre 3 de cette communication ont montré qu'en enduit renforcé pouvait supporter des déflexions beaucoup plus élevées qu'un enduit non renforcé. Cette solution ouvrira donc peut-être la voie à des techniques de chaussées souples à faible trafic utilisant des matériaux considérés actuellement comme impropres, mais rendus acceptables par une association avec des géotextiles améliorant leur comportement en fatigue et réduisant l'orniérage ; ces techniques, qui sont également elles-mêmes au stade de la recherche, conduisant à des structures qui restent assez déformables, une couche de roulement tolérant ces déformations est nécessaire : l'enduit renforcé peut être la solution qui leur permette de devenir viables.

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Simulation Testing of Geotextile Membranes for Reflection Cracking

Essais de simulation de la propagation des fissures avec membranes en géotextile

The use of geotextiles to prolong the life of bituminous concrete overlays is rapidly increasing in the U.S. This market has attracted many manufacturers to introduce fabrics for the paving industry. A laboratory analysis has been completed which identifies the fabric properties which are most critical to performance. The analysis separates the fabric contributions into two main mechanisms; first, they provide a moisture barrier that protects the underlying pavement structure from further degradation, and second, they provide a measure of reinforcement to the overlay which characteristically is weak under tension. Properties critical to performance are dense fabric structure and high tensile modulus. The structure of the fabric determines the amount of tack coat required to develop an effective moisture barrier and the modulus is a direct indication of the reinforcement potential of the fabric. The conclusions of this study are that high modulus fabrics consisting of a dense structure comprised of many small denier filaments provides maximum resistance to reflection cracking.

L'emploi des matières géotextiles aux Etats-Unis, afin de prolonger la durée des revetements en béton bitumineux, augmente rapidement. Nombreux producteurs, attirés par ce marché, ont présenté leurs tissus à l'industrie de pavage. Une analyse laboratoire identifie les charactéristiques du tissu les plus critiques à la performance. L'analyse donne aux contributions éffectués par le tissu deux mécanismes: les tissus présentent une barrière contre l'humidité, qui protège la structure de pavée sous-jacente contre quelque dégradation de plus: et ils pourvoient, dans une certaine mesure, unreforcement au revêtement, lequel est typiquement faible en traction. Les propriétés les plus critiques au fonctionnement sont la structure du tissu et le module de traction. La structure du tissu détermine la quantité requise du ciment asphaltique pour achever une barrière éffective contre l'humidité; et le module sécant indique directement le renforcement potentiel que peut fournir le tissu. Cet étude conclut que les tissus à module plus élevé, consistants d'une structure épaise d'un grand nombre de filets fins, pourvoient une résistance maximale aux craquements de reflexion.

INTRODUCTION

Premature failure of bituminous concrete pavements has been a serious problem for highway engineers. Escalating petroleum and construction costs emphasize the need to maximize the service life of all road work. An area of major concern is the reflection cracking of relatively new asphaltic overlays. Many attempts have been made to solve this problem including the use of stress relieving layers, seal coatings and rubberized asphalts. A method developed to reduce reflection cracking is the addition of a geotextile in the pavement structure. This use of fabrics is rapidly increasing in the U.S. and has attracted many manufacturers to introduce fabrics for the paving industry. These fabrics differ significantly in polymeric composition, methods of construction, and fabric structure. The resulting fabric properties are equally variable. While there exists substantial literature on this application, the principal mechanisms of fabric reinforcement are not well understood; the author seeks to make a contribution in this area and identify the fabric properties which are critical to system performance.

OBJECTIVES

Most industry authorities agree that a major benefit from geotextiles is that, once saturated with an asphalt cement tack coat, they provide a moisture barrier which protects the underlying structure against further degradation. There is some disagreement, however, on whether a geotextile can mechanically reinforce the pavement structure. It has been suggested that high modulus fabrics provide a measure of tensile reinforcement to the characteristically weak asphaltic concretes similar to the function of reinforcing bars in portland cement concretes. Conversely, thick, low modulus fabrics are believed to function as a stress relieving layer allowing for relatively minor displacements between the overlay and the base foundation. The objectives of this study are to evaluate a variety of commercially available geotextiles and determine which fabric properties are most critical to performance. Due to the complexity of field evaluations, the majority of this study involves the development of controlled laboratory tests for geotextiles.

EXPERIMENTAL DETAILS

A complex relationship exists between the many environmental factors and pavement performance. To determine relative differences between geotextiles, tests were developed which separate moisture barrier characteristics from reinforcement characteristics. Overall fabric performance was based on both moisture barrier and reinforcement characteristics. The relative importance of the moisture barrier versus reinforcement characteristics cannot be fully assessed without an elaborate field testing program which is beyond the scope of this study. The variable and complex environment makes it difficult to develop a direct correlation between laboratory results and anticipated field performance. All laboratory test work will be further explained in detail in the following analysis. Table I describes all of the geotextiles used in this study.

TABLE I. GEOTEXTILES USED IN REFLECTION CRACKING STUDIES

FABRIC	TRADENAME/TYPE	BASIS WEIGHT
A	Reepav® — Spunbonded Polyester	$102/m^2$
B	Petromat*—Needle punched Polypropylene	$153\ g/m^2$
C	Bidim** — Needle punched Polyester	$203\ g/m^2$
D	Petromat*—Needle punched Polypropylene	$203\ g/m^2$
E	Mirafi 900X*** — Woven Polypropylene Polyester Fill	$164\ g/m^2$

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*Registered Trademark of Phillips Petroleum.
**Registered Trademark of Rhone-Poulenc-Textile.
***Registered Trademark of Mirafi, Inc.

HYDROSTATIC TESTING[1]

Moisture barrier characteristics were determined by subjecting the geotextiles, saturated with an asphalt cement, to a hydrostatic head. A schematic diagram of the test apparatus is included in Figure 1. The basic mechanics of the apparatus allows for the gradual raising of a hydrostatic head above the test specimen. The rate of head rise was controlled to between 5-10 cm/min. Specimen failure was defined as the average in head between the values shown when the first and third droplets appeared on the geotextile. This equipment was limited to a maximum head of 150 cm which is satisfactory for this application.

(1) All hydrostatic test work was conducted by Dr. R. D. Weimar at the Du Pont Spunbonded Products Research Laboratory in Wilmington, Delaware.

(2) Asphalt absorption testing conducted per Texas Department of Highways and Public Transportation Special Specification Item 3099.

(3) Hydrostatic testing per ASTM Test D.751-73

Figure 2 shows hydrostatic head at specimen failure as a function of asphalt level. First the maximum asphalt absorption[2] capability or saturation point was identified for each fabric. A series of hydrostatic tests[3] was conducted starting at the saturation point and proceeding through a set of decreasing levels of asphalt cement. Results were plotted indicating asphalt level versus hydrostatic head. Figure 2 shows values from several geotextiles plotted on the same graph with results indicating a significant difference among fabrics. All fabrics reached an asphalt level where only a marginal decrease in asphalt level caused a significant decrease in the moisture barrier. This asphalt level is defined as the threshold asphalt level.

The hydrostatic series of tests demonstrates that all the geotextiles tested will form an adequate moisture barrier provided that the threshold amount of asphalt cement tack coat is present. The exact amount of tack coat required varies depending on the individual geotextile.

Hydrostatic Testing Apparatus
(ASTM D751-73)

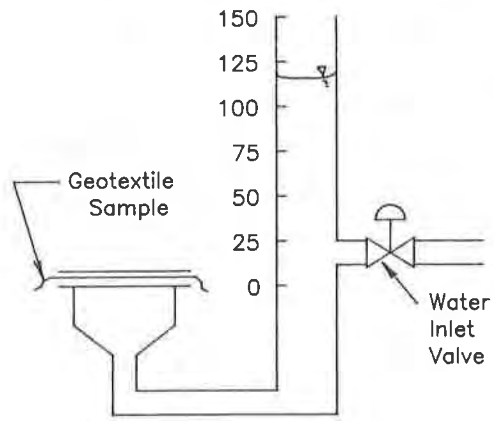

Figure 1

Resistance to Water Penetration vs. Asphalt Level

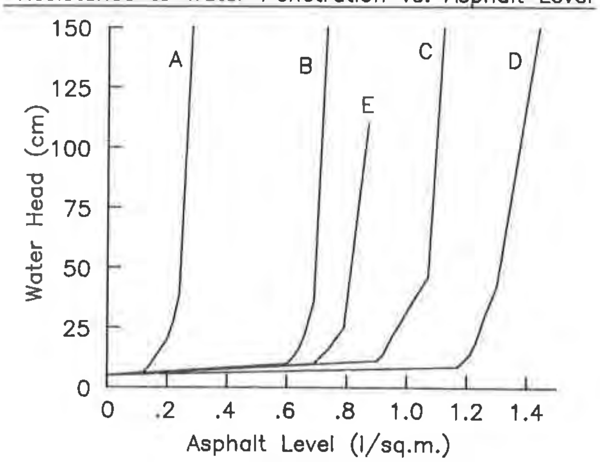

Figure 2

DYNAMIC SIMULATION TESTING

Reinforcement characteristics were determined by subjecting geotextile reinforced asphaltic beams to a dynamic load simulating interstate traffic. The equipment used to accomplish this consists of a pneumatically actuated repeated loading machine as shown in Figure 3. The apparatus was designed and assembled for Du Pont by the Georgia Institute of Technology and consists of commercially available parts. This machine can control the loading frequency from as low as 6 cycles/min. to a maximum of 130 cycles/min. at an equivalent surface pressure ranging from 275 kPa (40 psi) to 1800 kPa (260 psi).

The accuracy and reproducibility of this test program was highly dependent on the construction of uniform asphaltic beam test specimens. To insure uniformity Geotek Engineering Company, Nashville, Tennessee was contracted to manufacture the test specimens based on the State of Tennessee Surface Course Specifications 411E. The procedure used to construct the beams involved, (1) weighing of each ingredient, (2) heating and thoroughly mixing the ingredients, and (3) compaction. Analysis of the beams showed that density was uniform throughout the cross-section of each 5 cm x 7.5 cm x 35 cm (2" x 3" x 14") specimen. A summary of mix properties is included in Table II.

Testing of the fabric reinforced test specimens was conducted under controlled conditions. A standard 6.25 mm (1/4") square edged crack was constructed in the foundation with the test specimen centered above. A 6.25 mm (1/4") elastomeric pad provided flexibility to the system and was positioned below the foundation. The complete system was then installed in the dynamic testing machine and repeatedly loaded. All tests were run at a temperature of 22 ± 1°C (72 ± 2°F), a loading frequency of 72 ± 3 cycles per minute and loaded at an

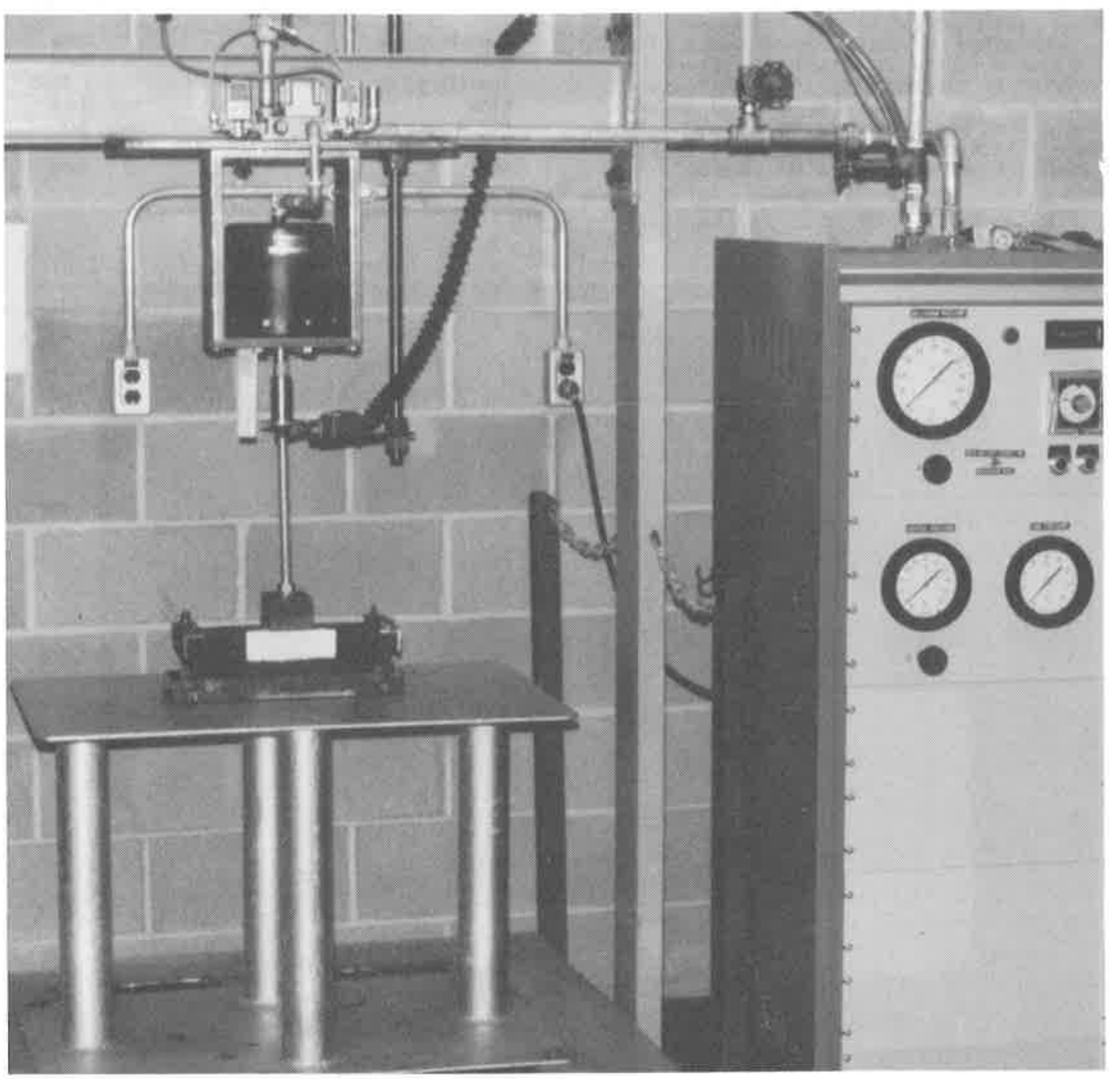

Figure 3 : Dynamic Simulation Testing Apparatus

equivalent surface pressure of 690 ± 10 kPa (100 psi). The development of a crack was monitored and failure was considered when the crack propagated to the surface of the overlay specimen. Table III lists the cycle life of all products tested and the life of non-reinforced specimens. Figure 4 shows the development of a reflected crack in the test specimen. In all cases the geotextile reinforced specimens had at least twice the life of the unreinforced samples, clearly demonstrating that geotextiles do provide a measure of tensile reinforcement.

ANALYSIS OF RESULTS

Moisture characteristics of paving fabrics were determined by simply measuring the degree of asphalt saturation required to provide a significant moisture barrier. The results in Figure 2 show that when each fabric is fully saturated an adequate moisture barrier is formed. The major discrepancy between products involves the amount of tack coat required to adequately saturate the geotextile. With asphalt cement tack coat costing approximately $400 per .5 l/sq.m. per lane km ($650 per .1 gal/sq.yd./mi.), the importance of minimizing consumption is easily recognized. The amount of liquid a fabric structure can absorb is directly related to the porosity of the structure itself. A thick bulky structure has a larger volume capacity than a thin dense structure of the same unit mass. Figure 5 is a plot of asphalt level versus fabric thickness. A good correlation is evident.

TABLE II. ASPHALTIC CONCRETE SURFACE COURSE SPECIFICATIONS

TABLE IIa. Combined Gradation for Asphalt Surface

Screen	Test Specimen Design Specifications	TN DOT Surface Course Specification 411E
12.7 mm	100	100
9.5 mm	94 ± 1	94 ± 6
4.75 mm	68 ± 1	68 ± 12
2.36 mm	50 ± 1	50 ± 10
0.600 mm	28 ± 1	28 ± 10
0.300 mm	17 ± 1	17 ± 9
0.150 mm	10 ± 1	10 ± 5
0.075 mm	6 ± 1	6 ± 4

TABLE IIb. Surface Course Mix Design

	Test Specimen Design	TN DOT Surface Course Spec. 411E
Asphalt Content, %	5.3	None
Maximum Density, Kg/m^3	2450	None
Stability, Kg	4390	≥ 2650
Flow	11.5	8-15
Air Voids, %	3.3	3-7
% Voids Filled	77	None
Swell, %	0.5	None

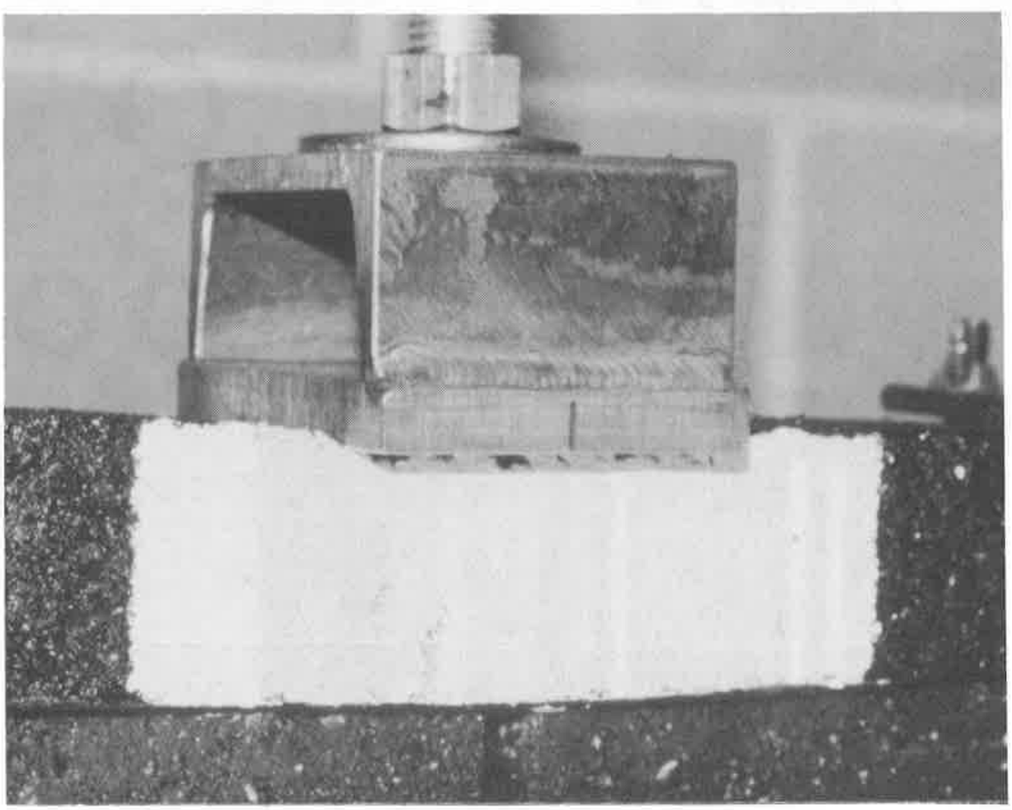

Figure 4 : Development of Reflected Crack in Specimen

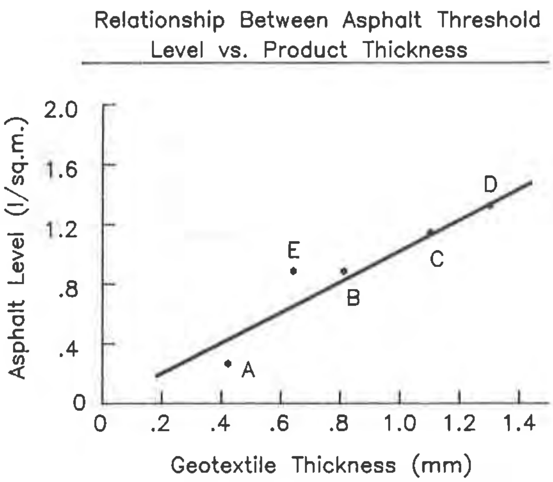

Figure 5

Examination of the dynamic simulation test results suggests that differences in fabric properties significantly affect the cycle life of the system. As shown in Table III, the cycle life of the products tested varied markedly. Depending on the fabric used, the improvement in cycle life ranged from a 2X increase over the non-reinforced control to a maximum 16X increase, thus representing an 8 fold variance in performance.

TABLE III. The Resistance to Reflection Cracking of Geotextile Reinforced Specimens

Geotextile	Dynamic Cycle Life	Standard Deviation
Control	480	50
Fabric A	7,650	575
Fabric B	1,000	55
Fabric C	2,300	880
Fabric D	3,260	610
Fabric E	2,760	570

Although many fabric properties (such as ultimate tensile strength, modulus and toughness) contribute to performance, the 5% secant modulus is the single property that best correlated with the dynamic simulation test results. This correlation is shown in Figure 6. The correlation is good except for two exceptions. First fabrics B and C have approximately the same modulus but there is a significant difference in performance. This discrepancy is the result of the difference in curvature of the respective stress-strain curves; i.e., fabric C has a higher tangential modulus at 5% than fabric B, therefore fabric C has a greater resistance to deformation at 5% elongation than fabric B. Secondly fabrics A and E both have a relatively high modulus but have a different level of performance. This difference is related to fabric slippage that prevents realization of the modulus benefits. Fabric A is a non-woven consisting of many relatively small filaments that develop a strong bond with the pavement whereas fabric E is a woven fabric consisting of a few relatively large filaments which do not bond as well.

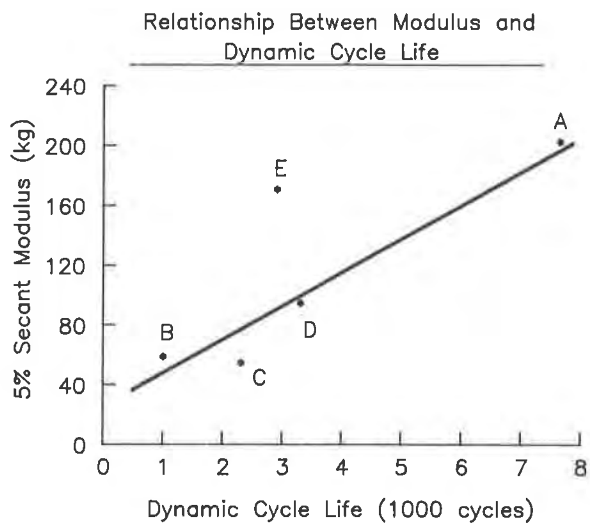

Figure 6

CONCLUSIONS

On the basis of the work described here it is concluded that:

(1) Geotextiles effectively improve the resistance of a pavement structure to reflection cracking. The principle benefits that fabrics provide to a pavement structure are:

- a moisture barrier which protects the underlying pavement structure from further degradation, and
- a degree of reinforcement to the bituminous concrete overlay which characteristically is weak when placed in tension.

(2) The fabric structure of the geotextile determines the threshold asphalt level required to form a moisture barrier.

- Dense thin structures require a significantly lower amount of asphalt than a thick porous structure.

(3) The reinforcement performance of the geotextile best correlates with the 5% secant tensile modulus. [And not the ultimate tensile strength of the product.]

- Fabric elongation is limited by the deflection of the pavement structure. Greater than 5-10% fabric elongation indicates excessive pavement deflection.

(4) A sufficient bond must be developed between the geotextile and overlay for the full reinforcement potential to be achieved.

- Fabric structures comprised of many small denier filaments form a much stronger bond than structures consisting of few large denier filaments.

(5) Engineers must consider these fabric properties in the design of overlay pavement structures.

- Overlay thickness is proportional to the amount of reinforcement required.
- Tack coat costs vary depending on the threshold asphalt level of the individual geotextile.

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The Control of Reflection Cracking with the use of a Geotextile. A Ten Year Case History

Controle des lezardes causées par reflection en utilisant une natte géotextile—Un historique de dix ans

During 1973 preventative maintenance was carried out on a major freeway section that had manifested severe cracking of the asphalt due to reflection from the underlying cement stabilized base course layer. A levelling course of 20 mm was laid after placing sand 300 mm wide and 5 mm thick as bondbreaking material at every crack, the average crack spacing being 2,5 m. A geotextile mat was then placed and overlaid with 50 mm asphalt. The pavement has been monitored since the completion of the work and after 8 years is still giving excellent service. However, the development of reflection cracks now appears to be taking place at an increasing rate. An inspection of the cracks revealed that they had probably been caused by one or more of several factors. From the evaluation it was clear that the geotextile asphalt system was capable of dealing with plain strain conditions. However, more complex conditions resulting from degradation of the structural system apparently causes vertical movement which cannot be accommodated by the system. Future maintenance will have to cater for this condition.

Pendant 1973, un entretien préventif a été fait sur une majeure partie de l'autoroute, là où il s'était manifesté des lézardes importantes dans l'asphalte, causées par la réflection de la couche sous-jacente de base, stabilisée avec du ciment. Une assise de nivellement de 15 mm a été posée apr`es avoir placé par dessus chaque lézarde, en tant que matériau non-agglutinant, 300 mm de sable de 5 mm d'épaisseur, la moyenne d'espacement entre les lézardes étant de 2,5 m. Une natte géo-textile a alors été placée puis recouverte de 50 mm d'asphalte. Ce pavage a té surveillé depuis installation, et après 7 ans est en état excellent. Cependant le dévelopement des lézardes due à la reflection semble être en accélération. Après inspection, il semblerait que les lézardes seraient causées par un ou plusieurs facteurs divers. A l'analyse, il était clairement évident que le systême géo-textile asphalteux était parfaitement capable de tenir le coup aux déformations ordinaires. Les conditions plus complexes, découlant de la dégradation structurelle, causait de toute èvidence un mouvement vertical que le systême ne savait pas accomoder. Les entretiens futurs devront prendre cette condition en considération.

PILOT STUDY

During 1971 it was decided to carry out preventative maintenance on a section of a major freeway near Johannesburg that had manifested severe cracking of the asphalt due to reflection from the underlying cement stabilised base course. The climate of this region is mild with average temperature ranging between 4°C and 26°C. Average annual summer rainfall is 720 mm. Attempts at sealing of these cracks by means of a rubberised emulsion had not been very successfuly, mainly due to the high stress on the joint material and the rapid deterioration of such material as a result of exposure to the environment. Preventative maintenance had to be done since it was feared that possible pumping action could lead to rapid deterioration and early failure of the pavement.

The pavement structure consisted of the following:

Cont. graded asphaltic concrete surfacing (AC)	25 mm
Cement stabilised Crushed Stone Base Course	150 mm
Cement Stabilised natural gravel sub-base	150 mm
Selected Subgrade	150 mm
Insitu Subgrade-Residual decomposed granite	

Small experimental sections were initially constructed in November 1972, using Structofors, a bitumen impregnated polyester woven fibre with a grid pattern of approximately 15 mm square. From these initial investigations it became apparent that a minimum surfacing of 50 mm was required on top of the geotextile, since thinner layers suffered from stresses set up in the geotextile fibre causing the overlay to crack and crumble. Nevertheless, the work showed great promise and it was decided to carry out a pilot study before executing any major construction work. Accordingly, the following steps were taken:

1 In January 1973 a test section of approximately 50 m in length was built on a portion of the freeway requiring maintenance.

2 The use of the geotextile as reinforcing material was investigated by constructing a control section alongside without any reinforcing. In addition, steps were taken to investigate the effect of a bondbreaker at the cracks since it had been determined that this was a necessary feature of the system.

3 Subsequent to the carrying out of the field experiment a model test was also conducted in the laboratory in order to establish limits for the application of the method. These laboratory tests were supplemented by using the Heavy Vehicle Simulator of the National Institute for Roads and Transport Research in Pretoria. The pilot study was found to be very successful and full reports have been published elsewhere. (1) (2) (3). For the sake of completeness a brief review of the results will be discussed.

Field Experiment (1)

4 In the case of the field experiment it was found that the geotextile was successful in preventing reflection cracking related to the effect of environmental forces. After 15 months in service the sections with Structofors had between 30 and 45% of the original cracks reflected through, whereas the unreinforced asphalt mixture had 60% reflection cracks.

The lime bondbreaker had not functioned well primarily because it was too thin. The effect of traffic had also been investigated by means of the HVS and the equivalent of more than 4 x 10^6 - 80 kN axle loads had been applied. It was found that some of the reflected cracks tended to close up due to the kneading action. It must, however, be conceded that these axle loads had been applied over a relatively short period of time with the result that long term the interaction between load application and environmental forces was not present. This is a fact that will be considered again when reviewing the present situation.

Laboratory Experiment (2)

5 The laboratory study was carried out by constructing a gap graded asphalt mixture (AC) on two adjacent concrete slabs lying on rollers supported by a steel frame, thus forming an artificial crack between the two. One slab was fixed to the steel frame and horizontal movement at the crack was affected by a weight applied to a cable over a pulley on the other side. An attempt was also made to simulate relative vertical movement at the joint using a cam under one of the slabs. The asphalt was placed in two layers, 25 mm as levelling course and a 75 mm overlay.

The following overlays were evaluated in this experiment:

(a) Reinforced AC overlay without bondbreaking.

(b) Reinforced AC overlay with a 500 mm wide bondbreaker 5 mm thick straddling the crack.

(c) AC overlay only.

(d) Unreinforced AC overlay with sand bondbreakers as before.

Results of Laboratory Experiment

6 Figure 1 indicates some results of this experiment. At a crack width of 2 mm it can be seen that for the AC overlay alone the strain at the joint was 3,5 times that of the overlay consisting of reinforced AC with bondbreaking. Also the strain at the crack in the reinforced overlay without bondbreaking was 2,8 times lower than that in the unreinforced overlay. Bondbreaking alone reduced tensile strains by approximately 25%. Movement at the joint was distributed over a much larger area with the reinforcing and bondbreaker.

During this experiment it was also noted that with reinforced overlays the crack could be opened considerably more before cracks appeared on the pavement surface as indicated in the following table.

Table 1 Strain Behaviour of Overlay Systems

Overlay Type	Maximum crack width before reflection (mm)
AC	3
AC with bondbreaking	4
Reinforced AC without bondbreaking	9
Reinforced AC with bondbreaking	18

7 From the results (2) it was found that the strain immediately prior to rupturing of the overlay was in excess of 1,5%. The rate of deformation had not been equal for the four tests but varied between 2,0 mm/hr and 5×10^{-3} mm/hr.

8 Despite this wide range the maximum strain at the surface of the AC overlay did not vary much.

9 When the crack was opened the stress concentration tended to rupture the overlying asphalt. However, the crack only proceeded a short distance before it was checked by the geotextile-sand system and reflected parallel to the surface. As the strain was increased micro cracks formed. Ultimately reflection occurred some distance from the crack. Fig. 2 shows one such test in progress.

10 A repetitive relative vertical movement of 1 mm applied to the system at 1 Hz indicated that the unreinforced overlay could take 27 000 applications whereas the reinforced system with bondbreaking was capable of withstanding 3 x 10^6 repetitions. Of course these results are only relative since the method of application of the vertical movement did not simulate a wheel load moving over a crack, but since the test was strain related, it gave confidence in the reinforced system. The experiment thus indicated that load and temperature strains in the overlay could be catered for using reinforced AC plus a bondbreaker over the crack.

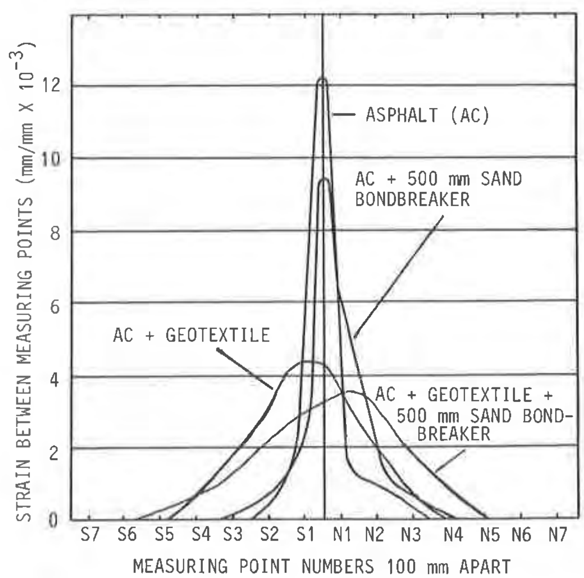

FIG. 1: STRAIN PATTERN FOR DIFFERENT TYPES OF OVERLAYS AT CRACK OPENING OF 2 mm

FULL SCALE CONSTRUCTION

In the light of the success of the preliminary study, it was decided to award a contract for the construction of 80 000 m² of 50 mm geotextile reinforced asphalt with 300 mm of bondbreaking material at every crack, the average crack spacing being 2,5 m. The pavement has been monitored since the completion of the work and after 8 years it is still giving excellent service. However, the reflection cracking has steadily been increasing and it now appears to be taking place at an increasing rate. The subsequent discussion relates to the details of the construction and the subsequent behaviour of the pavement structure.

Fig. 2 Test with Structofors and Sand Bondbreaker in Progress

The method of construction used in practice was as follows:

(a) Cracks were cleaned properly using compressed air prior to the application of 0,35 l/m² of 45% cationic bitumen emulsion.

(b) Sand was spread over the cracks using a specially designed manually operated sand distributor. The thickness of the sand was 5 mm and the width 300 mm. Sand used was a fine mine sand, crushed in the gold mining process, the grading of which can be seen in Table 2. This sand spreading operation was done immediately in front of the paver in order to ensure as little disturbance of the sand by trucks moving over it, as possible.

Table 2 Grading of Sand

Sieve size (mm)	% Passing
1,18	100
0,600	98
0,300	68
0,150	30
0,075	15

(c) After the sand had been placed, a levelling course of 20 mm thick AC was paved followed by another tack coat. Marshall and other properties of the AC mix used for the levelling course are given in Table 3.

(d) The geotextile reinforcement used was manufactured from a polyester fibre impregnated with bitumen. It was a woven 15 mm mesh and supplied in rolls 1,6 m wide and 150 m long. Overlaps of 150 mm at longitudinal and 300 mm at transverse joints were used. The geotextile was unrolled on the tack coat and nailed down to the surface at approximately 20 m intervals.

(e) A 50 mm gap graded asphaltic concrete layer with precoated chips rolled into the surface was paved over it. The properties of the AC wearing course are given in Table 3.

Table 3: Properties of Asphaltic Concrete Mixes

	Levelling coarse	Final coarse
Bitumen content	7,0	6,5
Marshall flow	2,2	2,3
Marshall stability	5,62	6,03
Voids in mix %	6,4	7,6
Binder type	60/70 Pen	60/70 Pen
Grading:		
Sieve size (mm)	% passing	% passing
26,5	100	100
19,0	100	88,9
9,5	98,9	64,9
6,35	74,0	64,1
4,76	68,1	62,8
2,36	63,2	58,4
1,00	60,8	54,3
0,500	57,4	50,8
0,295	38,6	29,9
0,150	19,8	15,4
0,075	8,5	7,4

Construction Problems

The following problems were encountered during construction:

(a) It was found that the paver tended to push the sand somewhat in the direction of paving. For this reason the sand width was reduced from the originally intended 450 mm to 300 mm and it was spread slightly eccentric. Cores drilled out afterwards indicated bondbreaking to a width of 450 mm symmetrical over the crack. At least 5 mm of sand was necessary to ensure proper initial bondbreaking.

(b) Trucks moving over the sand did not pose any greater problems as far as disturbing the sand is concerned. However, windy conditions did pose a problem and could bring construction to a halt.

(c) Provided the geotextile was nailed down properly no problems were encountered with trucks moving over it.

(d) In constructing the final layer on top of the geotextile it was very important that no horizontal movement took place between the geotextile and the AC overlay during compaction, since this caused the asphalt to disintegrate.

Such movement would cause shearing off of the overlay just above the geotextile resulting in potential slip between the two layers and also a loss in the effectiveness of the reinforcing layer. Care also had to be taken with the design of the AC to ensure that the stone content was high enough to prevent slippage on the underlying levelling course.

PERFORMANCE OF THE OVERLAY

The overlay was completed in 1974 and has since carried a traffic load of around 2 million equivalent 80 kN axle loads. Very little deterioration in riding quality has been experienced and reflection cracking has also been slow in developing, reaching about 30% by 1979. However, since 1979 cracks have been developing at an increasing rate reaching 60% in 1982. This trend is indicated in Fig. 3 showing the length of cracks which have apparently been reflected as a percentage of the original cracks. Fig. 4 shows a general view of the site during construction of the test section. Fig. 5 shows to what extent the overlay has cracked to date: The cracks have been demarcated with liquid lime.

Evaluation of Cracks

The nature and effect of cracks was considered by carrying out a survey and coring through cracks. The following was found:

1. In general the crack pattern was strongly related to the underlying cracked CTB. (Compare figures 4 and 5).
2. Two types could be distinguised, viz.

    Type 1: conventional true vertical reflection cracks extending from the top down into the CTB, and

    Type 2: randomly spaced, short discontinuous cracks orientated similarly to *Type 1.*
3. 6% of cracks were *Type 1* and 94% *Type 2.*
4. Some cracks were found to extend only through the overlay and the underlying CTB being absent inbetween. Apparently these cracks had been deflected horizontally. Clear proof of such deflection was found.
5. The longitudinal cracks had largely reflected through whereas the transverse cracks had not. Less cracks had also reflected through the areas with low traffic volumes eg. zone between off-ramp and slow lane.
6. Whereas the cracks were formerly concentrated in a section of the road they were now spread over 90% of the road length.
7. Little or no evidence of pumping was found.
8. Some evidence of debonding of the overlay was found.

The geotextile was very effective in preventing reflection cracks of *Type 1* and the behaviour of the geotextile system was very similar to that predicted by the laboratory experiment. Furthermore, the fact that the longitudinal cracks have largely reflected through is probably due to the fact that the pavement structure is not contained laterally. The smaller geotextile overlap that was used for the longitudinal joints could also have been a contributing factor. *Type 2* joints could be related to reflection but could also have been caused by fatigue of the asphalt either in plain strain or as a result of the structural behaviour of the system under traffic loads. The limited evidence of debonding is gratifying and lends support for the use of the sand as part of the system.

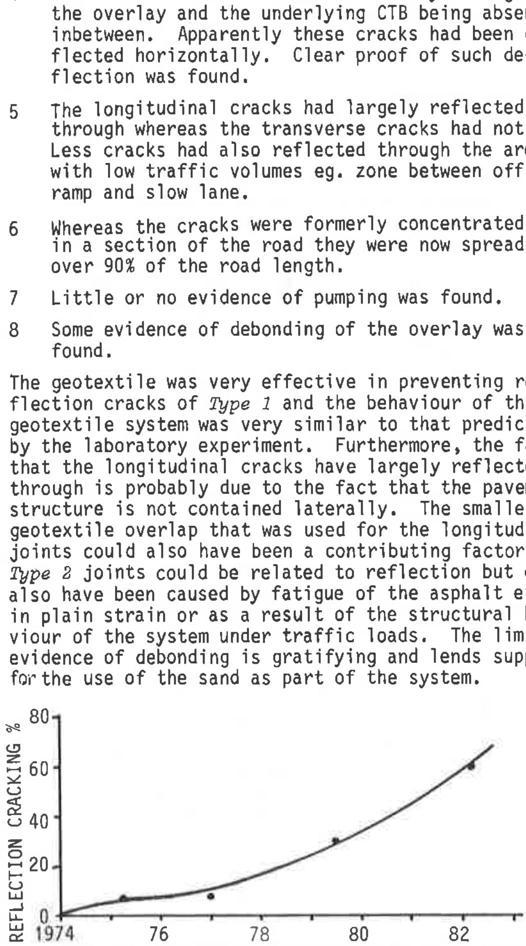

FIG. 3: REFLECTION RELATED TO AGE IN CALENDER YEARS

Fig. 4 General View of Site during Construction of Test Section

Fig. 5 Cracks demarcated with Lime during a Survey February 1982

Evaluation of Deflections

Deflections were measured with a modified Benkelman beam which had the capability of also measuring relative vertical movements under a moving 80 kN axle load at a crack. Deflections measured prior to overlaying were in the order of 0,17 mm with very little relative movement at the reflected cracks. The deflections have increased by about 30% since 1973 and the relative vertical movement under a moving 80 kN axle load at the *Type 1* cracks is now of the order of 0,22 mm. These readings are summarised in Table 4.

Table 4 Deflections under 80 kN Axle Load

	Mean (mm)	Standard Deviation
Prior to overlaying (1973)	0,17 mm	0,10
After overlaying and 8 years service (1981):		
Between cracks	0,21	0,09
At *Type 1* cracks	0,27	0,16
Relative vertical movement *Type 1*	0,22	0,14

The information obtained from deflections, as shown in Table 4, indicates that not only has the deflection increased, despite the overlay of 50 mm but scatter has also increased, especially at the cracks.

Considering the stiffness of the cement stabilised base layer, the pavement can almost be classified as a rigid pavement. Applying measured deflections to a theoretical model, pavement characteristics were calculated by back analysis. The results are shown in Table 5.

Table 5 Theoretical Materials Properties as Calculated from Deflection Measurements and Original Slab Stiffness

Stage	Slab Stiffness (GPa)	Subgrade Stiffness (MPa/m)
Before Overlay (1973)	22*	880
After Overlay and 8 years of use	12	750

*Material stiffness as determined by laboratory testing just after construction.

These results indicate a decrease in stiffness of both the stabilised base layer as well as the subgrade. This seems to point to degradation of the base and the subgrade over the past 8 years and thus a general decrease in structural capacity. The poor condition of some CTB cores as well as the reduction in tensile strength of about 30% as found by limited indirect tests gave support to the conclusion.

Deflections have so far been related to the structural performance of the stabilised base. Another important aspect that also needs consideration, is the performance of the gap graded asphaltic concrete overlay. Cores were analysed and it was found that the physical properties of the asphalt had changed due to traffic and environmental influences. In particular the permeability and density were affected. This in turn affected the stiffness. The capability of the mix to sustain deformation has thus deteriorated somewhat which could have caused the greater incidence of cracking. Results are shown in Figures 6 and 7 and Table 6.

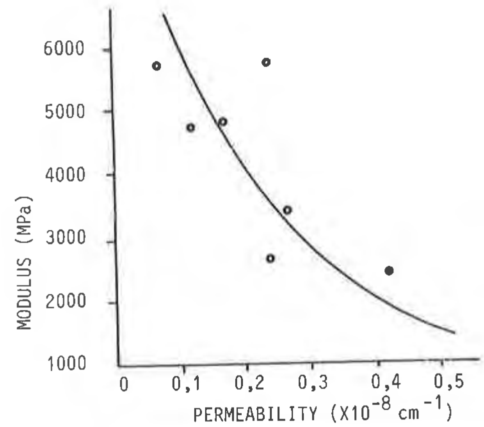

FIG. 6 : STIFFNESS MODULUS VS. PERMEABILITY

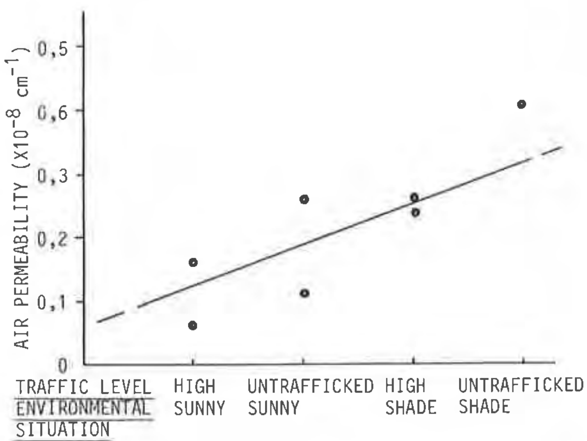

FIG. 7 : INFLUENCE OF THE ENVIRONMENT AND TRAFFIC ON PERMEABILITY

Table 6 Properties of the Asphaltic Overlay

	When constructed (1973)	1981
Stability (kN)	6,0	-
Flow (mm)	2,3	-
Lab. voids (%)	7,6	-
Binder (%)	6,5	-
Air permeability (x10-8/cm)	1,12	0,17
Mix stiffness (MPa)	3600	5200
Road voids (%)	8,2	4,0

Analytical Evaluation

In the light of the foregoing a finite element analysis was carried out to determine what effect these developments would have. Indications are that the max. tensile stresses in the asphalt have been increased by 50% whilst the tensile stress in the CTB has been reduced by 30%. In the light of the degradation of the CTB this is not expected to alter the stress ratio. However, an overall reduction in the fatigue life is to be expected. In fact, the increase in stress level in the asphalt together with the increase in stiffness is likely to cause a decrease in fatigue life of almost one order of magnitude. The deflection results lend support to this argument particularly the increase in relative vertical movement between adjacent slabs. If it is accepted that the structure is basically a rigid pavement with an overlay, an analysis using nomographs developed by Mc Cullough et al (4) predicts a 50% failure of the pavement.

CONCLUSIONS

1 The geotextile reinforced asphalt overlay system has successfully served its prime function to date viz. preventing ingress of water into the pavement structure and consequential damage. It therefore serves as an important tool in the pavement management system. It is, however, clear that the system is not capable of increasing the structural capacity of the pavement since it can only absorb a limited amount of differential vertical movement.

2 It appears as if the structural integrity of the pavement has now reached a critical situation. However, if the life of the overlay can be extended and the asphalt softened by the application of a rejuvenator it may be possible to control the degradation of the CTB to the point where it will function as a conventional crushed stone base.

3 A stabilised pavement system can be treated as a rigid pavement for design purposes but it is important to consider vertical movement of the cracks during the design process. In order to minimise relative vertical movement, the stabilised base should not be too stiff so as to ensure structural cracking in the form of random rather than well defined transverse cracks and the subbase should be of good quality to avoid weathering of the subbase and thus movement of fines and subsequently loss of support of the CTB.

ACKNOWLEDGEMENTS

This project has extended over more than ten years and the authors wish to express their appreciation to all persons who assisted during this period. The project was sponsored by the Department of Transport Affairs and permission to publish the results of the investigation was granted by the Director-General. This is gratefully acknowledged.

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Fabric Interlayer for Pavement Overlays

Textile interposé entre revêtement ancien et nouveau

Laboratory experiments were conducted to establish the mechanisms responsible for the performance of commercially available engineering fabrics as effective reflection crack arrestors and determine fabric properties which provide the desired field performance. Tests were developed and performed to determine asphalt requirements and shrinkage characteristics of fabrics. Tests on paving mixtures containing fabrics were conducted to determine resistance to thermal reflection cracking, shear strength of the fabric interlayer, flexural fatigue characteristics, tensile properties and probability of pavement cracking caused by fabric shrinkage. Results showed that fabrics improved tensile properties, increased fatigue life, reduced crack propagation rate and will not compound overlay slippage problems when properly installed.

Des essais de laboratoire ont été effectués pour mettre en évidence les mécanismes permettant à des géotextiles disponibles sur le marché d'arrêter la propagation des fissures et pour déterminer les propriétés du textile qui régissent la performance en place. Des essais ont été mis au point et effectués pour déterminer les spécifications du bitume et les caractéristiques de retrait des textiles. Des essais sur des revêtements contenant des textiles ont été réalisés pour déterminer la résistance à la propagation des fissures thermiques, la résistance au cisaillement au niveau du géotextile, les caractéristiques de fatigue en flexion, le comportement en traction et la probabilité de fissuration du revêtement causée par le retrait du géotextile. Les résultats montrent que les textiles améliorent le comportement en traction, augmentent la résistance à la fatigue, réduisent la vitesse de propagation des fissures et n'aggravent pas le problème du glissement de la couche rajoutée lorsqu'ils sont correctement installés.

INTRODUCTION

The major portion of highway pavement expenditures in the United States during the next 20 years will be for reconstruction, rehabilitation and maintenance of our existing facilities rather than construction of new facilities. This ecomomic prognosis has increased research and development efforts aimed at pavement overlay systems that will eliminate, reduce or delay cracks from reflecting from the old pavement through the new overlay. The use of fabrics in combination with asphalt concrete overlays is one of several promising systems for reducing or delaying reflection cracks.

This is a summary report of a laboratory study (1) sponsored by the Celanese Fibers Marketing Company.

Laboratory tests were developed to help identify parameters which contribute to early cracking of fabric overlays. Fabric shrinkage tests were conducted by both Celanese and Texas A&M University and a "construction crack test" was developed. Results from these tests illustrate the importance of using "low" shrinkage force fabrics and the importance of proper construction techniques to reduce fabric wrinkles.

The investigation of potential slippage problems included the development of a special "airport shear" test to determine the shear strength of the interface between the fabric and the old asphalt concrete and the new asphalt concrete overlay.

Specific objectives of the laboratory study are listed below:

1. Establish the mechanisms responsible for the performance of fabrics as effective reflection crack arrestors,
2. Determine fabric properties which provide the desired field performance under a variety of conditions and
3. Define and delineate satisfactory field installed procedures for utilizing fabrics as part of an overlay system to reduce or prevent reflection cracking.

The laboratory testing program includes testing of fabrics to determine the following properties:

1. Asphalt content at saturation,
2. Temperature shrinkage characteristics,
3. Shear strength of old pavement-fabric-new overlay interface,
4. Tensile strength of fabric-mixture system,
5. Flexural fatigue properties of fabric-mixture system and
6. Resistance to reflection cracking (overlay tester).

MATERIALS

Eight fabrics, labeled A,B,C,D,E,F,G, and H, supplied by Celanese Fibers Marketing Company, were tested to determine the properties listed above.

Asphalt concrete paving mixtures containing AC-10 asphalt cement were utilized to evaluate the fabrics under simulated field conditions.

DESCRIPTION OF TESTS

Laboratory apparatus were designed and developed to mechanistically evaluate fabrics in a logical sequence of tests. These new test methods were developed to simulate field loading conditions and hence are capable of evaluating overlay systems on a relative basis. First, a fabric was evaluated to see if it could withstand temperatures encountered in hot mix pavement construction; if so, the appropriate quantity of asphalt tack was determined. Asphalt concrete specimens containing fabric were fabricated and tested to define the effects of fabrics in overlay slippage, and to separately evaluate the performance of fabrics in the reduction of fatigue and thermal cracking.

SATURATION TEST

This test method involves the saturation of a piece of fabric 200 x 200 mm (8 x 8 - in) in AC-10 asphalt cement at 121°C (250°F) for 1 minute. The saturated fabric is allowed to cool and then pressed with a hot iron between two absorbent papers to remove the excess asphalt. This method produces a uniformly appearing saturated fabric.

Fabric saturation contents are shown in Table 1.

Table 1. Fabric Saturation Quantities and Recommended Tack Coat Quantities.

Fabric	Saturation Content, $m^3/m^2 \times 10^{-4}$ (gal/yd^2)	Asphalt Tack Coat Quantity, $m^3/m^2 \times 10^{-4}$ (gal/yd^2)
A	1.8 (0.04)	6.3 (0.14)
B*	0.9 (0.02)	5.4 (0.12)
C	1.8 (0.04)	6.3 (0.14)
D	5.9 (0.13)	10.4 (0.23)
E	14.9 (0.33)	18.1 (0.40)
F	1.4 (0.03)	5.9 (0.13)
G	4.5 (0.10)	9.1 (0.20)
H	6.8 (0.15)	11.3 (0.25)

These quantities were utilized as the design asphalt content for the test specimens prepared in this laboratory testing program.

Fabric asphalt saturation content is one parameter that is utilized to determine field tack coat quantities for adequate adhesion between pavement layers. The tack coat quantity may be estimated from the equation below:

$$Q_d = 0.08 + Q_s \pm Q_c \qquad \text{(Equation 1)}$$

where:

Q_d = design tack coat quantity, gallons per square yard

Q_s = fabric asphalt saturation content, gallons per square yard,

Q_c = correction to tack coat quantity based on asphalt demand of old surface, gallons per square yard.

The quantity, 0.08 gallons per square yard is based on field experience for overlays with no fabric. This equation, developed earlier in this research program (2), was utilized to determine asphalt tack coat quantities for laboratory testing purposes. A value of +0.02 was selected for Q_c based on the surface conditions of the laboratory samples.

LINEAR SHRINKAGE TEST

This test involves soaking of the fabric in asphalt cement to simulate the application of a hot asphalt concrete overlay. Four pieces of fabric with dimensions of 100 x 100 mm (4 x 4-inches) were submerged in 121°C (250°F) and 149°C (300°F) asphalt cement. One piece of fabric was removed after elapsed times of 1, 5, 15 and 30 minutes and allowed to cool then measured along the run of the fabric to determine the effects of heat on length change as a function of time and temperature.

The results at 149°C (300°F) are plotted in Figure 1.

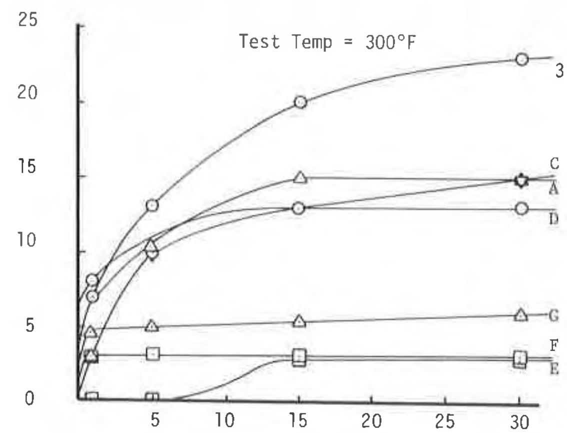

Figure 1. Temperature Stability of Fabrics in 300°F Asphalt Cement.

As expected the higher temperature caused more fabric shrinkage. The lower test temperature (149°C) appears to be located near a critical temperature; that is, below this temperature very little shrinkage occurs, but above this temperature significant shrinkage occurs in most of the fabrics. Fabrics E,F and G, with linear shrinkage values of 5 percent or less after 15 minutes, have the lowest temperature susceptibility. Fabric B, the most temperature susceptible fabric tested, as well as C and G apparently would have continued to shrink after 30 minutes exposure to the hot asphalt cement. Most of the other fabrics reached a maximum shrinkage after 30 minutes exposure.

CONSTRUCTION CRACKING TEST

It is fairly common knowledge that heat (149°C or more) will cause varying degrees of shrinkage in most currently available fabrics. This shrinkage may be advantageous at least temporarily, as a "post-tensioned" fabric would improve the tensile properties of the system, particularly at low strains. The temporary nature of these benefits are due to stress relaxation in the viscoelastic system.

When wrinkles (or cuts without adequate overlap) are present in a fabric during an overlay operation, tensile forces caused by fabric shrinkage can produce a significant displacement of the fabric normal to the wrinkle or cut. Shrinkage occurs while the asphalt concrete overlay is hot and without appreciable tensile strength; thus, the fabric displacement carries the hot overlay with it resulting in a crack in the new overlay along the wrinkle or cut.

A laboratory test was developed to identify possible causes of cracking within hot asphalt concrete during the early life (first few hours) of the overlay. The test consists of the placement of a hot asphalt concrete mixture over a fabric which has been placed in a rectangular mold.

Two rectangular molds 1,220 mm (48-inches) long by 140 mm (5 1/2-inches) wide, were fabricated from wood (Figure 2). One mold was fabricated with 32 mm (1/8-inch) transverse crack near the center; the other mold contained no crack. An appropriate quantity of tack was placed in the bottom of the mold, depending on the

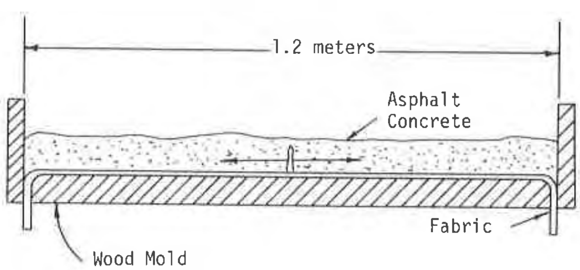

Figure 2. Apparatus Used to Determine Likelihood of Construction Cracking.

requirements of the fabric. Fabric was placed over the tack coat in one of three different orientations:

(1) control samples - either no fabric or smooth fabric with no wrinkles, (2) one 10mm (3/8-inch) wrinkle in the fabric near the center of the mold (wrinkle down in crack when using mold with crack), and (3) fabric cut transversely near the center of the mold. The fabric was 140 mm (5 1/2-inches) wide and securely fastened to each end of the mold (Figure 4). Hot mix asphalt concrete was placed in the mold over the fabric and compacted using a hand tamper. The compacted asphalt concrete, which ranged in thickness from 19mm (3/4-inch) to 38 mm (1 1/2-inch) was observed periodically to check for cracking. If cracks did appear in the overlay it was usually within 15 minutes after placement and compaction. In some tests smaller cracks appeared after more than one hour of time elapsed.

Test results using the four foot mold are given in Table 2. No shrinkage cracks appeared in any of the tests conducted using a two foot mold which was used initially. However, cracks did appear in certain similar tests using the four foot mold. This indicates the shrinkage forces that accumulated over a one foot length of fabric on each side of the wrinkle were not sufficient to cause cracking in the hot overlay. Furthermore, the shrinkage forces in some fabrics are capable of acting from a distance greater than one foot to produce cracking in a hot asphalt overlay.

Fabrics E and F, which exhibited very little shrinkage in the temperature stability test, did not produce cracks in this test. And conversely, Fabric B,

Table 2. Construction Cracking Test Results.

Fabric	Overlay Thickness, Inches	Type Test	Results
B	3/4 1 1/2 3/4	C+W C+W Cut	Crack Crack Large Crack
E	3/4 1 1/2 1 1/2 3/4	W W C+W Cut	No No No No
F	3/4 3/4 3/4 3/4	C+W C+W W Cut	No No No No
G	1 1/2 3/4 1 1/2 3/4	W C+W C+W Cut	No Small Cracks No Small Cracks
Control	3/4 3/4 3/4	C (No Fab) C (Smooth) No Crack No Fabric	No No No

C+W = Crack in mold + wrinkle in fabric
W = Wrinkle in fabric, no crack in mold
C = Crack in mold, no wrinkle in crack

which exhibited excessive shrinkage in the temperature stability test, produced the largest cracks and was the only fabric tested that produced a crack in the 38 mm (1 1/2-inch) overlay. In the temperature stability test, Fabric G exhibited comparatively little shrinkage but shrank over a relatively long time period, similarly, in this test, it produced small cracks in the thin overlays and more than an hour elapsed before they appeared.

Although the number of tests were not sufficient to make positive statements, the following conjectures are made based on test observations: (1) There was no noticeable difference in crack propagation between tests using emulsion and asphalt cement tack coats, (2) The cut fabric allowed slightly more cracking than the wrinkle alone or the wrinkle in the crack, (3) The thicker overlay was less likely to crack due to fabric shrinkage, (4) Fabrics with free shrinkage (1) in excess of about 7 percent may cause cracking during construction, (5) Fabrics with high shrinkage forces (1) are more likely to cause cracking, (6) Fabric-asphalt cement systems with linear shrinkages greater than 5 percent after soaking in 149°C (300°F) asphalt for 30 minutes are likely to cause cracking during construction.

INTERFACE SHEAR STRENGTH

Adequate shear strength must be attained or pavement slippage failures will occur. Slippage failures, typically crescent shaped, are associated with high shear stress areas of a pavement and are most likely to occur during braking or turning operations when ambient temperatures are high.

A test method was developed and used to determine the shear strength of the fabric interface. Tests were conducted at 20, 40, and 60°C (68, 104 and 140°F) at a deformation rate of approximately 220 mm/sec (13 in/sec). A statistic vertical load of 178 N (400 lbs.) was applied to the 75 x 75 x 50 mm (3 x 3 x 2 inch) samples. Specimens at 60°C (140°F) would not support the vertical load and were thus tested with no appreciable vertical load.

Individual and mean values of interface shear strength are presented in Table 3. Optimum tack coat was established by use of Equation 1. Low tack coat is one-half the optimum value while high tack coat is twice the optimum value.

Table 3. Shear Strength Test Results.

Specimen Identification	Test Temp°c(°F)	Shear Strength in psi @		
		Low Tack	Opt. Tack	High Tack
A	20 (68)	230	240	-
	40 (103)	160	190	240
	60 (140)	-	120	-
D	20 (68)	200	280	-
	40 (103)	190	190	290
	60 (140)	-	120	-
E	20 (68)	310	390	400
	40 (103)	240	265	-
F	20 (68)	185	340	390
	40 (103)	170	170	-
G	20 (68)	150	260	-
	40 (103)	130	180	170
Control-1 No Fabric 0.05 Tack	20 (68)	-	350	-
	40 (103)	-	180	-
	60 (140)	-	130	-
Control-2 Asphalt Concrete with no interface	20 (68)	-	440	-
	40 (103)	-	290	-
	60 (140)	-	150	-

Control-1 samples were fabricated to simulate typical old pavement-new overlay interfaces using 0.00023 m^3/m^2 (0.05 gal/yd^2) of tack. Control-2 specimens were fabricated with no construction interface in the plane of shear. As expected, the shear strength of the Control-2 sample is greater than that of the Control-1 samples. At the optimum tack coat and low temperatures the shear strength of those samples without a fabric at the interface (Control-1) is usually in excess of those samples with fabric at the interface. At the higher temperatures the shear strength of samples with fabric at the interface approaches the shear strength of those samples without fabric at the interface. The tack coat quantity called "high" was, without doubt, more asphalt cement than should be used in an actual overlay operation. Shear strength, however, increased with increased tack coat for most of the fabrics. Hence, the optimum tack coat based on maximizing shear strength is different from that indicated by Equation 1. This increase in shear strength with increased tack coat is probably due to excess asphalt which migrated into the mixture adjacent to the shear plane thus creating a more tenacious bond in the critical area. However, this may not always occur with certain fabrics, especially at higher temperatures.

Shear strength with fabric E is, in general, notably higher than that with the other fabrics. It is noteworthy that Fabric E is thicker and fuzzier than the other fabrics. Therefore, it appears that shear strength is directly related to the bulk of a fabric or, more likely, the asphalt saturation level and surface friction of the fabric. This seems reasonable in that the "fuzz" could act as numerous little roots to provide reinforcement at the fabric interface and this provide increased shear strength.

From a pavement performance standpoint, it is important to have sufficient shear strength at the interface between the old pavement and the new overlay to prevent slippage failures. The magnitude of the required shear strength is dependent upon the type of traffic, speed of traffic, severity of braking and wheel turning movements, ambient temperature and location of the interface within the pavement structure. At the present time an acceptable level of interface shear strength cannot be firmly established.

As a general guide, it is desirable to have interfacial shear strength of the same order of magnitude as that associated with conventionally constructed overlays (Control-1). By adjusting tack coat quantity and/or asphalt grade, all fabrics can meet this interim criteria.

FLEXURAL FATIGUE

Beam fatigue tests were performed to provide information for prediction of the fatigue life of pavements. Fatigue cracking of pavements is caused by repeated wheel loads and will appear as cracks in the wheel path. These cracks will have a pattern similar to "chicken wire" or "alligator skins".

Flexural fatigue characteristics of asphalt concrete mixtures with and without fabric were determined at 20°C (68°F). Loads are applied at the third points of the beam, four inches on center, using one inch wide steel blocks. The machine is operated in the load control mode with a half-sine wave form at a frequency of 100 cycles per minute (1.67 Hz) and a load duration of 0.1 seconds. The test specimens are oriented such that the fabric is subjected to tensile stress during the loading phase. A reverse load is applied at the end of each load cycle to insure that the specimen returns to its original at-rest position after each cycle.

Peak stress, initial bending strain (bending strain at the 200th cycle), initial stiffness modulus (200th cycle) and estimated total input energy were calculated for each specimen. Table 4 summarizes results of those tests conducted at a peak stress level of 100 psi. Test results indicate that, generally, certain fabrics with appropriate tack coats placed within a flexural fatigue specimen in the region of tensile stress will improve fatigue life.

Table 4. Simple Statistics of Flexural Fatigue Data.**

Sample No.	Bending Strain, in/in	Cycles to Failure	Initial Stiffness Modulus, psi	Total Energy Input, lb-in
Control	.00074	6,400	151,000	5,500
Fabric A (Low)	0.00130	3,100	80,000	4,800
Fabric A (Optimum)	0.00064	9,200	177,000	7,200
Fabric A (High)	0.00130	4,900	78,000	7,800
Fabric D (Optimum)	0.00056	9,300	196,000	7,200
Fabric E (Optimum)	0.00087	79,000	147,000	76,000
Fabric F (Optimum)	0.00123	4,600	88,000	6,500
Fabric G (Low)	0.00077	7,400	133,000	7,600
Fabric G (Optimum)	0.00081	8,600	134,000	9,600
Fabric G (High)	0.00064	16,200	133,000	7,600

* Log Mean ** Only those specimens tested at a stress near 100 psi are included in the mean.

Fifteen fatigue tests were conducted on the control beams and beams containing Fabric G to define the relationships between bending stress or initial bending strain and number of load applications to failure. The plotted results are given in Figure 3. Dashed lines on this figure represent the locus of the regression equations for the Control specimens. Hence, at a given bending stress the control beam would fail in fewer load applications than the beam containing Fabric G. (The same is true for initial bending strain.)

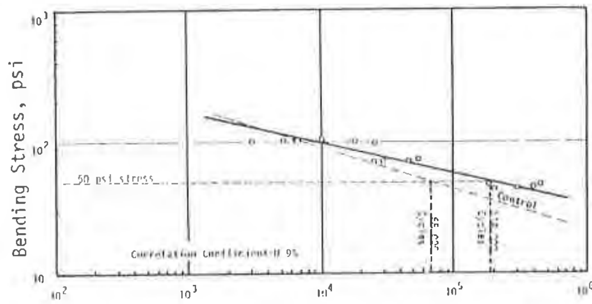

Figure 3. Stress versus Load Applications to Failure for Specimens Containing Fabric G at Optimum Asphalt Content.
1 psi = 6.894kPa

There appeared to be some relationship between fatigue characteristics of a specimen and the fabric's ability to hold asphalt as well as the fabric's surface texture. Fabrics A and F, thin slick fabrics not capable of retaining an appreciable quantity of asphalt cement, exhibited relatively poor fatigue performance. Fabric E, on the other hand, a very thick fuzzy fabric with the ability to retain a considerable quantity of asphalt, exhibited significantly longer fatigue lives than any of the other specimens.

It should be noted that increased fatigue performance at high asphalt tack rates may be attributed in part to excess asphalt which migrated into the hot asphalt mixture as a result of the kneading action during compaction. The additional asphalt cement will decrease air voids in the mixture and thus enhance fatigue performance. In order to dispel this notion, it would be desirable to fatigue test specimens with interlayers containing asphalt tack coat but no fabric.

RESISTANCE TO THERMAL REFLECTION CRACKING

The "overlay tester" (Figure 4), developed at Texas A&M University, is essentially a displacement controlled fatigue testing machine designed to initially produce a small crack (due to tension) in a test specimen and then continue to induce repetitive longitudinal displacement at the base of the crack which causes the crack to propagate upward through the specimen. This process is intended to simulate the cyclic stressing of a pavement due to periodical thermal variation. Results obtained with this apparatus should prove very useful in predicting pavement service life extension effected by systems purported to reduce reflection cracking.

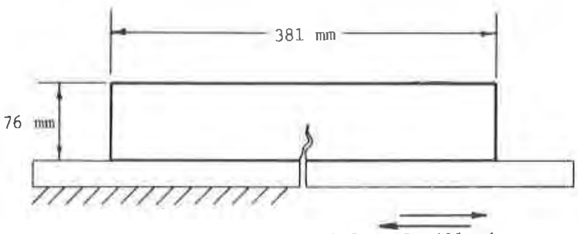

Figure 4. Skematic of Overlay Tester.

Construction materials and fabrication procedures for the specimens tested in this experiment were identical to those used for the flexural fatigue specimens. All test specimens containing fabric were made using only the optimum tack rate except those containing Fabric G which utilized the three tack rates. All tests were conducted at 25°C (77°F).

The machine was allowed to oscillate until complete specimen failure occurred, that is, until the crack propagated completely through the beam specimen. Ideally, complete failure would be defined as the cycle at which the load approached zero, however, with those specimens containing fabric, a measurable load was supported by the fabric even after the asphalt concrete specimen was completely ruptured.

Test results showed that the number of cycles to failure is proportional to tack coat quantity. This demonstrates the strain relieving ability of the thicker asphalt tack layer, however, it may be at least partly a result of migration of excess asphalt tack into the voids within the adjacent asphalt concrete during compaction which would increase tensile strength of the asphalt concrete.

Figure 5 shows average peak loads as a function of number of load cycles. On the average, those specimens containing fabric exhibit about six times more cycles to failure than the Control-1 specimens.

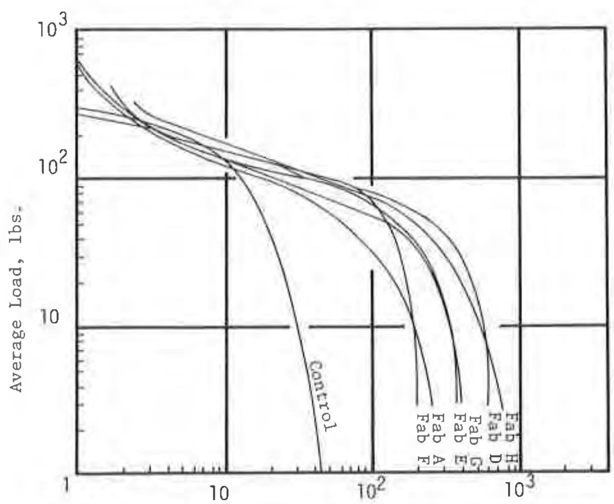

Figure 5. Peak Load Supported by Specimens Containing Different Fabrics.
2.2 lbs. = 1 kg.

GENERAL CONCLUSIONS

Based on the laboratory test results the following conclusions appear warranted:

1. To date, field performance is not well defined, however, fabrics show promise in retarding reflection cracking from pavement exhibiting fatigue distress,

2. At the current state-of-the-art, economic benefits to be gained from the use of fabrics in overlay applications are marginal.

3. Shrinkage of some fabrics associated with the high temperatures of newly placed asphalt concrete can cause premature cracking of the overlay. This cracking can be controlled by utilizing proper construction techniques and by modifying the fabrics shrinkage characteristics.

4. The potential for pavement slippage at the fabric-pavement interface is no greater for fabric overlay systems than for conventional overlays. Fabrics do not affect the interfacial shear strength of an asphalt overlay at the higher temperatures where shear strength becomes critical. Fabrics will decrease interfacial shear strength at lower temperatures where shear strength is already more than adequate.

5. Fabrics will improve pavement fatigue performance.

6. Fabrics will improve resistance to reflective cracking in asphalt concrete overlays.

7. Tensile properties of asphalt concrete is improved by the use of fabrics.

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A Mechanistic Design Procedure for Fabric-Reinforced Pavement Systems

Un procédé de modèle méchanique pour des systèmes de pavés en tissu renforcé

This paper presents a Mechanistic Design Procedure for fabric reinforced flexible pavement systems. A fabric reinforced pavement structure is one in which an engineering fabric or geotextile is embedded within the asphalt bound layer. For an overlay, the fabric is placed on top of the existing pavement. Purpose of the fabric is to enhance the fatigue life (or cracking resistance) of the pavement. Fatigue and rutting distress functions are the basis of the design procedure. Design for fabric reinforced pavements requires characterization of the fatigue life enhancement of the geotextile, called Fabric Effectiveness Factor (FEF) and determination of the in-situ stiffness of the existing pavement layers including subgrade. A laboratory fatigue testing procedure using a beam on an elastic foundation has been developed to determine a fabrics FEF. The laboratory test procedure is described in detail. Experience with FEF testing is summarized. Required characterization of the pavement layer properties needed for the design model is also presented. A design example demonstrates the design procedure.

Cet article traite un procédé de modèle méchanique pour des systèmes de pavés souples en tissu renforcé.Une structure de pavé renforcé de tissu est celle dans laquelle un tissu industriel ou géotextile est encastré dans une couche d'asphalte.Pour une couverture,le tissu est placé au dessus du pavé existant.Le but du tissu est de réhausser la durée de la détérioration interne (ou du craquage de la résistance) du pavé.Le modèle pour pavé renforcé de tissu demande une définition du réhaussement de la durée de la fatigue du géotextile,appelée Facteur d'Efficacité du Tissu (FEF) et la détermination de la fermeté originale des couches du pavé existant ainsi que la surface.Une procédure d'essai de fatigue en laboratoire,utilisant un rayon sur une fondation élastique , a été développée pour déterminer le tissu (FEF).La procédure d'essai en laboratoire est décrite en détail. L'expérience avec le FEF essai est resumée.La caractérisation exigée pour les propriétés de la couche du pavé nécessaire pour la conception du modèle est aussi présentée.Un modèle exemplaire démontre le procédé.

INTRODUCTION

The rational design of pavements must lead to the prediction of their performance during their service life. Overall pavement performance, as measured by its serviceability and maintainability under induced environmental and loading conditions, is directly related to the occurrence of pavement distress and associated pavement roughness. The engineering properties of paving materials and geometrical variables such as thickness and relative position of various component layers greatly contribute to the structural integrity of pavement systems, and thus the occurrence of pavement distress. The pavement material and its structural arrangement need to be selected to provide optimum serviceability so as to resist detrimental forces of load and environment and provide satisfactory performance.

This paper presents a Mechanistic Design Procedure for fabric reinforced flexible pavement systems. The design procedure is based upon the enhancement in fatigue life of asphalt concrete beams reinforced with geotextiles and tested in the laboratory. A fabric reinforced pavement structure is one in which an engineering fabric or geotextile is embedded within the asphalt bound layer. For a new pavement the fabric would be located between intermediate courses of asphalt concrete. For an overlay the fabric is placed on top of the existing surface as illustrated in Figure 1, prior to placement of the new asphalt concrete. Purpose of the fabric is to enhance the fatigue life (or cracking resistance) of the pavement when subjected to traffic loads. Only traffic forces are currently considered by the design procedure. Thermal induced cracking is not considered.

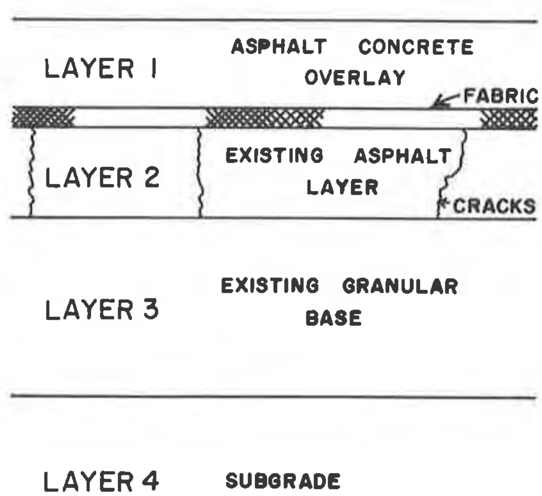


FIG. 1
FABRIC-REINFORCED ASPHALT OVERLAY

However other researchers have developed evidence that fabric reinforced pavements are also more resistant to thermal induced cracking.

DESIGN METHODOLOGY

Both rutting (distortion) and fatigue (cracking) distresses are considered by the design. Distress functions which utilize elastic strains at critical locations in the pavement are the basis of the design. For rutting distress vertical strain at the surface of the subgrade layer is the critical strain. For fatigue cracking distress, horizontal strain at the bottom of the asphalt bound layer is the critical strain. For overlays of severely cracked pavements horizontal strain at the bottom of the overlay is the critical fatigue cracking strain. These critical strain locations are consistent with other flexible pavement design procedures (1).

The phenomenological characterization of fatigue in bituminous mixtures has been extensively studied within the past few decades (2, 3). In this approach, the fatigue life of a flexible pavement, N_f, is related to the maximum tensile stress, ε_h, or tensile strain, ε_h, developed in the under-side of the bituminous layer by semi-empirical relations of the form:

$$N_f = c_1 \left(\frac{1}{\varepsilon_h}\right)^{m_1} \text{ for controlled stress tests} \quad (1)$$

and

$$N_f = c_2 \left(\frac{1}{\varepsilon_h}\right)^{m_2} \text{ for controlled strain tests} \quad (2)$$

where c_1, c_2, m_1, m_2, are constants to be determined experimentally on laboratory beam specimens using prescribed testing procedures. Despite certain inherent limitations, the phenomenological approach provides a reasonably simple procedure which has been widely accepted by various research and pavement design organizations. Recently a sophisticated pavement stress analysis program (1) was utilized to analyze AASHO Road Test data and develop the following fatigue expression:

$$N_f = 7.56\text{x}10^{-12}\ (1/\varepsilon_h)^{4.68} \quad (3)$$

Equation 3 is an improvement over laboratory fatigue equations since full scale field construction and real stress-dependent material properties were considered during development of the expression.

To reflect regional differences in pavement performance resulting from different climates and materials the Regional Factor (RF) defined by AASHTO (4) is applied. Fatigue life of unreinforced pavement structures is thus given by

$$N_{fu} = 7.56\text{x}10^{-12}\ (1/\varepsilon_h)^{4.68}\ /RF \quad (4)$$

In Figure 2, a typical laboratory distress function ε_h -N_f relation for an unreinforced mixture (line A), and a fabric reinforced specimen (line B) are compared. At a given strain level, the life for fabric-reinforced beam (N_{fr}) and life for unreinforced (N_{fu}) are estimated from Figure 2. The Fabric Effectiveness is then expressed as

$$\text{FEF} = \text{Fabric Effectiveness Factor} = \frac{N_{fr}}{N_{fu}} \quad (5)$$

Fabric Effectiveness Factor, represents the benefical effect of the fabric in reducing the fatigue distress and prolonging pavement life. Laboratory testing described in subsequent portions of this paper, has shown that FEF is dependent upon fabric location, thickness and stiffness of the asphalt layer, stress level, and fabric type. These variables which affect FEF are reflected in the expression used to calculate fatigue life for fabric reinforced pavements:

$$N_{fr} = N_{fu} \text{ x FEF } (i, \varepsilon_h) \text{ x GEO} \quad (6)$$

where FEF (i, ε_h) = fabric effectiveness factor for fabric i at strain $_h$

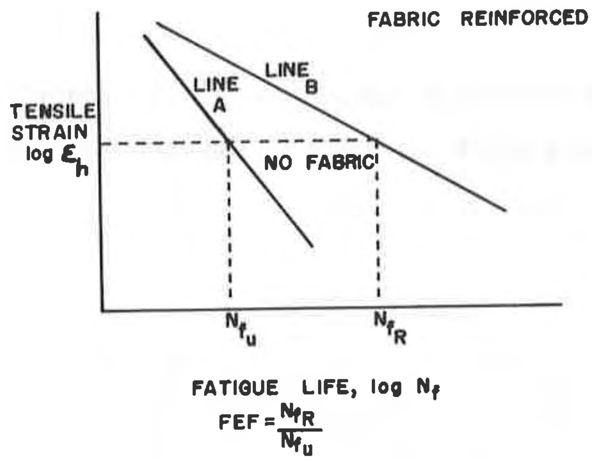


FIG. 2 FATIGUE LIFE VERSUS STRAIN

GEO = geometry correction accounting for the effect of placing the fabric at different depths within the asphalt layer.

The strain dependent FEF function is of the form:

$$\text{FEF} = a_1\ (\varepsilon_h)^{a_2} \quad (7)$$

where a_1 and a_2 are constants determined from laboratory testing on beams reinforced with the fabric.

The GEO function is a relationship between the ratio d'/z and reduction in FEF as the fabric is placed higher in the pavement structure (Figure 3). Depth of fabric from the top of the pavement structure is given by d', with z representing the depth of the neutral axis (zero horizontal bending strain) under traffic load. The GEO function currently used is based upon a limited number of experiments with fabrics placed at various positions within the asphalt layer. This function is given by:

$$\text{GEO} = 0.64\ (d'/z)^{1.60} \quad (8)$$

and will be updated as additional experimental data becomes available. A limiting GEO value of 1.00 is used to prevent the extrapolation of higher FEF's for pavements which have higher d'/z ratios than that used in the experimental beam study.

The rutting model, which designs against excessive repetitive consolidation and/or shear movements within the subgrade layer, utilizes a relationship between allowable vertical subgrade strain and number of load applications. The Dorman/Metcalf expression (5) was used to develop the relation:

$$N_{f\,rut} = 1.46\text{x}10^{-10} (1/\varepsilon_v)^{4.98}\ /RF \quad (9)$$

where $N_{f\,rut}$ = rutting life of pavement
RF = regional factor
ε_v = maximum vertical strain at top of subgrade layer.

Note that the same rutting life equation is used for both reinforced and unreinforced pavements. It is assumed that placement of fabric reinforcement within the asphalt layer does not have a significant affect upon rutting life of the underlying subgrade layer. Rutting considerations are important since the use of fabric reinforcement for fatigue life enhancement will yield a reduction in the required thickness of the asphalt concrete layer. The reduction in bound layer thickness will produce higher vertical stresses on the subgrade

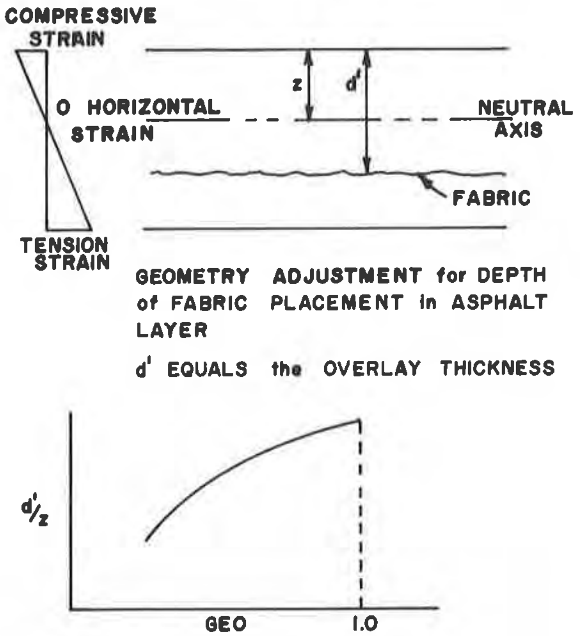


FIG. 3 GEOMETRY ADJUSTMENT TO FEF

which must be below tolerable levels. Use of the rutting distress function should ensure an adequate design.

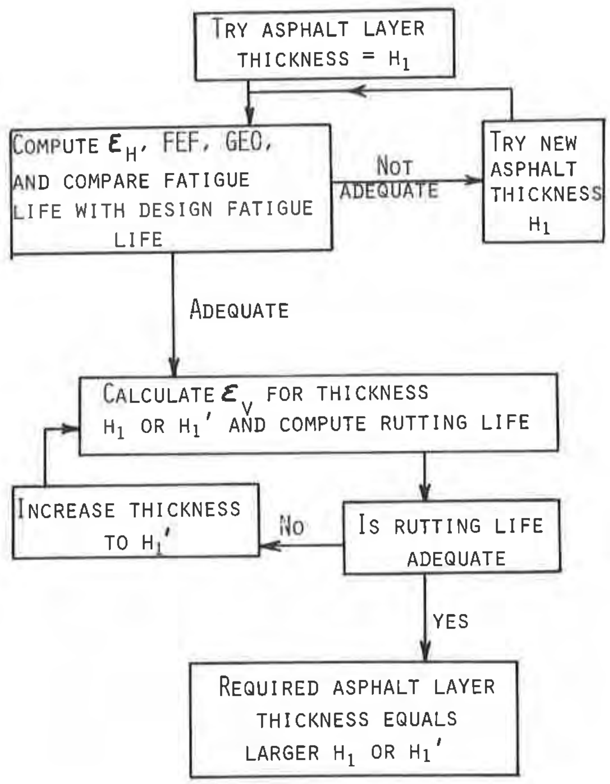
Equations 4, 6, 7, 8, and 9 form the model for design of fabric reinforced pavements. Table 1 summarizes the design methodology. For new pavements h_1 in Table 1 equals the total thickness of the asphalt bound layers while for overlay design h_1 is the overlay thickness. Similar logic is applied for both the unreinforced and fabric reinforced alternatives to permit comparison of required asphalt concrete thickness. Computed pavement strains under traffic load, a single 80 kN(18,000lb.)axle, are required for the procedure. In recent years several computer programs such as CHEVRON, ELSYMS, VESYN, and recently OAF have been developed to compute multilayered pavement stresses and strains (1, 6, 7). Any of these programs could be utilized to compute the necessary strains as input into the distress functions used by the design procedure.

LABORATORY CHARACTERIZATION OF FABRIC EFFECTIVENESS

The fatigue experiments were conducted using a beam on elastic foundation with geometry as shown in Figure 4. The selection of this experimental set-up was based on the two dimensional modeling of a pavement structure, in which a beam representing the pavement is supported on an elastic foundation representing the subgrade. The dimensions of the beam and foundation as well as the stiffness characteristics of foundation are selected with consideration to stress and strain at the bottom of pavement structure subjected to traffic loading. The test setup is the same as that used previously by researchers studying the fatigue properties of asphaltic mixtures (8),(12).

The fatigue tests were performed using a dynamic load function of Haversine shape. An MTS electro-hydraulic testing system was used to generate the load function. To insure complete recovery of the sample before

TABLE 1 DESIGN METHODOLOGY



the next load cycle, a rest period of .4 seconds was allowed between each load application. The duration of load application in all tests was kept constant at .1 seconds. All tests were conducted at a temperature of 22°C with tests conducted at a minimum of three stress levels per fabric. Tests were repeated to obtain statistically based FEF versus strain relationships.

The asphalt mixtures used in the preparation of fatigue specimens were selected to meet the Ohio Department of Transportation specifications for asphaltic concrete surface course, Item 404. The asphalt mixtures were prepared using limestone aggregate and an AC-20

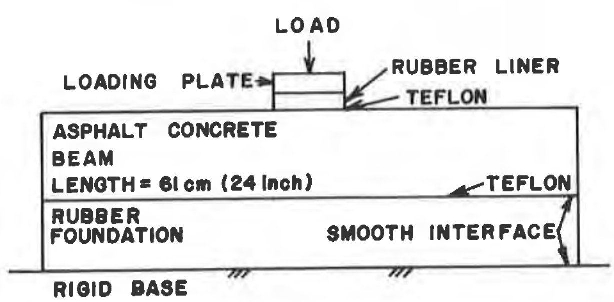


FIG. 4 SET-UP for FATIGUE TESTS

asphaltic cement. The asphaltic mixtures were prepared using an optimum asphalt cement content of 6.6% of total mix basis. The preparation of asphaltic mixtures followed recommended standard procedures for mixing and compaction of bituminous mixtures.

The fatigue specimens used in this investigation were beams with dimensions of 7.6 cm (3 in.) thick, 7.6 cm (3 in.) wide, and 61 cm (24 in.) in length. These specimens were prepared as laminated beams of 2.5 cm (1 in.) and 5.1 cm (2 in.) in thickness. The 5.1 cm (2 in.) layer is compacted on top of the previous manufactured 2.5 cm (1 in.) thick layer. Beams are compacted in steel mold by a testing machine. The compaction pressure is increased until the asphaltic mixture is compacted to desired thickness. The pressure is then maintained constant for about 2-3 minutes, until pressure stabilizes and asphaltic mixture attains constant density. The compacted test specimen is then removed and placed at room temperature for a minimum of 24 hours prior to testing. The lower part of the fatigue specimen, a beam 2.5 cm x 7.6 cm x 61 cm (1 in. x 3 in. x 24 in.) is manufactured first. If the sample is to be a fabric reinforced specimen, the surface of the 2.5 cm (1 in.) thick beam is tack coated as per fabric manufacturers published recommendations. AC-20 at an application rate of 1.1 liter/m^2 (.25 gallon/sq. yd.) was the most commonly used tack coat. Precut 7.6 cm x 7.6 cm x 61 cm (3 in. x 3 in x 24 in.) fabric strips were placed on the sticky tack material.

After the placement of fabric on the top of tack coated base beam, the fabric surface is smoothly brushed to bring it into complete contact with the binder. The prepared base beam (2.5 cm thick beam, tack coat and the fabric) is then placed back into the mold and sufficient amounts of asphaltic mixtures required for fabrication of 5.1 cm (2 in.) thick specimens are weighed and placed into the mold. A leveling device is passed through the loose material for even distribution of the mixture in the mold. Similar to the procedures used in the fabrication of one inch specimens, the mixture is pressed under steadily increasing load, until a 7.6 cm (3 in.) thick specimen is obtained. The mold is dismantled and the specimen is carefully removed. The fabricated fatigue specimen is placed on a stiff support to await testing. All precautions are taken to prevent bending and any possible damage to the beam sample prior to testing. Control samples without fabric were manufactured in the same manner except no tack coat was used between the 2.5 cm and 5.1 cm layers. Fatigue test data for fabric reinforced beams was compared with that of the control (no fabric) samples to determine the constants a_1 and a_2 for the FEF strain relationship presented in equation (7). Fatigue testing of over 600 fabric reinforced beams indicated the following general conclusions regarding the ability of geotextiles to enhance fatigue life:

1. FEF for most lightweight, about 135g/m^2 (4 oz/sq. yd.) nonwoven or woven fabrics is in the 3 to 10 range;
2. FEF tends to increase as fabric weight increases;(Research is in progress to identify other fabric properties affecting FEF).
3. FEF generally increases as stress or strain level decreases.

and 4. Tack coat quantity and type recommended by manufacturers generally produces optimum FEF performance. However tack coat influence on FEF is being further investigated.

MATERIAL PROPERTIES AND REQUIRED PAVEMENT EVALUATION

Computation of traffic load stresses and strains in a pavement structure requires characterization of the elastic modulli or stiffnesses for each layer of the pavement. For a new pavement laboratory testing of compacted asphalt concrete, granular base (with confining pressure), and undisturbed soil subgrade samples is performed to determine stiffness of the material (7), (11). In lieu of laboratory testing modulli values can be assigned based upon previous tests or experience with similar materials. Assigning of modulli rather than performing laboratory testing should only be done for low volume highways, or small resurfacing projects.

Characterization of existing pavements by laboratory testing is more difficult since undisturbed sampling is expensive and many samples are usually required to reflect actual variability in field conditions. Therefore current state-of-the-art pavement evaluation techniques are based upon nondestructive deflection measurements obtained along the existing roadway. There are several commercially available deflection measuring devices including Dynaflect, Road Rater, and Falling Weight Deflectometer (1, 9, 10). Pavement evaluation can be conducted using any of the devices.

Nondestructive testing is a fast and economical way to obtain a large quantity of test information about a roadway. The development of interrelations between deflection and pavement structural performance data back to the 1950's where the concept of allowable deflections to produce a certain pavement life was formulated by California Highway Department and subsequently at the AASHO Road Test. During the early 70's techniques were developed to "back-calculate" in-situ stiffnesses of pavement layers from deflection measurements (10)for known layer thicknesses. A recently developed U.S. DOT-FHWA flexible pavement overlay design procedure is based upon calculation of pavement layer modulli from deflection measurements (1). For overlay design of high type roadways it is suggested that one of the techniques to back-calculate modulli be used to characterize the existing pavement. In addition to nondestructive testing pavement cores should be obtained at a rate of 1 core per 5000 m^2 of pavement to determine layer thicknesses.

When cracks develop in a pavement there is a reduction in the "effective" stiffness of the pavement from that of the no crack condition. Therefore modulli back-calculated from deflection measurements are usually lower than those calculated for laboratory samples. Therein lies the advantage of the nondestructive technique, since the ability of the materials to perform in pavement system under field conditions is evaluated.

It is likely that nondestrictive will not be readily available or economically feasible for some projects. Although not preferred, visual condition survey of pavement cracking could be used to assign a stiffness to the existing asphalt concrete layer. Tables 2 and 3 present a relationship between effective modulus of an existing asphalt concrete surface and severity and extent of wheel track cracking. There has been very little research relating visual condition of a pavement to effective stiffness. The information in Tables 2 and 3 should be considered as preliminary, subject to verification by future research.

Similarly, Table 4 presents ranges of modulus for granular base, subbase and subgrade materials. Again, it is preferable to use laboratory testing or nondestructive measurements to calculate the modulus values. Once the modulus values are known, and the fabric has been characterized in terms of FEF the methodology presented in Table 1 can be used to design a fabric reinforced pavement.

DESIGN EXAMPLE

The following example illustrates the procedure for design of a fabric reinforced overlay.

1. Conditions

The existing flexible pavement consists of a 150 mm (6 in.) asphalt concrete layer over 75 mm (3 in)granular

TABLE 2. SEVERITY AND EXTENT OF WHEEL TRACK CRACKING

DESCRIPTION

CRACKS LOCATED WITHIN OR NEAR THE WHEEL TRACKS. WHEEL TRACK CRACKING USUALLY STARTS AS INTERMITTANT SINGLE LONGITUDINAL CRACKS PROGRESSING TO MULTIPLE LONGITUDINAL CRACKING, AND EVENTUALLY INTERCONNECTED OR ALLIGATOR CRACKING. WHEEL TRACK CRACKING USUALLY RESULTS FROM FATIGUE FAILURE OF THE ASPHALTIC LAYER. REFLECTION OF UNDERLYING CRACKS IN OVERLAID PAVEMENTS CAN ALSO RESULT IN WHEEL TRACK CRACKING.

SEVERITY LEVEL

SEVERITY IS BASED UPON BOTH CRACK WIDTH AND MULTIPLICITY OF THE CRACKING. BOTH CRITERIA MUST BE SATISFIED WHEN ASSIGNING SEVERITY LEVEL.

LOW: SINGLE OR INTERMITTANT MULTIPLE CRACKING WITH AVERAGE CRACK WIDTH LESS THAN 3 mm (1/8 in.) OR BARELY NOTICEABLE.

MEDIUM: MULTIPLE CRACKING (MAY ALSO INCLUDE REGIONS OF INTERMITTANT ALLIGATOR CRACKING) WITH AVERAGE CRACK WIDTH GREATER THAN 3 mm (1/8 in.) WITH LITTLE SPALLING OR LOOSE PIECES.

HIGH: MULTIPLE CRACKING WITH EXTENSIVE ALLIGATOR CRACKING. SPALLING IS FAIRLY COMMON WITH AVERAGE CRACK WIDTH GREATER THAN 6 mm (1/4 in.) AND SOME ALLIGATOR BLOCKS ARE EASILY REMOVED.

EXTENT LEVEL

EXTENT IS BASED UPON PERCENTAGE OF THE WHEEL TRACK LENGTH WITHIN THE SECTION WHICH EXHIBITS CRACKING:

OCCASIONAL - LESS THAN 20%
FREQUENT - BETWEEN 20 AND 50%
EXTENSIVE - MORE THAN 50%

TABLE 3. REDUCED "EFFECTIVE" MODULUS OF CRACKED ASPHALT LAYERS

	CRACK SEVERITY LEVEL		
EXTENT LEVEL	LOW	MEDIUM	HIGH
OCCASSIONAL	2585 MPa (375,000 PSI)	1550 MPa (255,000 PSI)	1045 MPa (150,000 PSI)
FREQUENT	2080 MPa (300,000 PSI)	1045 MPa (150,000 PSI)	430 MPa (60,000 PSI)
EXTENSIVE	1560 MPa (250,000 PSI)	430 MPa (60,000 PSI)	245 MPa (35,000 PSI)

TABLE 4. SUGGESTED MODULLI OF GRANULAR LAYERS

GOOD QUALITY GRANULAR BASE	245 MPa (35,000 PSI)
LOW QUALITY GRANULAR BASE	175 MPa (25,000 PSI)
GOOD QUALITY GRANULAR SUBBASE	140 MPa (20,000 PSI)
LOW QUALITY GRANULAR SUBBASE	85 MPa (12,000 PSI)
SUBGRADE	
CBR* =10 -	105 MPa (15,000 PSI)
CBR = 6 -	60 MPa (9,000 PSI)
CBR = 3 -	30 MPa (4,500 PSI)

* CBR IS CALIFORNIA BEARING RATIO (11).

base over a silty clay subgrade. Deflection testing indicates in-situ stiffness of the pavement layers of 1045 MPa (150,000 psi) for the existing asphalt concrete, 156 MPa (25,000 psi) for the granular base and 43 MPa (6000 psi) for the subgrade. The design traffic is 80 daily 80 kN (18 kip) axle loadings for a design period of 10 years. The AASHTO Regional Factor is 1.5. The required fatigue and rutting life (DESL) are given by:

DESL = (80) (10) (365) = 292,000 applications

2. Strain and Fatigue Analysis

The multilayer elastic program OAF (1) was used to calculate pavement stresses, strains and neutral axis position for various overlay thicknesses. The modulus for the new asphalt concrete was 3345 MPa (480,000 psi). Only the strains corresponding to the required overlay thickness are presented here.

2a. Unreinforced (No Fabric Design)

With overlay thickness of 76 mm (3.0 in.) the horizontal strain of the bottom of existing asphalt layer is 262 microstrain. The fatigue life given by equation (4) is

$$N_{fu} = 291{,}600 \text{ applications}$$

N_{fu} nearly equals DESL, therefore the required overlay thickness without fabric reinforcement is 76 mm(3.0 in.). Calculation of ε_v with a 76 mm overlay and determination of rutting life by equation (9) indicates that rutting life exceeds DESL, therefore fatigue governs the design.

2b. Fabric Reinforced Design

With overlay thickness of 44 mm (1.7 in.) the horizontal strain at the bottom of the existing asphalt layer is 330 microstrain. The depth of zero bending strain (neutral axis, z) is 38 mm (1.5 in.). The fatigue life given by equation (4) is:

$$N_{fu} = 99{,}030 \text{ applications}$$

For fabric placed on the surface of the existing pavement prior to overlay, GEO is calculated by equation (8) with d' of 44 mm and z of 38 mm:

$$GEO = .80$$

Assume the fabric has a FEF of 3.7 at ε_h of 30 microstrain. Then the fatigue life of the fabric reinforced overlay is given by equation (6) as:

$$N_{fr} = 99{,}030\ (3.7)\ (.80) = 293{,}100 \text{ applications}$$

N_{fr} nearly equals DESL, therefore the required overlay thickness with fabric reinforcement is 44 mm (1.7 in.). Calculation of ε_v with a 44 mm overlay and determination of rutting life of equation (9) indicates that rutting life exceeds DESL, therefore fatigue governs the design.

3. Conclusions

For the conditions given in the design example, a savings of 32 mm (1.3 in.) in asphalt concrete thickness is indicated by using a fabric to reinforce the overlay. The actual decision of whether or not to utilize fabric reinforcement should consider not only fatigue life enhancement but also other considerations including fabric cost, fabric laydown and handling characteristics, and bonding properties between the asphalt and fabric. Also the methodology presented for design of fabric reinforced pavement structures is based upon laboratory data. Future correlation with field test results is needed to verify or calibrate the design procedure.

ACKNOWLEDGEMENTS

The authors have been working with various geotextile manufactures to characterize the fatigue performance of fabric reinforced asphalt systems since 1972. Sponsors have included Monsanto Textile Company, Phillips Petroleum, Dupont Chemical Company, Johns Manville Company, International Paper Company, and U.S. Department of Transportation (Federal Highway Administration and Federal Aviation Administraion).

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Long Term Performance of MESL Road Sections in Australia

Essais de comportement de longue durée des chaussées MESL en Australie

Two pavement sections installed in 1974 in main roads in Australia, using the MESL method (basecourse material wholly enclosed in a geotextile) have been monitored ever since for their performance under traffic. This paper describes that performance over 7 years, giving information on the durability and performance of an MESL. The effectiveness of the design procedure, which had novel theoretical aspects, is also demonstrated.
The geotextile, whose upper surface is buried only 100 mm below the road surface, remains in excellent condition. Though the soil within each MESL was of lower than standard basecourse quality, both sections have performed extremely well, in one case so far better than the control sections that the latter have required reconstruction, whilst no distress is apparent in the MESL. The stress-strain characteristics of the geotextile appear to have conferred a degree of resilience under heavy traffic, since no appreciable permanent deformation was measured. The implications of these field trials for future practice are discussed

On a mesuré le comportement, depuis 1974, de deux sections de chaussée construites en Australie, selon la méthode MESL (une couche de base entièrement enveloppée d'un textile). Ce rapport donne des détails de ce comportement, et de la durabilité de ces sections MESL.
L'efficacité de la méthode du dessin, qui a des éléments théoriques nouveaux, est aussi demontrée.
Le textile, dont la surface supérieure n'est emplacée que cent mm au dessous de la surface de la route, reste encore dans un état excellent. Tous les deux sections ont rendues un comportement très supérieur, en dépit d'une qualité inférieure de la couche de base dedans le MESL. Ce comportement fut si nettement supérieur à l'un des deux sites, qu'il faut reconstruire aujourd'hui à cet endroit les contrôles non-MESL, tandis que le MESL n'a souffrit jusqu'encore aucun défaut. Il parait que le textile ait donne une certaine elasticité à la chaussée sous le traffic lourd, puisqu'on ne peut trouver aucune déformation permanente. On discute ensuite les implications de ces essais in situ pour la pratique future.

1. INTRODUCTION

The MESL (Membrane Encapsulated Soil Layer) concept has been successfully used in many short term field trials (for a full historical account, see Lawson(1)), and has been judged one of the most useful and practical concepts for future soil stabilization (Bell,(2)).

But because robust and durable geotextiles for civil engineering work have become available commercially at competitive prices only in the last decade, results of long term field trials of such materials have not yet appeared in the scientific literature.

This point is especially important considering that the organic nature of commercial geotextiles has frequently exposed them to criticism on the grounds of durability. As is well known, the durability of construction material is one of the most difficult parameters to measure, since no accelerated weathering test is able to simulate exactly the very complex field conditions which may be encountered.

This report therefore is thought to represent one of the first long term field appraisals, under normal service conditions, of an MESL (or its individual plastic layers) in a road pavement. Two test sections, with different subgrade and traffic patterns, have been intermittently monitored over a period of 7 years: equal to 50% of the current normal life expectancy* of a flexible pavement in Australia (12 to 15 years).

* i.e., to major maintenance: overlay, reconstruction, etc, Though the design life is usually reckoned for 20 years, this is not achieved in practice due to underestimated traffic growth, environmental factors, etc.

The test sections, described presently, were designed according to new principles particularly suited to semi-arid zones; but which are also relevant in other areas - e.g. frost regions - by appropriate adaptation of the design method. These principles were enunciated by Ingles (3) at the 1st International Conference on Geotextiles.

2. DISCUSSION

To provide adequate long term performance, the membrane itself must remain in a stable condition and must continue to perform its required functions throughout the design life of the pavement structure.

It has long been recognised that geotextiles incorporated in a road pavement may exert

(i) *a separation function*# - preventing loss of coarse material into finer, improving the compactability of superimposed layers, etc.
(ii) *a reinforcement function* - increasing tensile strength in unbound layers, inhibiting shear and cracking failures, etc.

Though we are well aware that this latter function has been challenged by proponents of the "analytical approach" to pavement design (e.g. Harrison & Gerrard (4); Brown, Brodrick & Pappin (5)), field performance has failed to

\# This function can sometimes be utilised as a *filtration* function, provided the pore sizes in the geotextile are controlled to allow the passage of liquid water and the retention of soil particles.

support* such analyses because, we believe, of defects in the modelling which assumes that only low strains are allowable - an overdesign situation quite inappropriate to these materials.

Much less well recognised, except in drainage applications, has been their potential role in

(iii) *a permeability control function.*

As pointed out by Ingles & Lawson (6), and Ingles (3), this role assumes paramount importance in regions of intermittent, seasonal, or sparse rainfall (and, by extension, of seasonal frost, etc.). In such conditions it is not essential that the geotextile be impermeable, only that its pore sizes be sufficiently small that water transfer is by vapor phase rather than by bulk flow.

For road pavements incorporating MESL's, the membrane imparts improved pavement performance by utilising these 3 basic membrane functions - boundary separations of distinct soil layers (the separation function), suppression of crack propagation (a form of reinforcement function), and control of moisture movements in the pavement.

The membrane encapsulating a soil acts as a separator, preventing intermixing of the surrounding soils with the encapsulated soil. This can be of particular benefit at the *underside* of the MESL if the encapsulated soil layer is constructed directly on top of a highly moisture-susceptible (strengthwise) subgrade (e.g. silts and clays). If the moisture-susceptible subgrade is prone to severe wetting during periods of excess moisture, then the bottom membrane of the MESL will prevent migration (under traffic) of the subgrade material into the encapsulated soil layer. Such migration causes premature pavement failure. To act as a separator over a long period of time, the membrane must have the required stress/strain properties which will render it immune to damage during installation, and that caused by traffic stresses. The pore sizes of the membrane must also be small enough to control soil particle migration.

The membrane may also act to prevent the migration of cracks through both bound and unbound materials in contact with it. This could be of advantage at the underside of the MESL if the subgrade was subject to volume change; but more than likely be of most advantage at the *top* of the MESL where the upper membrane could inhibit the propagation of cracks in the surface layers of the pavement, caused by horizontal stress components of the traffic loading. For maximum crack propagation suppression in the upper pavement layers, it is important for the membrane to be placed at the underside of the layer of material to be protected. To control the propagation of cracks it is important for the membrane to behave in an elastic manner and to be robust enough to resist traffic-induced stresses. Another important requirement is for the membrane to be bonded adequately to the surrounding material so as to minimise membrane slippage.

The use of membranes to control the flow of moisture has also long been recognised. However, the requirements for a water-repellancy function have tended to concentrate on the realisation of highly impermeable characteristics (e.g. plastic liners). Less well recognised has been the potential for commercial geotextiles to be utilised for moisture *control*.

To control moisture movement (both liquid and vapor) it is essential that the wetting pressure (sometimes termed water head support) of the membrane remains greater than the water pressures in the surrounding soil. When this condition is observed, water vapor movements are the only moisture flows which can occur across the boundaries of the membrane.

The rate at which water vapor migrates across the boundaries of a membrane can be determined by the use of Fick's Law:

$$\frac{\partial w}{\partial t} = K.A.\frac{\partial c}{\partial x} \tag{1}$$

where, $\frac{\partial w}{\partial t}$ is the rate of water vapor diffused (by weight)

$\frac{\partial c}{\partial x}$ is the vapor pressure gradient acting across the membrane

A is the surface area of the membrane

K is a diffusion constant for the membrane

The diffusion constant K is a measure of the rate at which water vapor permeates through the membrane, and is termed the membrane *permeance#* (as distinct from membrane permeability, k, which determines the flow of bulk (liquid) water).

The degree of water vapor migration control required of a membrane in an MESL pavement may be determined using the following relationship (assuming that moisture can penetrate the MESL from all directions):-

$$K = \frac{(w_{t_n} - w_{t_0}).\gamma_d.H}{2(t_n - t_0).\overline{\Delta c}.F} \tag{2}$$

where, $(w_{t_n} - w_{t_0})$ is the allowable moisture content change in the MESL over the time period $(t_n - t_0)$

γ_d is the dry density of the MESL

H is the thickness of the MESL in the pavement

$\overline{\Delta c}$ is the mean vapor pressure difference across the membrane during time $(t_n - t_0)$

F is a safety factor (normally equal to 2)

Since commercially available geotextiles very rarely exhibit wetting pressures greater than 80 mm, it is normally necessary (for design purposes) to apply an inert filler compound to the geotextile to ensure that the wetting pressure of the membrane remains greater than the external water pressures, and to control the migration of water vapor. Relatively small additions of inert filler compounds (saturants) are required to reduce significantly the permeance of the geotextiles, as is shown in Figure 1.

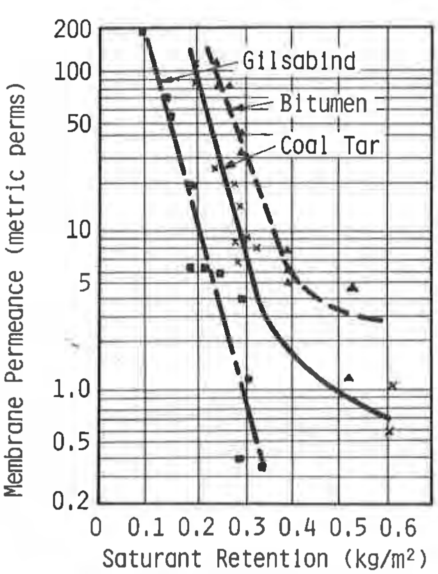

Figure 1 : Relationship between Membrane Permeance and Saturant Retention Weight for an impregnated 140 g/m² Melt-bonded Non-woven Geotextile (after Lawson (7)).

* cf. Ingles & Lawson (6)

\# An A.S.T.M. standard test exists for the measurement of the constant K - A.S.T.M. E96, 1966.

If the maximum duration time for saturated conditions external to the MESL can be calculated from climatic records, the foregoing equation (2) can be used to calculate the geotextile permeance required to reduce moisture changes within the MESL below any chosen level. The saturation technique makes reductions of geotextile permeance by more than 2 orders of magnitude relatively easy (see Fig. 1).

During the dry season following saturation of a soil, the moisture content within an MESL will again fall. The role of the geotextile is thus to reduce (markedly) moisture changes in the protected pavement layer. This in turn allows thickness calculations to be made, not on the basis of soaked strengths, but rather on the basis of construction moisture content strength or near thereto: with considerable economy of materials. It permits also a resistance even to totally flooded conditions within the design period; and it allows the use of otherwise inferior (i.e. non-standard) materials inside the MESL, thus increasing local resources of pavement materials. A further benefit is that compaction can be effected somewhat drier of optimum, using heavier rollers, giving additional strength to the pavement layer.

Fully to utilise the maximum benefit of all three membrane functions, it is important that the bottom membrane layer of the MESL be in contact with the top of the subgrade (for possible separation benefit) and the upper membrane layer of the MESL be as near the surface of the pavement as is practicable having regard for the expected traffic stresses at that level. This maximises possible crack suppression benefits.

Achieving maximum benefit from the membrane would result in the MESL forming a substantial unit of the pavement structure, with the encapsulated soil required to resist high traffic loading stresses in the upper part of the MESL. Most soils, even heavy clays, exhibit high deformation resistance when compacted (CBR's 80 - 110%) provided the ingress of moisture is controlled. Thus the extension of the MESL from subgrade level to the upper levels of the pavement should not present a problem, provided the encapsulating membrane is able to control moisture movements into and out of the encapsulated soil layer.

The principal problems involved in achieving these objectives are

(a) the lack of guidance data on thickness and placement depth for such MESL layers in a pavement, and
(b) whether the durability of the construction is comparable to that of normal pavement construction materials.

To examine these matters, two sections of MESL construction were laid in 1974 in inland Australia, on main roads, and their performance monitored intermittently since that time. Both sections carry approximately the same traffic loading in terms of standard axles (circa 240 per day), but the traffic pattern is different, being in one case 2000 v.p.d. with 5% heavy, in the other 400 v.p.d. with 35% heavy.

The performance of these MESL sections has been compared with those of control sections, constructed according to normal practice, immediately contiguous to each end of the MESL.

In one MESL section, an attempt was made to obtain information pertinent to the minimum permissible design thicknesses by tapering the MESL section to a minimum of 100 mm. (Site 2). At the other site (Site 1), the MESL section was deliberately placed across a T-junction so that any adverse influence of turning vehicles could be detected. The sections adopted are shown in Figure 2.

Both sites were subject to water infiltration into the subgrade, and possessed low CBR values even in the dry season. Subgrade properties at the two sites are shown in Table 1. It should be noted that even assuming the best (dry season) values of CBR, pavement thickness required by current Australian rules (NAASRA,(8)) is 320 mm. Though the control sections appear somewhat underdesigned by this standard, such practice has been usual in the semi-arid interior, and performance is normally found to be satisfactory because stage construction procedures mostly result in some old relict pavement material stiffening the subgrade immediately below the new pavement.

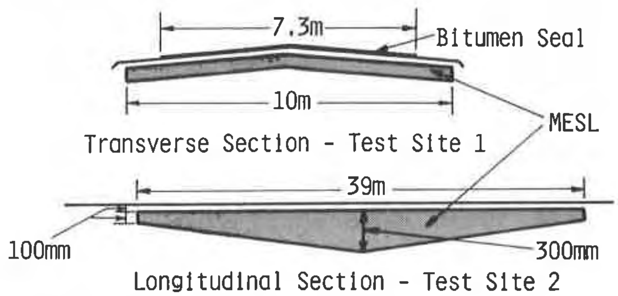

Figure 2 : Sections at Sites 1 and 2

Table 1. Test Site Soil Conditions

	Site 1 subgrade	Site 2 subgrade	Encapsulated Soil
Soil Type	MH/CH	MH/CH	GW
CBR "wet season"	4%	3%	-
CBR "dry season"	7%	7%	-
Depth of water table			
"wet season",mm	350	50	-
"dry season",mm	750	450	-
Grading			
Gravel (%)	nil	nil	51
Sand (%)	36	36	23
Silt + Clay (%)	65	62	26
Plasticity Index (%)	34	30	9
Linear Shrinkage (%)	14	14	5
Specific Gravity	2.69	2.69	2.65
Cover required (CBR = 7)	320 mm	320 mm	-
Cover provided (control)	300 mm	200 mm	-
(MESL)	320 mm	200 to 400 mm	-

3. PERFORMANCE OF THE TEST SITES

Monitoring at the test sites comprised detailed surface profile surveys to detect permanent deformations, Benkelmann beam deflection surveys to detect elastic strains, and moisture content measurements both within and without the MESL: together with pavement condition surveys to detect cracking, etc.; and a roughness count at each site. A photographic record has been maintained throughout.

The short term performance has been reported elsewhere (Ingles & Lawson,(6)). The surface profile measurements were pursued for some 5 years, without significant rutting being detected in either MESL (less than 5 mm maximum) and only minor rutting (5mm) found in one control section at Site 2. These results have also been reported elsewhere (Lawson & Brunner,(9)), with details of the measurement techniques. No measureable longitudinal movements were observed (accuracy attained, ± 0.5 mm). However, transverse movements in the surface layer, reaching to 15 mm, occurred on the low side at test site 2.

Longitudinal gradient at both sites was negligible, but the transverse gradient at site 1 averaged 1.5% and at site 2 averaged 3%. It is interesting to note that the higher gradient at site 2 corresponded with transverse movements which were greatest in the very areas where subsequent distress appeared (see Figure 3).

Of more immediate interest are the beam deflections, and the moisture content changes, as well as pavement condition changes. The two sites will be discussed separately.

At site 1 (with T-junction and uniform MESL thickness), the MESL has fulfilled its role of "smoothing-out" moisture fluctuations admirably. It remains, as at the time of construction, *drier* than the control. Deflection follows a similar pattern, showing much steadier values over the MESL section than over the controls. Though all deflections are somewhat higher than would normally be desired,# the MESL shows the better deflection. At site 1, *all* sections continue to perform very well, notwithstanding: and no signs of any type of failure (except perhaps some edge fretting along the control sections only) can be discerned.

Though these observations do not thus permit any firm conclusions to be drawn about the relative performance of the MESL and the normal construction sections (but residual life has been calculated to be three times greater in the MESL than in the northern control, on the basis of the deflection readings), they do confirm that the MESL is a durable construction under traffic, and that it does perform the envisaged design role of controlling moisture fluctuations. Despite a thickness slightly less than the NAASRA design standard, performance suggests that no section is underdesigned. This is thought to be due, at least partly, to some old pavement gravel remnants below the present basecourse : and core recoveries have suggested that a depth of 420 mm to wholly natural subgrade might apply. This would correspond reasonably well with the NAASRA design thickness requirement over CBR 4 (440 mm), which is near to the wet season value for this site.

At site 2 rather different circumstances pertain. Here the subgrade is almost permanently saturated by water seeping from irrigation trenches parallel with the road. One of these trenches is, indeed, superelevated with respect to the road. The magnitude of this effect was not wholly appreciated at the time of construction, with the result that the MESL section at this site has no significant dry-out period and has, in fact, steadily wetted up internally until a saturated or near-saturated condition applies.* This has, in turn, resulted in very high beam deflections, both over the MESL and also the controls.

Despite these adverse conditions for pavement performance, the MESL has continued to perform impeccably at site 2, and retains its original surface shape without any indications of failure at the pavement surface. The control sections have, by contrast, so far deteriorated that extensive alligator cracking was already in evidence before 1979, and necessitated extensive patching. Complete reconstruction of the control sections has now had to be scheduled, whilst the MESL remains sound. Figure 3 shows the surface condition at site 2 in November 1979, and further deterioration has followed.

Moisture content within the MESL and in the external controls at this site has now approximately equalised, and the deflections in both are accordingly similar. Hence the break-up of the control sections - chiefly due to cracking caused both by subgrade failures and transverse movements in the surface layer - can only be attributed to a *lack* of the internal confinement and shear restraint that is afforded by the MESL. Again, the long term field evidence has confirmed the excellent durability of the geotextile itself under service conditions: recovered samples showed no evidence of any deterioration after cleaning from soil and saturant (in this case, bitumen).

Wallace (10) considers that for traffic of the present type, beam readings should not exceed about 1.0 mm.

* Moisture content within the MESL is now almost 50% above optimum moisture content, due partly also to the original construction density falling below standard.

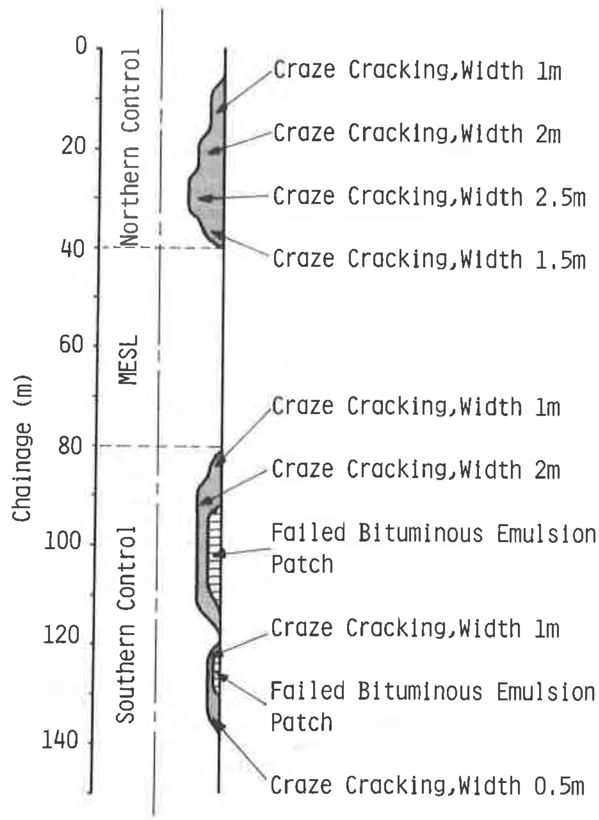

Figure 3 : Surface Condition Map at Site 2, November 1979+

Table 2 displays the moisture and deflection behaviour at the two test sites. The data suggest thata "settling-in" period of 2 - 3 months after construction may have applied for the beam deflections.

Table 2. Moisture Content and Deflection Changes at the Test Sites++

Reading Date		May 1975	Sept. 1975	Dec. 1975	May 1976	Aug. 1976	Dec. 1976	May 1979	Jan. 1982
Moisture	control*	9.3	12.3	10.9	10.5	12.6	11.9	10.0	-
Content#	MESL	9.4	8.4	8.6	7.6	7.4	7.7	7.0	-
(Site 1)									
Deflec-	control	2.2	0.8	1.8	1.0	1.3	1.9	1.6	1.0
tion**	MESL	2.2	1.3	1.5	0.8	0.8	1.0	1.0	0.8
(Site 1)									
Moisture	control	17.1	19.0	-	18.1	17.7	18.0	16.0	-
Content#	MESL	10.0	13.2	-	15.0	15.0	15.5	15.8	-
(Site 2)									
Deflec-	control	0.8	1.3	1.0	0.8	0.8	0.9	2.3	1.5
tion**	MESL	1.8	1.2	-	1.0	1.2	1.4	1.6	1.4
(Site 2)									

* control values averaged over both sides
# in %, dry basis.
**in mms, by Benkelmann beam

\+ crack widths shown are greater than 3 mm
++ optimum moisture content of the MESL and basecourse (control) material was 11.0% (standard compaction)

4. IMPLICATIONS AND CONCLUSIONS

The performance of these MESL pavement sections has important implications for conventional pavement design procedures, inasmuch as new materials (membranes) and unsaturated soil strength parameters may be used in the design of more economical pavement structures. The MESL sections have shown that membranes perform adequately over a long period of time with no visible loss of performance due to membrane degradation or function reduction.

The degree of moisture control the encapsulating membrane exerts on the encapsulated soil enables materials which would normally be rejected because of low saturated strength parameters to be utilised as structural layers in a pavement. Particular examples of where MESL's could be used effectively are to stabilize substandard granular materials, to stabilize expansive or heavy clays, to stabilize frost susceptible soils, and to protect against pavement failure due to unforeseen "random" events (e.g. flooding).

Certain types of granular materials are considered substandard for road pavements if their saturated strength parameters are too low to support repeated traffic loads. Most of these materials have high deformation resistance provided they are kept in an unsaturated condition. The MESL can be used to control the migration of moisture into these materials, thus ensuring maintenance of the required deformation resistance. An added benefit of the MESL for this application is that if the subgrade becomes saturated then the bottom membrane layer will also act as a separator, inhibiting intermixing of subgrade and MESL soil particles and thus protecting against premature pavement failure due to loss of effective subgrade support.

Expansive and heavy clays may be used as structural pavement layers when encapsulated in a controlled permeability membrane. The membrane protects the expansive (or heavy) clay from the effects of fluctuating external moisture content changes, thus ensuring negligible volume change or cracking of the encapsulated soil. The membrane could also inhibit the propagation of cracks up through the MESL if the subgrade was also subject to fluctuating moisture conditions.

Frost susceptible soils could be encapsulated in controlled permeability membranes to ensure that critical loss of strength does not occur during the thaw period. The membrane of the MESL would inhibit the entry of water into the frost susceptible soil layer during the freeze cycle, thus preventing saturation of the MESL during the thaw. The bottom membrane layer of the MESL could also act as a separator, to prevent loss of subgrade support from the unstabilised frost susceptible subgrade during the thaw period.

Protection from random events such as flooding, could be afforded by an MESL, using a controlled permeability membrane. The membrane would inhibit the entry of excess moisture into the encapsulated pavement layer during periods of flooding. This enables the encapsulated pavement layer to remain structurally sound throughout. The membrane would also protect the encapsulated pavement layer from scour during the flood inundation.

From observations of two MESL test sites in main roads in Australia, made over a period of 7 years, corresponding to approximately 0.3×10^6 standard axles, a substantial part of the design life. It is concluded from measurements of deflection, permanent deformation, moisture changes and visual observation of the pavement surface conditions, as well as recovered sections of the geotextile, that:

(i) *no degradation* is yet observable in the geotextile membrane itself, though this thermally bonded, non woven fabric was only 0.6 mm thickness

(ii) *a strong bond* was at all times evident between the soil and the mebrane saturant. This appears to have contributed to the performance of the MESL, especially at test site 2, where crack growth was successfully inhibited only by the MESL.

(iii) *the improved performance* due to moisture control followed the expectations included in the original designs at test site 1, where deflection variations were reduced in the MESL although performance of all sections was considered adequate.

(iv) the improved performance of the MESL section at site 2 was clearly distinguishable from the controls; and this must be attributed solely to the crack suppression and separation functions of the membrane, since at this site the moisture control function was largely inoperative. It is especially notable that the MESL section tolerates, without any observable distress, what would normally be considered as excessive deflections. We attribute this to the stress-strain properties of the geotextile, and to the soil-membrane bond.

(v) it is noteworthy that no estimate of equivalence could be obtained from these trials, since even the *thinnest* MESL section (100 mm) has continued to perform impeccably for 7 years under traffic.

(vi) the new design principles adopted in these trials appear so far to be justified, and to warrant more investigation

5. ACKNOWLEDGEMENTS

The authors would like to thank Dr. F. Brunner, and Messrs. Porter, Cush and Armstrong, for assistance with the site measurements at various times.

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Outdoor Exposure Tests of Geotextiles

Expériences d'exposition à la lumière de divers géotextiles

Several geotextiles were exposed in 3 different locations. All specimens in a test series were started at the same time and samples withdrawn at 4 to 8 week intervals up to 52 weeks (or failure). Grab strength and elongation of exposed and unexposed controls were measured.

Geotextiles exposed in Florida degraded faster than in Arizona - in most tests. Degradation in North Carolina tended to be slower - but there was considerable variations in this respect between fabrics.

The eleven fabrics tested could be divided into three main groups: the poor performers that must be protected from the sun except for short installation periods; the intermediate performers with only minor strength loss during protracted installation delays and suitable for short life silt fences, temporary walls or very shades locations. The long term performers can be considered for more severe exposure for temporary applications. No geotextile should be left exposed to light for permanent installations.

La résistance et l'allongement de rupture de l'éssai d'arrachement sont mesurés pour les géotextiles exposés à la lumière pour divers intervalles, en trois emplacements de divers climats.

La dégradation est la plus vite en Floride, moins dans l'Arizona et la plus lente dans la Caroline du Nord - dans la plupart des tests. Mais on a constaté des variations sensibles dans les divers géotextiles et même dans les séries de tests.

Les géotextiles testés sont séparés dans trois catégories de rendement: (a) médiocre: les textiles doivent être protégés contre le soleil - excepté pendant une période courte pour l'installation (b) intermédiaire: les textiles qui peuvent demeurer exposés à'la lumière pendant une installation prolongée et sont utilisables pour des usage temporaires (silt fence, mur provisoire etc.) (c) long terme: les textiles qui peuvent être considerérs pour des expositions plus rigoureuses et semi permanentes.

Pour les applications permanentes aucun geotextile ne doit rester exposé à la lumiere indéfiniment.

1 INTRODUCTION

In late 1977, the first samples of geotextiles were sent to Arizona and Florida testing laboratories for outdoor exposure tests. Fadeometer tests had shown poor reproducibility and their relevance to outdoor performance had been frequently questioned.

Correlation between indoor and outdoor tests continues to be hotly debated and it is generally recognized that there are many factors that can and do affect exposure results. Even if indoor tests cannot be relied on for outdoor performance predictions, they can be used for screening or rough comparisons between different fabrics.

2 EXPERIMENTAL DETAILS

2.1 Sample Size and Exposure Angle

The sample size was .3m x .3m and they were mounted at 45° angle south exposure using two samples of each fabric for each exposure time: one with the machine direction (MD) parallel with the horizontal plane, the other with the transverse direction (TD) parallel to that plane. At each exposure time, one sample of each fabric and orientation was withdrawn and cut into four .1m x .15m grab test specimens and tested in the MD direction. The two 50mm x 300mm off-cuts from either side were stored in the dark and retained for additional tests and observations.

Ten specimens of the machine direction (MD) and ten of the transverse direction (TD) were tested without being exposed outdoors. Earlier results had shown that although different strengths may be found in the MD and TD, the percent loss of strength was essentially the same in both directions. All data here reported are the averages for both directions: for exposed specimens and controls.

2.2 Test Locations, Times and Weather Records

Three different locations were used. The Florida (FL) and the Arizona (AZ) locations are commercial firms for exposure testing. In North Carolina (NC) exposure racks were placed on the roof of the Monsanto Research Building and for most fabrics the tests there compared well with the Florida tests, although degradation in N.C. appeared somewhat slower. Not all the fabrics were used in all the test series, but there was considerable duplication and overlap of fabrics and test location. Each test series included a sample of B1 (polyester Bidim® C-22* off the same roll to act as a control to make relative fabric performances independent of variations between different test series. This was a .150 kg/m^2 material. Table I lists a brief description of all the fabrics tested in the various series.

It is known that degradation of the polymers depends on light duration and intensity of relevant wave lengths and such additional factors as temperature, presence of water (liquid or vapor) or other atmospheric components (ozone, nitrous oxides, hydrocarbons, etc.). Further, the strength loss of fabrics depends on the polymer degradation as well as on additional factors such as the fabric structure and the rate at which degraded layers are removed and new layers exposed to the actinic rays.

*Footnote: Bidim® is a registered trademark of Monsanto Company for polyester nonwoven fabrics.

This process can be affected by precipitation (type, total amount or rate of application) and wind (speed, direction and whether combined with rain or dust).

There are several ways in which the energy in the sunlight received by the fabrics can be measured. All stations recorded the total sun hours. Some also used the energy recorded in Langleys - either as total Langleys or as U.V. Langleys. Analysis of our test results did not establish significantly better agreement between different times of the year whether related to sun hours, Langleys or any other reported weather parameter. It was therefore decided to give all our results merely as total times (in weeks) left exposed. This simplifies the charts and does not introduce any important errors.

TABLE 1 FABRIC DESCRIPTION AND GRAB TEST RESULTS OF CONTROLS

Fabric Code	μ* kg/m²	Polymer, Construction	Color †	Grab F_G, kN MD	Grab F_G, kN TD	Grab ϵ_G, % MD	Grab ϵ_G, % TD
B1	.150	PE,C,N,NW	GREY	.658	.560	74	83
B2	.270	PE,C,N,NW	GREY	1.041	1.133	64	67
B6	.160	PE,C,N,NW	WHITE	.534	.455	72	89
CO	.270	PP,C,N,NW	BUFF	1.170	.791	110	158
F2	.360	PP,S,N,NW	WHITE	.734	1.205	129	112
M4	.150	PP,C,B,NW	WHITE	.525	.511	127	124
T4	.135	PP,C,B,NW	GREY	.609	.484	60	69
T6	.200	PP,C,B,NW	GREY	1.112	1.023	58	69
S4	.125	PP,S,B,N,NW	GREY	.436	.502	51	52
S5	.170	PP,S,B,N,NW	GREY	.498	.703	60	67
M5	.140	PP,C,W,SF	BUFF	.823	.925	21	18
A3	.105	PP,C,W,SF	BUFF	.609	.418	21	16
P8	.260	PP,C,W,MF	BLACK	2.063	1.325	26	32
L8	.235	PP,C,W,MF	BLACK	1.965	1.125	27	32
P5	.175	PP,C,W,MF	BLACK	.712	.832	22	25

† Key: B = heat bonded, C = continuous filament, MF = monofilament, N = needled, NW = nonwoven, PE = polyester, PP = polypropylene, S = staple fibers, SF = slit film, W = woven

* Mass per unit area

2.3 Grab Tests: Strength and Elongation

Standard test procedures were used (ASTM D-1682): .15m x .10m specimens, 50mm x 50mm and 25mm x 25mm rubber lined clamp faces, 5mm/sec head speed. The maximum load and elongation (expressed as a percentage of the original, 75mm, gauge length) were recorded and all strip charts retained.

2.4 Microscopic Investigation

Three needled fabrics, i.e. B1, CO (continuous filaments), and F2 (staple) were observed in the scanning electron microscope (SEM) before and after exposure for 12 weeks at the Miami test site. Fiber appearances are shown in Plate I. For that exposure time the percentage strength and elongation loss can be determined from the data plotted in Figs. 1 and 3. B1 and CO both had about 45 to 50% strength retention and 70% retained elongation at break, although the appearance of the polyester fibers (B1) under the SEM was essentially unchanged, while the polypropylene fibers had undergone very significant changes. Comparing CO and F2: the latter had only 10% retained strength and 55% retained elongation at break and yet the appearance of the circumferencial cracks of the former were much longer, deeper, and more numerous. Thus, the more severe cracking of the CO was accompanied by a smaller loss in strength and elongation at break than the very badly degraded, seriously changed F2.

3 RESULTS

3.1 Reproducibility of Tests, Polyester Fabrics

The control B1 fabric was included in all test series and was used to compare repeatability of tests. Figs. 1A & 1B show the grab strengths and elongations retained - expressed as a percentage of the unexposed controls.

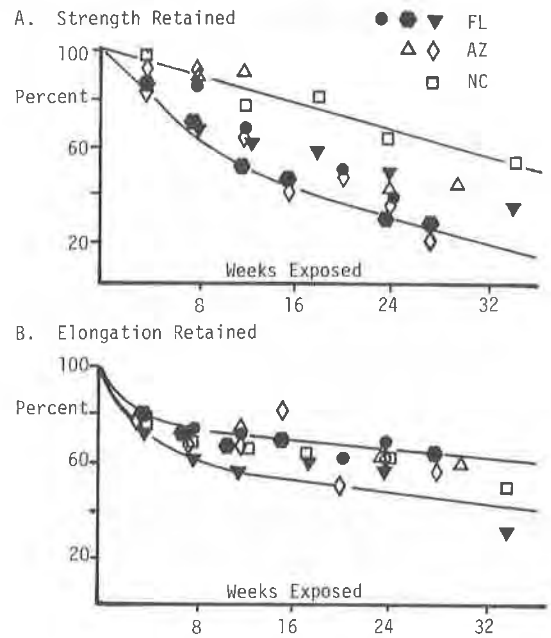


Fig. 1 Grab Strength and Elongation Retained vs. Exposure Time. Polyester Fabric B1; various test sites.

It is clear that considerable differences between different runs were observed. The two Florida test series were similar but differences reported from Arizona for the two test series were much greater: one gave degradations similar to the Miami tests while the other degraded more slowly.

Variations from site to site in the breaking elongation were less than for the grab strength but followed the same pattern. The breaking elongation rapidly decreases in the first 2-4 weeks of exposure and then continues at a much slower rate.

Figs. 1A & 1B show lines (drawn by eye) representing the upper and lower limits of the observations in these tests at different locations and seasons and roughly correspond to North Carolina and Florida rates. All results reported below (when not expressly stated otherwise) will be the actual or expected performance in North Carolina.

3.2 Comparisons Between Different Polyester Fabrics*

Tests were run at the Florida test site in the summer of 1978. The agreement between B1 and B6 was very good and shows that the small amount of carbon black in the grey product had no significant effect on the U.V. resistance. This was in keeping with expectations. Similarly, the better resistance to U.V. of the thicker B2

*Footnote: All polyester fabrics tested were made by essentially the same process.

was as expected and showed a significant effect of the shielding action of the upper layers on the remaining strength of the fabric. The grab test data for the three polyester fabrics are shown in Fig. 2.

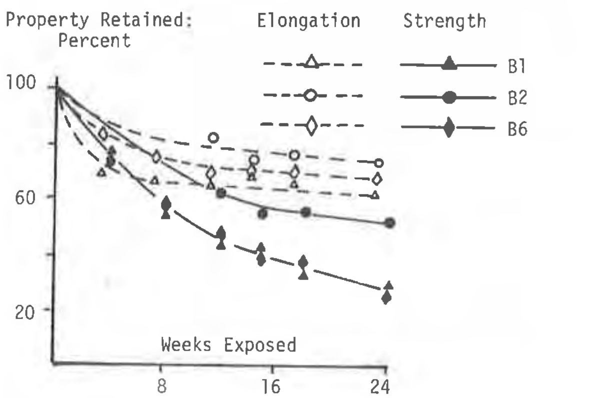


Fig. 2 Grab Strength and Elongation Retained vs. Exposure Time. Needled polyester fabrics (FL).

3.3 Needled Polypropylenes: Fabrics CO and F2

The behavior of these fabrics are shown in Fig. 3 Fabric CO tested in Florida and Arizona gave very similar results although the Florida site was more rapid. The fabric F2 showed slower strength loss in North Carolina and an initial delay period. Both fabrics showed similar losses in elongation with exposure times: At first a rapid decrease to 70% of the original, then a plateau at that level lasting from 4 to 20 weeks, and then a very sudden drop as the strength reaches 10% of the original. This course of degradation is made clearer by the surface changes observed in the microscope and already described in 2.4. Both fabrics appeared dusty (with broken filaments) after a few weeks exposure.

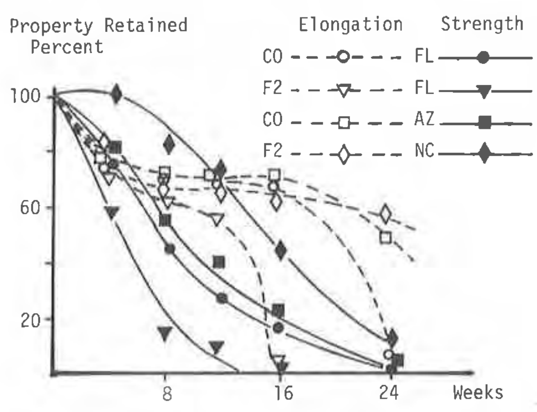


Fig. 3 Grab Strength and Elongation Retained vs. Exposure Time: Needled polypropylene, CO & F2.

3.4 Needled and Bonded Fabrics: P4 and P5

Both these fabrics have a smooth and a fuzzy side, hence tests were run on both faces. The strengths and elongations after exposure did not show significant differences that could be related to which of the surfaces had been exposed.

Fig. 4 shows the very poor light stability of P4 & P5. As had been seen with B1 and F2, the Florida location gave much more rapid deterioration where these fabrics were essentially useless after 3 to 4 weeks and had almost disappeared after 8 weeks. In North Carolina deterioration proceeded at about half the rate but even there the fabrics had lost 70% of the strength and 30% of the elongation after 8 weeks exposure.

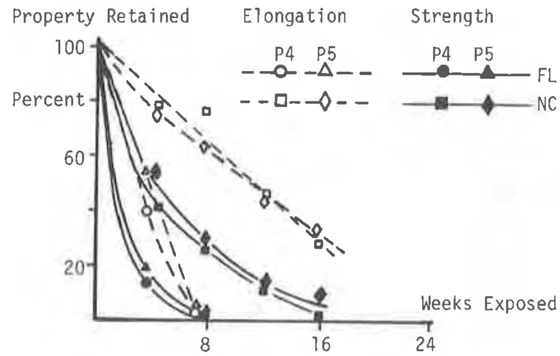


Fig. 4 Grab Strength and Elongation Retained vs. Exposure Time. Needled/bonded polypropylene fabrics.

Fig. 5 shows the results of the bonded polypropylene fabrics. Fabric M4 gave very good reproducibility of fabric strength loss at the Florida and Arizona sites. The material has very poor resistance to sunlight, being essentially useless after 8 weeks, having lost 50% of strength and elongation in only 4 weeks.

Fabrics T4 and T6 were tested in Florida, Arizona and T6 also in N.C. T4 showed minor differences between the Florida and Arizona locations. The initial effect of sunlight was to cause a small increase in strength and little loss in elongation possibly due to cross linking. Subsequent degradation was rapid and the fabric can be considered useless after 14 to 18 weeks.

Fabric T6 showed an approximately linear decrease in grab strength up to about 14 to 18 weeks. Additional exposure caused accelerated deterioration - although this also corresponded with the summer months. As previously noted the Florida location was more severe than the North Carolina site. The fabric was too brittle to handle after 16 weeks in Florida or 30 weeks in North Carolina.

3.6 Woven Geotextiles: Monofil Fabrics P8, L8 and P5

The woven monofil geotextiles are heavy duty fabrics composed of high denier filaments of thickness comparable to the width. They are very stiff, usually black and of very uniform construction. They have been in use in geotechnical applications such as shore protection and drainage for some time. The exposed fabrics, all heavily loaded with carbon black, showed excellent strength retention (Fig. 6A) and only small loss in breaking elongation (Fig. 6B). The fabrics P8 and L8 showed some 20% decrease in grab strength and elongation, while P5 actually increased in strength and elongation but not significantly.

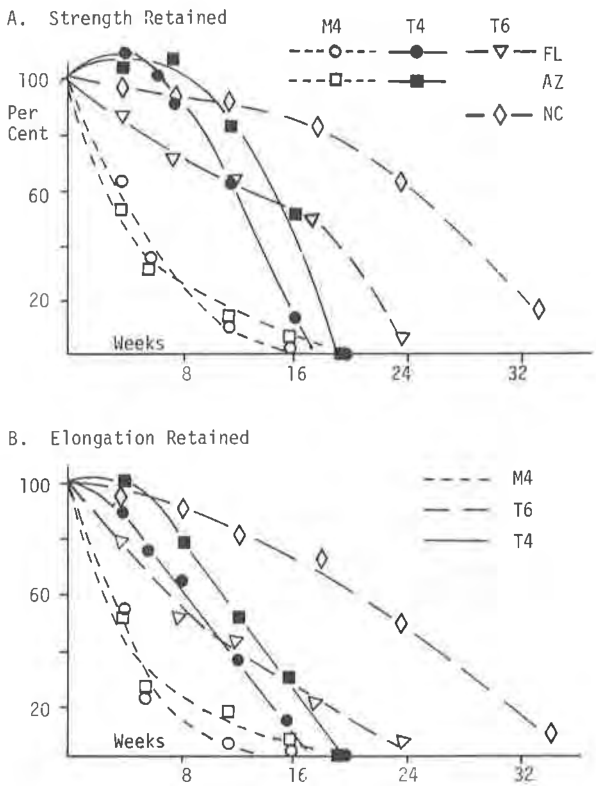


Fig. 5 Grab Strength and Elongation Retained vs. Exposure Time. Bonded Polypropylene Fabrics.

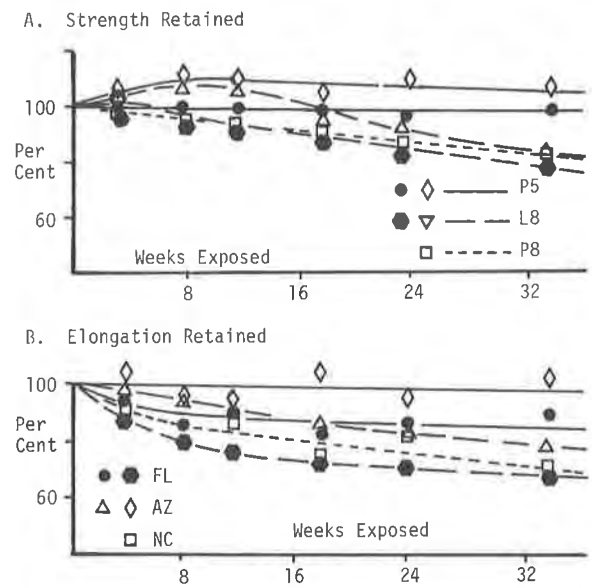


Fig. 6 Grab Strength and Elongation Retained vs. Exposure Time. Woven Popylene Monofils.

3.7 Woven Geotextiles: Slit Film Fabrics M5 and A3

These fabrics are woven from comparatively thin, wide ribbons produced by slitting polypropylene film and folded randomly during weaving. They are typically more flexible and more irregular in appearance due to the uneven folding of the ribbons. They have come into use in the geotextile field more recently and developed from fabrics used for sandbags, bale wraps or carpet backing. Exposure tests on fabric M5 showed good performance in Arizona but only fair in Florida where it started to deteriorate rapidly after 18 weeks exposure (Fig. 7). The other fabric A3 resembled M5 in appearance and construction but performed poorly even in North Carolina. It was too brittle to handle after about 16 weeks.

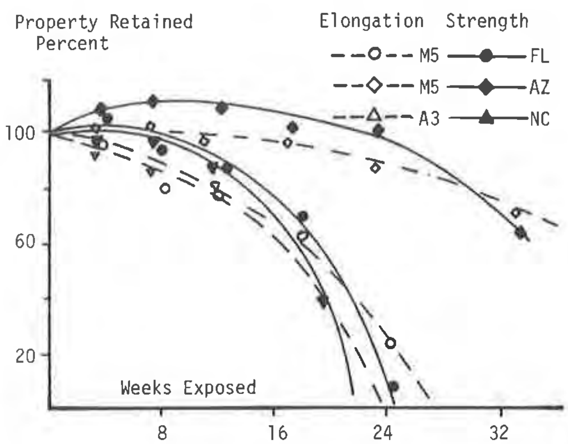


Fig. 7 Grab Strength and Elongation Retained vs. Exposure Time. Woven Slit Film Fabrics.

4 SUMMARY

4.1 Comparative Fabric Performances

Comparisons of fabric performance can be carried out in several ways, namely to compare relative resistance to degradation of different fabrics based on their unexposed properties or the absolute values of strength and breaking elongation. Comparison on a cost basis is also possible but since prices depend on so many factors, such comparisons will be excluded here.

4.1.1 Percent Property Retained

By plotting the relative values, it is easier to show visually the similarities between fabrics, to give better insight into the degradation processes, and to eliminate differences due to fabric weight or directionality of properties. It is also easier to use interpolation and proration of data generated in different locations and test series. Fig. 8 represents actual or expected performance in North Carolina. Those fabrics not tested there had constant multipliers applied to test reresults obtained elsewhere, bearing in mind the time of year, location and the performance of the control B1 polyester fabric. Based on 50 to 70% strength retention, the tested fabrics fall into three main groups: the poor performers, the intermediate performers, and the long-term performers.

1. Poor performers are here exemplified by the polypropylene geotextiles M4, P4 and P5. After 8 weeks of North Carolina exposure, their strength was less than one half the original values. Strength loss started at quite

small exposure times and it is presumed that no inhibitors were present in the polymer.

2. Intermediate performers here include most of the fabrics tested. They retain 80% of their original strength for 8 to 20 weeks, North Carolina exposure, and at least 70% of the original elongation to break. The fabrics included in this group are the polypropylene fabrics of woven slit film, heat bonded, or needled construction. All were too brittle to handle after 24 weeks exposure.

3. Long-term performers are exemplified by the woven monofil polypropylene fabrics L8, P5, P8, and the needled polyester geotextiles. The loss in strength of these was gradual and no sudden accelerated decay was observed up to 52 weeks exposure. They retained at least 50% of the strength and 60% elongation for one year.

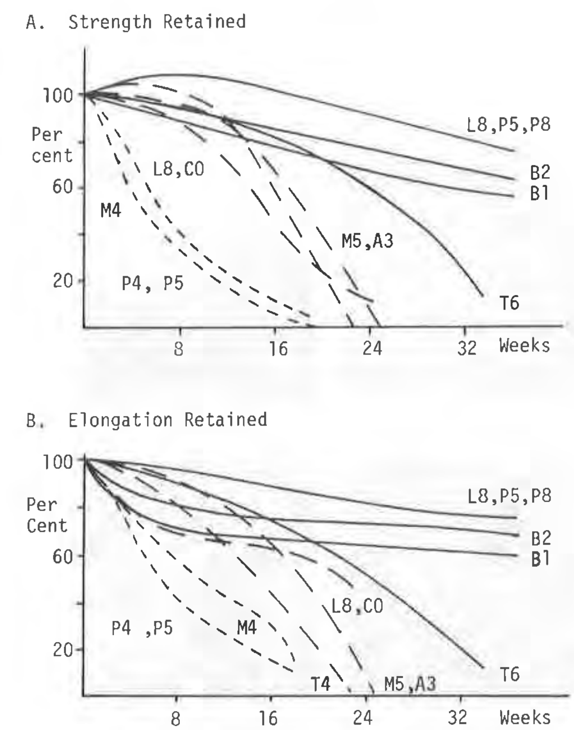


Fig. 8 Grab Strength and Elongation Retained vs. Exposure Times. NC performance.

4.1.2 Absolute Fabric Performance

The actual values of fabric strength and elongation to break are, of course, fundamental in the selection of a suitable geotextile for an engineering project and hence they are critical parameters for performance criteria and specifications.

For the average absolute values*, 3 broad regions can again be distinguished:

1. The poor performers include the same fabrics as before and are normally not suitable for exposure to direct sunlight except for the short periods during installation.

*Footnote: When comparing absolute values, it must be borne in mind that many of the fabrics tested are very anisotropic (see Table I) and this factor will alter rankings for specific applications.

2. The strengths retained for intermediate performers are shown in Fig. 9. They include a heat bonded fabric T4, a woven slit film fabric M5, and the needle bonded fabrics (CO and F2), all made from polypropylene fibers. Their original grab strengths of 0.6 to 1.2 kN are reduced to below 0.5 kN in 12 to 18 weeks exposure. At the same time, the breaking elongations, shown in Fig. 10, originally ranging from 26 to 130% are all reduced to about 20% after 14 to 18 weeks exposure: Exposure to sunlight during installation procedures lasting up to 12 weeks may result in considerable embrittlement and significant strength loss. However, applications such as: silt fences, silt curtains in rivers, temporary fabric walls could be considered.

3. Long-term performers are also shown in Figs. 9 & 10 and they include heatbonded polypropylene fabric T6, needled polyester fabrics B1, B2, and B6, and woven polypropylene monofil fabrics (P8, L8, P5 and P3). The geotextile T6 degraded most rapidly, both in strength and elongation loss. On the basis of grab strength alone, the woven monofil fabrics continued to outperform all other fabrics. However, because of their very low initial breaking elongation, a 30% loss of breaking elongation, means that their total grab elongation at break is 20% or less. For many applications, this would be considered unacceptably low. The needled polyester fabrics with their initial breaking elongation of near 60% are the only fabrics which retain more than 40% breaking elongations after long-term exposure to sunlight.

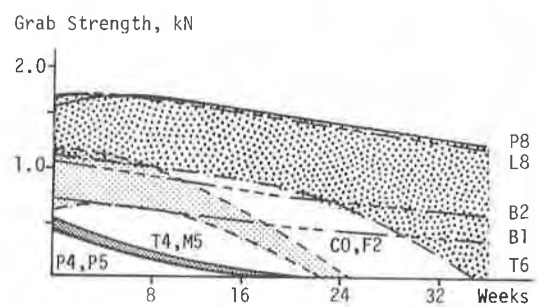


Fig. 9 Average Grab Strength vs. Exposure Time. Poor, Intermediate and Long Term Performers.

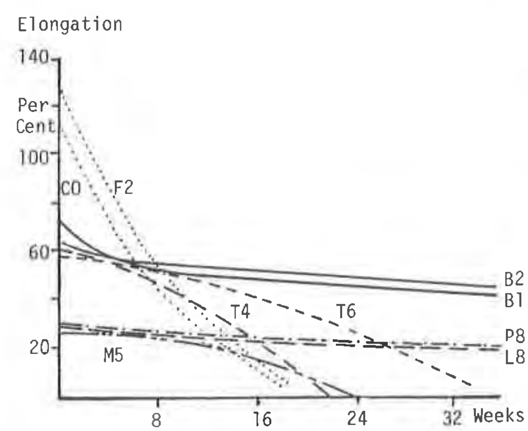


Fig. 10 Average Grab Elongation vs. Exposure Time. Intermediate and Long Term Performers.

Long-term performers can be expected to be suitable for more critical silt fences, silt curtains, and prolonged exposure in shade areas or high latitude climates.

For permanent installation, no geotextile should be left exposed to sunlight.

4.2 Predictions for Actual Fabric Performance

While the comparison of different fabrics is relatively straightforward, predictions of the expected behavior of fabrics in field situations is far from simple. It is clearly impracticable to do long exposure tests in every situation where geotextiles may be used, hence estimates, however approximate, are the only other course.

It is possible to estimate whether the expected degradation will be more or less severe than the data collected in the field tests by combining the information contained in curves such as Figs. 8 or 9 for the fabric in question and local information on:

1. Expected sun hours
   - Local weather data for the appropriate season
   - Reduction due to shading, snow cover, mud
2. Energy of sunlight
   - Fabric configuration and angle of exposure
   - Angle of incidence of sunlight
   - Increase due to altitude or proximity of reflecting areas
3. Additional weather information
   - Frequency of strong winds, sand or rain storms, snow, ice
   - Average temperatures and relative humidity
4. Possibility of atmospheric pollution

While such considerations are useful in general their limitations are exemplified by our own comparisons of data from the Arizona and Florida sites: Although of similar latitude and number of sun hours, the Florida site is normally more severe, but one series in Arizona showed similar degradation. The reason is not clear and may have been due to exceptional weather conditions (sandstorm, hail) or atmospheric pollution.

4.3 Pointers for Specifications

While individual engineers can make allowances for some of the factor outlined above, such an approach is unacceptable for performance criteria or specifications. Hence, the problem is to strike a balance between performance required in severe situations and the additional costs incurred for unneeded performance in the average or minimal situation.

Typically, strength loss with exposure times shows a low rate at low exposure times and a much more rapid rate after a given time interval. If the specification is set too low, a higher than expected exposure would produce severe strength loss in the fabric; if set too high, it may eliminate many fabrics that would perform well in the vast majority of situations. A uniform rate of degradation as shown by the polyester fabrics greatly reduces the risk of a serious failure due to light degradation, if the design otherwise follows sound practice.

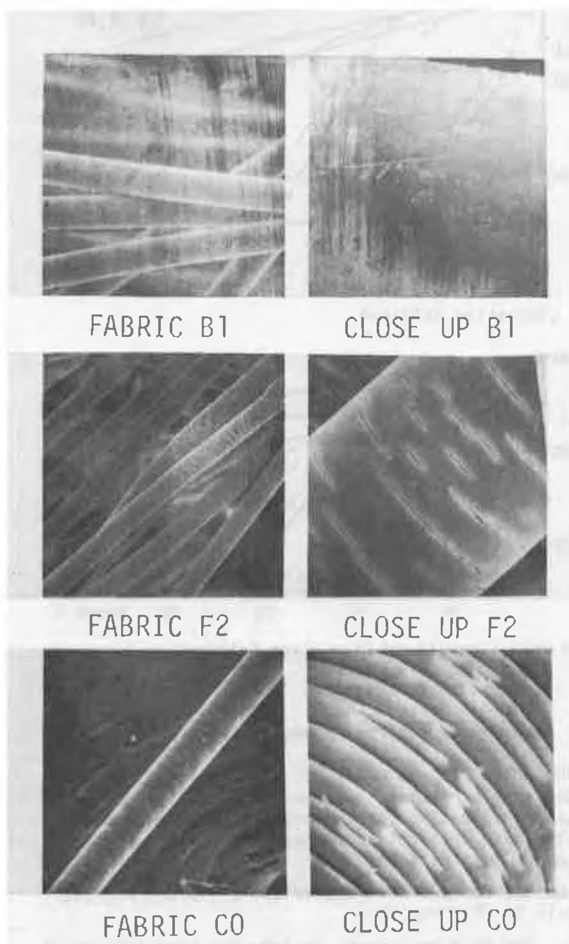
At this stage, few specifications address the subject of sunlight exposure. One Department of Transportation requires a grab strength retention of 90% after 30 days' exposure and is obviously aimed at eliminating only the very poor performers, which might represent a risk if construction is slow or interrupted.

Specifications for projects requiring longer exposure will be much more difficult to set. The choice of grab testing was based on convenience and is only one type of measurement and may not be appropriate for specific applications.

5 CONCLUSIONS

Although laboratory testing of U.V. and light resistance of geotextiles can be precisely specified and controlled, it is at least equally important to measure actual outdoor performance. The data presented here give a clear indication of the wide variations observed - not only between fabrics but between test locations and repeat exposure tests. Therefore great caution must be used when predicting geotextile performance from limited outdoor or indoor tests. Additional parallel exposure testing and laboratory U.V. tests are needed to arrive at useful standard test methods and specifications for the diverse applications of geotextiles.

Plate I SEM Views of Fabrics B1, F2 and CO Exposed for 12 Weeks (Florida).



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Methods Used for Testing the Bio-Colmatation and—Degradation of Geotextiles Manufactured in Romania

Méthodes utilisées pour la détermination du biocolmatage et de la dégradation des géotextiles fabriquées en Roumanie

Four polypropylene fabrics, a polyester textile and a material prepared from residues of different synthetic polymeric textiles were studied.More than 1400 geotextile samples were incubated in 8 media (distilled water,media for culturing iron bacteria,desulfovibrios and levansynthesizing bacteria, liquid mineral medium, sea water,compost and soil) for 5-17 months. The results indicated that some colmatation occurred in the geotextiles incubated in cultures of iron bacteria,desulfovibrios and levansythesizing bacteria. But this bio-colmatation should be a slow phenomenon,not affecting the filtering-draining capacity of geotextiles,as no important modifications appeared in their permeability. The geotextiles are not toxic for the aquatic and soil microorganisms. Some water soluble substances from geotextiles can even promote the development of microorganisms. However,none of the geotextiles showed any sign of biodegradation. Tensile strength and infrared spectroscopy also indicated that the mechanical properties and structure of geotextiles remained unchanged.

Les auteurs ont étudié quatre types de géotextiles non-tissés (deux en polypropylène, un en polyester et un d'un mélange des polymères synthétiques) et deux tissus en polypropylène. Plus de 1400 échantillons ont été incubés dans huit milieux (eau distillée, milieu pour la cultivation des ferrobactéries,désulfovibrions et bactéries synthétisant le lévane,milieu minéral liquide,eau de mer, compost et sol) à une durée de 5-17 mois. Les résultats obtenus, ont montré qu'il y a un certain biocolmatage apparu dans les géotextilex incubées dans les cultures des ferrobactéries, des désulfovibrions et des bactéries synthétisant le lévane.Mais ce biocolmatage est un phénomène lent,qui n'affecte pas la capacité filtrante-drainante des géotextiles parce qu'on ne se produit pas des modofications importantes dans leur perméabilité. Les géotextiles ne sont pas toxiques pour les microorganismes aquatiques et terricoles.Certaines substances hydrosolubles des géotextiles peuvent même stimuler le développement des bacteries. D'ailleurs,aucune des géotextiles n'a montré des signes de biodégradation.Les propriétés mécaniques et la structure des géotextiles ont resté les mêmes.

INTRODUCTION

Microorganisms can damage the geotextiles by colmatation and degradation. Both inorganic and organic compounds produced by microorganisms can cause the colmatation of geotextiles. Biodegradation of geotextiles like that of other synthetic polymers depends,first of all,on their chemical structure. For understanding the recalcitrance (1) of the polyalkene-type geotextiles to biodegradation,the findings by Albertsson et al.(2-4) are of major importance.Working with carbon-14 labeled polyethylene films and powders they have shown that only the carbon of the low molecular weight fractions is converted to $^{14}CO_2$; the polymer is resistant to biodegradation.According to literature data reviewed for example by Higgins and Burns (5),evidence has been obtained for the biodegrability of many polyesters,but in most cases the rates of biodegradation are extremely low,being measured in months or years rather than days. This is presumably true also for the polyester-type geotextiles.

In this paper the results of testing six types of geotextiles,manufactured in Romania are described for their microbial colmatability and degradability under laboratory conditions. The studied geotextiles comprised four polypropylene fabrics,one polyester and a material prepared from residues of different synthetic polymeric textiles. The used methods made possible the observation of geotextiles colmatation by microbially produced ferric hydroxide,ferrous sulfide and polysaccharide (levan) as well as the microbial metabolization and co-metabolization of biodegradable materials.

MATERIALS AND METHODS

Tested Geotextiles.

The six types of tested geotextiles comprised both nonwoven and woven fabrics also differing in respect of their functionality and hydraulic properties.

MADRIL$^{(R)}$M is a nonwoven polypropylene fabric;its fibers have a fineness of 0.66 tex and a length of 60 mm and are mechanically bonded by needlepunching.

MADRIL$^{(R)}$V differs from MADRIL$^{(R)}$M only by fineness (1.9 tex) of its fibers and length (100 mm).

MADRIL$^{(R)}$P is a nonwoven polyester textile; its fibers of 0.44 tex fineness and of 60 mm length are mechanically bonded by needlepunching.

TERRASIN is the nonwoven prepared from residues of different synthetic polymeric textiles.The fineness of its fibers varies between 0.4 and 2.5 tex and the length between 30 and 60 mm.The fibers of Terrasin are bonded mechanically by needlepunching and chemically by Romacryl (an acrylate based binding agent manufactured in Romania).

ALPHA M is a woven polypropylene fabric with fibrillated yarns whose fineness is 229 tex (warp) and 268 tex (filling); its denseness is of 58 yarns/10 cm (warp) and 86 yarns/10 cm (filling).

ALPHA G, like Alpha M, is a woven polypropylene textile;its yarns are fibrillated in warp (fineness 267 tex) and multifilamentary in filling (fineness 297tex). The denseness of Alpha G is of 109 yarns/10 cm in warp and of 108 yarns/10 cm in filling.

Incubation Media

The geotextile samples were incubated in 8 media.

1.Distilled water served as control medium (D.W.).

2.For culturing iron bacteria the nutrient medium recommended by Fedorov (6) was used. This medium is prepared from fallen leaves which are extracted with hot tap water. The dark-colored extract is filtered, then diluted with water to render its color pale yellow.Finally, iron chippings are added to the diluted extract (5 g iron/1000 mL extract); pH=6.5 (M.I.B.).

3.Desulfovibrios were cultured in a variant of the van Delden medium (Allen (7)) consisting of 2 g peptone, 1 g K_2HPO_4, 1.5 g $MgSO_4 . 7 H_2O$, 5 g Na lactate,0.25 g $FeSO_4 . 7 H_2O$ in 1000 mL tap water; pH= 7.(M.D.).

4.For the cultivation of levan-synthesizing bacteria the method of Kiss and Drăgan-Bularda (8) was applied but the synthetic basal medium was replaced by beef extract. Composition of the medium: beef extract (Difco) 3 g,sucrose 100 g and tap water 1000 mL; pH = 7 (M.L.B.).

5.The liquid mineral medium (Schlegel (9))used was prepared from 1 g NH_4Cl, 0.5 g K_2HPO_4, 0.2 g $MgSO_4 .7H_2O$, 0.01 g $CaCl_2$, 1 mL Hoagland solution of microelements and 1000 mL tap water; pH = 7 (L.M.M.).

6.Water collected from the Black Sea (S.W.).

7.The compost applied was obtained from different plant residues in the Botanical Garden of Cluj-Napoca (C).

8.The used soil is a fertile alluvial soil from the vicinity of Cluj-Napoca (A.S.).

Experimental Variants.

Five discs and three strips (from Madril M,V,P or Terrasin) or three squares and three strips (from Alpha M or G) were put in a 2 L glass jar containing one of the six liquid media or introduced into a 10 L Mitscherlich vessel filled with compost or soil.Thus, 352 individual geotextile samples were necessary for a testing period. As the experiment comprised 4 testing periods,a total of 1408 samples were used.

The distilled and sea water in which geotextile samples were immersed,and the compost and soil in which similar samples were placed were not inoculated with microorganisms. In other words,the geotextile samples were submitted only to the action of the spontaneous microflora present in these four media.

The media for iron and levan-synthesizing bacteria and the liquid mineral medium, after immersion of geotextile samples,were inoculated with mixed populations of microorganisms from compost,alluvial soil and mud (the mud was collected from the Gheorghieni lake in Cluj-Napoca). The geotextile samples immersed in the medium of levan-synthesizing bacteria were inoculated also with the cell suspension of a Bacillus species very active in synthesis of levan. The geotextile samples introduced into the medium of desulfovibrios were inoculated only with lake mud. The experimental variants on liquid media were incubated at room temperature,while those with compost and soil,in the open air.

During incubation macro- and microscopic examinations and qualitative chemical analyses were carried out. The presence of Fe^{2+} and Fe^{3+} in the culture of iron bacteria and that of free H_2S in the culture of desulfovibrios was checked. The levan formed was analyzed by means of paper chromatography (8). The water evaporated during incubation was replaced by distilled water.

The incubation periods lasted 5, 8, 12 and 17 months.

In the end of the first incubation period the geotextile samples of this period were removed from the liquid media,compost and soil,then dried at 60°C for 24 hours and finally examined to determine their permeability and tensile strength. The geotextile samples of the other incubation periods were transferred to fresh media and inoculated the same as in the beginning of the experiment. This procedure was repeated in the end of the next incubation periods,excepting the last one (the 4th). Of course, the geotextile samples incubated for 8,12 and 17 months were also submitted to permeability and tensile strength determinations.

Determination of geotextile samples normal permeability (k_n) was carried out :

a) by the Bourdillon method (10),applied on a set of 5 discs of nonwoven fabrics, and

b) by means of a device (figure 1) for measuring the apparent filtration velocity on single samples of both nonwoven and woven geotextiles.

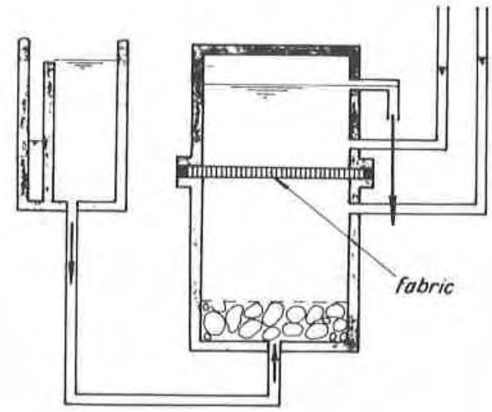


Fig.1. Device for measuring the apparent filtration velocity

For the determination of tensile strength the French standard method (11) was applied.

The geotextile samples incubated for 17months were examined also by infrared spectroscopy,using a SPECORD 75 IR type apparatus and working in the wave number ($\bar{\nu}$) range of 400-2200 cm^{-1}.

RESULTS

A. Macro- and Microscopic Examinations and Qualitative Chemical Analyses.

The results will be presented in the incubation media and the geotextile types description order.

1.Distilled water. In the distilled water in which Madril M samples were immersed no spontaneous microflora developed during the first 12 months of incubation. But by the end of the last incubation period (17 months) a scarce development of bacteria was observed. Similar observations were made in the case of Madril V, too.During the first and last incubation periods bacteria were found, although in small number, in the distilled water in which Madril P samples were placed. In the case of Terrasin samples numerous bacteria developed in the water during the whole incubation period. In addition ,transparent sediments appeared on the surface of Terrasin samples after 8 months of incubation. The sediments turned blackish by the end of 17 month incubation period.Excepting the first 5 months of incubation, a slight development of bacteria always took place in the distilled water with Alpha M and Alpha G samples.

Development of bacteria in the distilled water in which geotextile samples were incubated can be explained by solubilization of some substances serving as nutrients for bacteria. In the case of Madril P samples, the initial solubilization occurring in the first incubation period was followed by an other solubilization surprisingly taking place in the last incubation period. However, the solubilization did not affect the basic structure of geotextiles as none of the tested geotextile types showed any visible degradation during their 17-month incubation in distilled water.

2. Medium for iron bacteria. Abundant growth of filamentous iron bacteria occurred in this medium in each incubation period and in the presence of each geotextile type. Ferrous and ferric ions were constantly found in the culture liquid. Brown and black sediments covered step by step the surface of geotextiles. Figure 2 is the microscopic image of these sediments on the fibers of a Madril V sample kept in culture of iron bacteria for 12 months. Formation of sediments was not, however, associated with any visible degradation of the geotextiles.

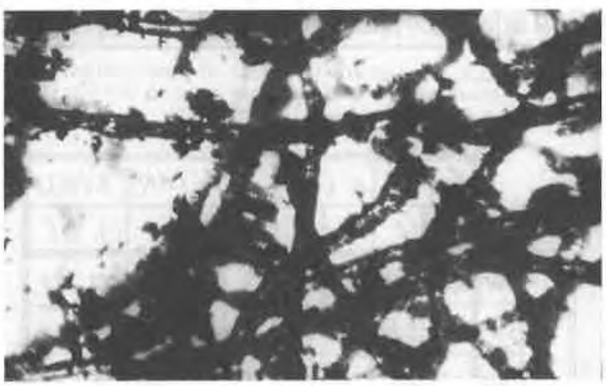


Fig.2. Sediments on geotextile fibers incubated in culture of iron bacteria

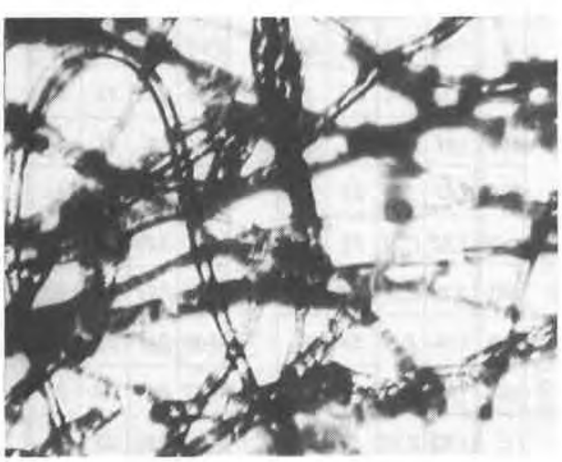


Fig.3. Sediments on geotextile fibers incubated in culture of desulfovibrios

3. Medium for desulfovibrios. These bacteria always developed in the presence of each geotextile type. The reaction for free H_2S in the culture liquid was intensely positive for 8 months but the formation of FeS took place during the whole incubation period. FeS sedimented on the geotextile fibers and yarns in form of black patches. Figure 3 shows the sediments on Madril V fibers after their 17-month incubation in culture of desulfovibrios. The desulfovibrios, like the iron bacteria, caused sediment formation, but they did not bring about any visible degradation of the geotextiles.

4. Medium for levan-synthesizing bacteria. Besides the abundant development of bacteria, a scarce growth of microfungi also accurred in each incubation period and in the presence of each geotextile type. The culture liquid constantly contained free levan during the first 8 months of incubation (figure 4). Later, free levan was not detected in the culture liquid. Instead, the sedimentation of a viscous brown material on the surface of fibers and yarns intensified. A picture of the sediments appearing on Madril M fibers following their 12-month incubation in culture of levan-synthesizing bacteria is presented in figure 5. The sediment formation was not accompanied, in this medium either, by visible decomposition of the tested geotextile types.

5. Liquid mineral medium. Both bacteria and microfungi constantly developed in this medium in the presence of each geotextile type. Their development should be attributed to some organic substances solubilized from the geotextile samples. However, there is no sign of degradation of any of the tested geotextile types.

6. Sea water. In each incubation period scarce development of bacteria occurred in sea water without any observable changes in the tested geotextile types.

7. Compost. During incubation, the pores of the geotextile samples were gradually blocked with fine compost particles. At the same time, some samples were perforated by the roots of weeds growing in the vessels. Nevertheless the geotextile basic structure remained unchanged even in the end of the 17-month incubation period.

8. Alluvial soil. The observations made are similar to those described above in the case of compost.

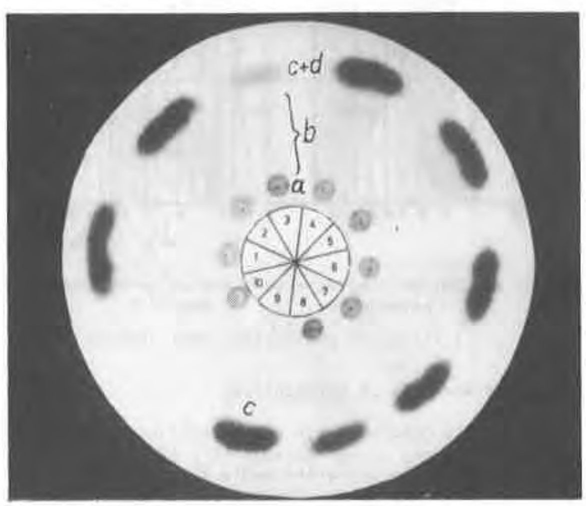
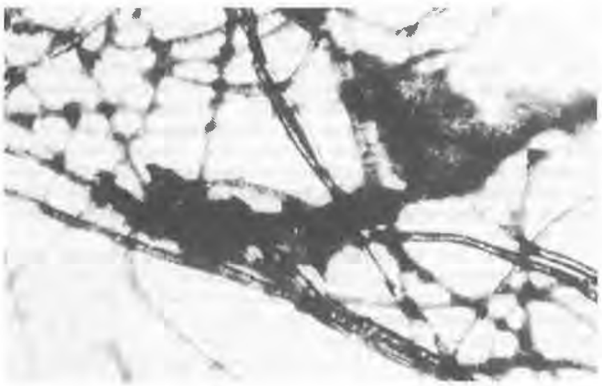


Fig.4. Bacterial levan formation in presence of geotextiles. 1-4: Cultures with Madril M samples.
5-8: Cultures with Madril V samples.
9: Sucrose solution. 10: Levan solution.
a: Levan. b: Oligofructosides. c: Sucrose.
d: Fructose.



Fig,5. Sediments on geotextile fibers incubated in culture of levan-synthesizing bacteria

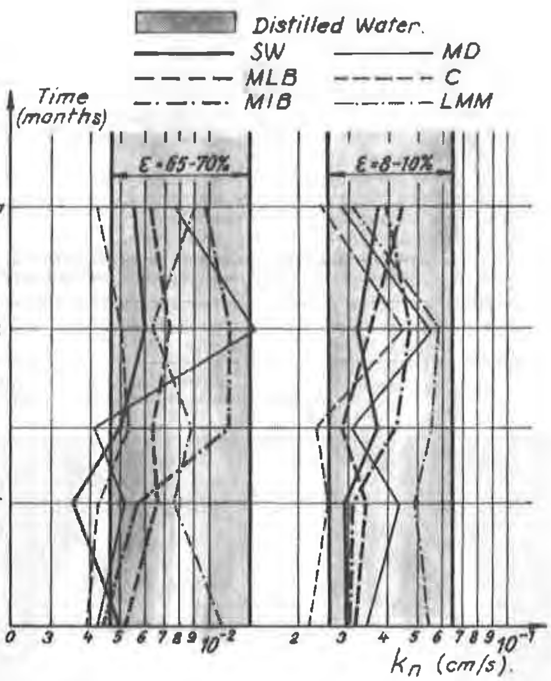


Fig.6. Incubation time-dependent variation of normal permeability (k_n) of Madril V

: Strain of geotextile under loading

B. Determination of Permeability

The results obtained with the Bourdillon method (10) have shown that the normal permeability of the geotextile samples incubated in different media for 5-17 months varies only within the ranges found for the nonincubated and distilled-water-incubated samples. These findings are exemplified in figure 6 in the case of Madril V. Similar results were obtained when the apparent filtration velocity was measured. The mean values registered with geotextile samples incubated in the six liquid media for 17 months, and the minimum and maximum values found with geotextile samples kept in compost and alluvial soil also for 17 months are presented in table 1. One can see from this table that no important changes occurred in the filtration velocity in the case of geotextile samples incubated in the liquid media. The minimum values were found in geotextile samples which were not perforated by roots of weeds growing in compost or alluvial soil, while the perforated samples gave the maximum values.

Table 1. Apparent velocity of filtration (cm/s)

	INCUBATION MEDIA									
	DW	MLB	MD	SW	MIB	LMM	C		AS	
							min.	max.	min.	max.
MADRIL® M	0,23	0,19	0,24	0,24	0,24	0,23	0,06	0,2	0,12	0,27
MADRIL® V	0,25	0,20	0,25	0,28	0,26	0,25	0,14	0,19	0,09	0,26
MADRIL® P	0,23	0,19	0,23	0,25	0,23	0,20	0,19	0,28	0,24	0,3
TERRASIN	0,25	0,23	0,24	0,25	0,25	0,22	0,11	0,24	0,19	0,28
ALPHA M_1	0,45	0,37	0,41	0,45	0,44	0,42	0,42		0,43	
ALPHA G_2	0,36	0,37	0,35	0,37	0,41	0,39	0,38		0,34	

Table 2. Mechanical characteristics of some geotextiles incubated in different media for 5 months (i) and 17 months (f)

		F_G (KN)		ε (%)			F_G (KN)		ε (%)	
		i	f	i	f		i	f	i	f
MADRIL® M	DW	2,51	2,47	92	90	TERRASIN	0,67	0,75	73	75
	SW	2,49	2,38	91	88		0,85	0,94	56	63
	LMM	2,55	2,47	94	89		0,77	0,82	55	50
	MIB	2,49	2,35	77	82		0,89	0,8	53	55
	MD	2,54	2,45	80	75		0,91	0,96	53	58
	MLB	2,52	2,49	90	90		0,89	0,87	55	57
	C	2,38	2,41	88	90		0,89	0,98	49	64
	AS	2,37	2,43	90	86		0,76	0,93	52	53
MADRIL® P	DW	1,81	2,21	62	63	ALPHA M	0,89	0,88	21	26
	SW	1,93	2,1	62	58		0,82	0,92	22	24
	LMM	2,22	2,10	59	59		0,91	0,89	22	24
	MIB	2,10	2,18	56	57		0,99	0,93	26	25
	MD	1,74	2,07	55	54		0,93	0,81	22	23
	MLB	1,98	1,96	63	66		0,83	0,81	23	23
	C	1,92	2,26	59	52		0,03	0,91	26	24
	AS	1,89	2,07	54	53		0,72	0,76	22	23

Since these determinations did not reveal any important modifications in permeability, the bio-colmatation macro- and microscopically observed should be a very slow phenomenon which does not affect the filtering-draining capacity of the geotextiles.

C. Determination of Tensile Strength

Some of the values of tensile strength and elongation of geotextiles incubated for 5 and 17 months are given in table 2. They prove that the microorganisms developed in media with geotextile samples did not cause any modifications in these mechanical properties of geotextiles. This finding is valid for each of the tested geotextile types. The small variations are undoubtedly due to the anisotropy characteristic for fabrics.

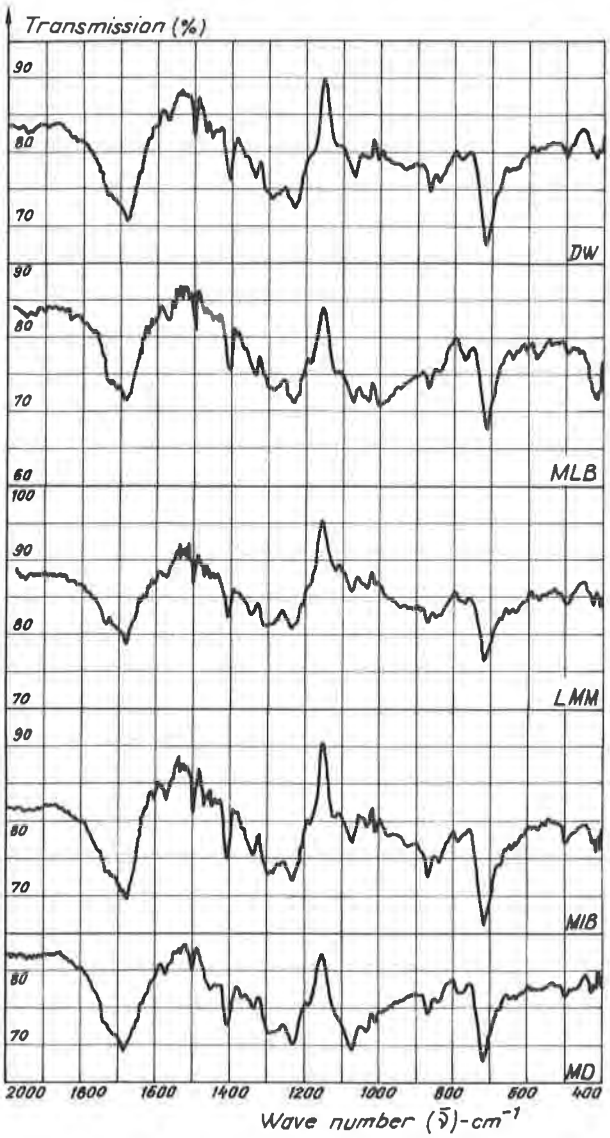


Fig.7. Spectrograms of Madril P fibers incubated in different liquid media

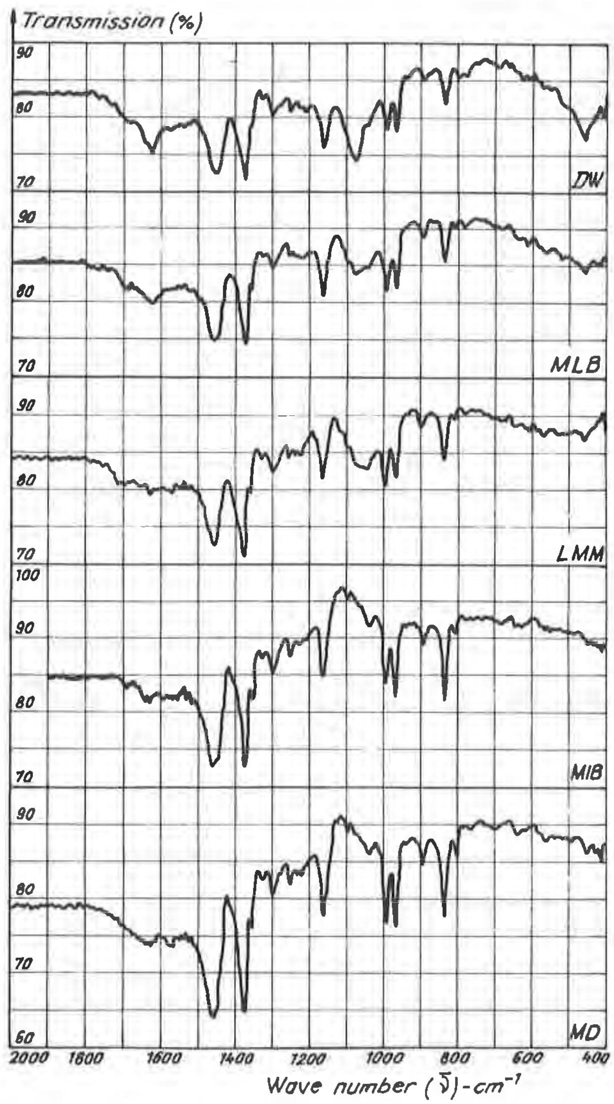


Fig.8. Spectrograms of Madril V fibers incubated in different liquid media

D. Infrared Spectroscopy

The results recorded by infrared spectroscopy indicate that the fibers of the tested geotextiles did not undergo any structural modifications, as the peaks on spectrograms of geotextile samples remained the same in all incubation media. The spectrograms registered with fibers of Madril P and V and Terrasin following their 17-month incubation are shown in figures 7-9.

CONCLUSIONS

1. Some colmatation occurred in the six geotextile types tested when they were incubated in cultures of iron bacteria, desulfovibrios and levan-synthesizing bacteria for 5-17 months. But the bio-colmatation should be a slow phenomenon, not affecting the filtering-draining capacity of geotextiles, as no important modifications appeared in their permeability.

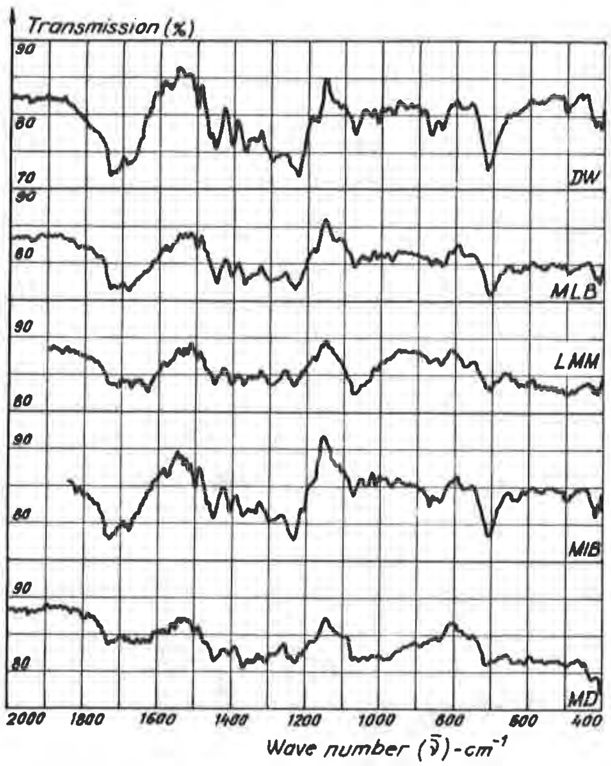


Fig.9. Spectrograms of Terrasin fibers incubated in different liquid media

2, The geotextiles are not toxic for the populations of aquatic and soil microorganisms.Some water-soluble substances from geotextiles can even promote the development of microorganisms. However,none of the six tested geotextile types showed any sign of biodegradation following their 5-17-month incubation in 8 media (distilled water;media for culturing iron bacteria,desulfovibrios and levan-sythesizing bacteria;liquid mineral medium;sea water;compost;alluvial soil). Tensile strength determinations and infrared spectrograms showed that the mechanical properties and the structure of geotextiles remained unchanged.

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Some Answer's Components on Durability Problem of Geotextiles

Quelques eléments de réponse au problème de la durabilité des géotextiles

To answer the questions about ageing and durability of geotextiles a large inquiry has been hold in France to with-draw samples of geotextile from civil engineering works in which they have been put for more twelve years. The samples concern as well needle spun non woven as woven geotextiles. Hydraulical and mechanical properties have been studied and also the main structural features of the fibrous polymer. Hydraulical characteristics will not be described here.

It has been ascertained that changes in mechanical properties of geotextiles and fibers they are made, for non-woven geotextiles, remain generally less than 30 %, in relation with those of standard samples. It is possible to account for these changes in fonction of the assumed "work" done by the geotextile during the first months of the building. For peculiar reasons (light exposure, punctures) geotextiles can lose more than 30 % of their mechanical properties. Woven geotextiles we examined were damaged by punctures essentially.

Une vaste campagne de prélèvements d'échantillons de géotextiles a été entreprise en France dans plus de 30 ouvrages (géotextiles tissé et nontissé). Les propriétés hydrauliques et mécaniques résiduelles des géotextiles ont été étudiées ainsi que les modifications de la structure du polymère fibreux. Les propriétés hydrauliques ne sont pas rapportées ici. Il est constaté que pour la majorité des nontissés examinés, les pertes de caractéristiques mécaniques n'excèdent pas 30 %. Ces pertes se produiraient dans les premiers mois après la mise en oeuvre et sont attribuables aux contraintes mécaniques supportées. Des raisons particulières (lumière, perforations) peuvent conduire à des pertes supérieures à 30 %. Les géotissus étudiés sont surtout endommagés par des perforations.

Dans de bonnes conditions de mise en oeuvre, et dans des utilisations conventionnelles, les géotextiles devraient pouvoir durer plus de 100 ans.

INTRODUCTION

Ainsi que nous le rappelons dans un autre article (1), outre les études de laboratoire qui consistent à soumettre les géotextiles à des tests de vieillissement accéléré afin d'apprécier leur résistance vis-à-vis de contraintes simples ou de combinaisons de contraintes et éventuellement de fournir des indications sur leur durabilité, il existe une autre démarche, plus empirique peut-être, qui consiste à essayer de retrouver les géotextiles là où les ingénieurs les avaient placés il y a 2, 5, 15 ans ou plus, pour voir s'ils ont vieilli !...

Evidemment, les moyens à mettre en oeuvre dans les deux démarches sont très différents. Nous décrirons dans ce texte, la manière dont les équipes du Comité Français des Géotextiles (CFG) ont conduit leur campagne de prélèvements et les résultats des études de laboratoire réalisées sur les échantillons de géotextiles récupérés. Ce travail a bénéficié d'une subvention du Ministère de l'Industrie français.

1 CAMPAGNE DE PRELEVEMENTS DES GEOTEXTILES

1.1 Enquête :

En vue d'entreprendre cette campagne de prélèvement de géotextiles, dans les ouvrages de génie civil où ils étaient placés, une vaste enquête préalable a été lancée par les laboratoires des Ponts et Chaussées afin de connaître :

- les modes d'utilisation des géotextiles les plus répandus en France ;
- le comportement des produits utilisés ;
- la possibilité de faire des prélèvements de géotextiles dans l'ouvrage pour apprécier l'éventuelle modification de leurs caractéristiques.

Cette enquête a fourni en Janvier 1979 189 fiches réponses dont l'analyse donne les résultats suivants en ce qui concerne la répartition des géotextiles (Table 1):

Table 1. Répartition des Géotextiles en France

Géotextiles Non-Tissés		Géotextiles Tissés	
Désignation	Nombre	Désignation	Nombre
BIDIM	138	ADITEX	19
SODOCA	7	GRILTEX	2
TERRAM	22	STABILROAD	1
POLYFELT	3	SCOTTLAY	1
		STABILENKA	2
	---	VIATEX	8
	170		33

Plus précisément le Table 2 fournit le nombre de prélèvements possibles, par nature d'ouvrage et par type de géotextile.

Table 2. Classement des géotextiles par type d'ouvrage

Désignation	Sous Remblai	Sous Couche de forme	Anticontaminant Sol/Chaussée	Filtre	Autres	Total
BIDIM	17	14	16	8	6	61
SODOCA	1	2	-	1	-	4
TERRAM	-	-	5	3	1	9
ADITEX	2	6	-	-	-	8
STABILENKA	-	-	-	-	1	1
SCOTTLAY	-	1	-	-	-	1

Ce premier ensemble constitue un échantillonnage fort intéressant dans lequel une série de 25 prélèvements a été sélectionnée par ordre d'ancienneté :

BIDIM qui apparaît en 1970 dans ces ouvrages
SODOCA -"- en 1972 " " "
ADITEX -"- en 1973 " " "
STABILENKA

Dans cette enquête, le TERRAM n'apparaît qu'en 1975.

D'autres échantillons ont été prélevés par les soins du CEMAGREF, notamment dans le barrage de MAUREPAS (SODOCA sur parement amont) et dans la piste de chantier du marais de REDON (BIDIM).

1.2 Mode de prélèvement des géotextiles

Ces prélèvements de géotextiles ont été faits en notant un nombre minimum d'indications et suivant des modalités communes de manière à ce que l'étude d'ensemble puisse revêtir un caractère suffisamment général.

Les Laboratoires des Ponts & Chaussées ont effectué en plus des opérations de prélèvement proprement dîtes, des actions permettant de décrire de manière détaillée l'environnement du géotextile :

. description de l'ouvrage et étude géotechnique de l'environnement immédiat du textile (granulométrie, limites d'Atteberg, teneur en matières organiques, etc), plans, photos, croquis;
. étude hydrologique : niveau de la nappe phréatique, pH;
. prélèvement du textile : excavation conduite avec soin de manière à éviter l'endommagement du textile, pour cela le terrassement à l'aide d'engin mécanique est arrêté à quelques décimètres au-dessus du textile. Le dégagement d'une surface de 1 m x 1 m de géotextile est ensuite conduit à la main (truelle, main de fer). Le géotextile prélevé est placé à plat entre 2 feuilles de polyane, qui sont scellées de manière à éviter toute dessication du textile avant les contrôles;
. prélèvement d'un complexe sol-textile intact : par carottage. L'étude de ces carottages font l'objet d'un autre article (2).

Sur les géotextiles ainsi prélevés, une campagne d'essais a été entreprise afin de déterminer :

. le taux de contamination (lavage intensif dans des conditions précises);
. le coefficient de perméabilité avant et après lavage;
. épaisseur du géotextile avant et après lavage;
. résistance et déformabilité par essai de traction sur manchon (test ST BRIEUC);
. résistance à la traction (test CFG, un essai-test AFNOR 5 essais);
. appréciations des modifications du polymère fibreux : dynamométrie sur fibres élémentaires, taux de cristallinité, orientation moléculaire, taux de COOH, indice de viscosité limite...).

2. RESULTATS

Il ne sera pas possible de faire apparaître ici le détail de tous les résultats, seuls les plus significatifs seront rapportés.

Un point essentiel est à considérer dans ces études sur le vieillissement, il s'agit du problème des échantillons témoin. Il est évident que les utilisateurs de géotextiles dans les années passées ne se sont pas préoccupés de ce point et n'ont pas conservé par devers eux d'échantillon de référence témoin. Des tentatives ont donc été faites pour trouver soit des échantillons "témoins" contemporains des géotextiles mis en oeuvre dans les ouvrages (échantillons conservés dans des conditions à peu près précisées), soit des données des producteurs sur les échantillons produits à l'époque : les deux valeurs n'ayant d'ailleurs pas la même signification. Par chance, dans beaucoup de cas, ces valeurs de référence ont été trouvées et ont donc permis d'apprécier l'évolution des géotextiles en polyester et polypropylène après plusieurs années d'utilisation.

Nous examinerons successivement le cas des géotextiles en polypropylène et celui des géotextiles en polyester.

2.1 Géotextiles en polypropylène

2.1.1 Géotextiles nontissés aiguilletés

Cas de plusieurs échantillons de SODOCA AS 420 prélevés en 1979 et 1980 :

a) sur la déviation de NOYALO (RN 780) réalisée en 1972; le géotextile assure une fonction d'anticontaminant entre le sol naturel (argile limoneuse) et le sol de recouvrement constitué successivement d'un sol 20/40 propre et non pollué (30 cm), d'une matière granitique (120 cm), et d'une arène granitique traitée au ciment (35 cm). Le géotextile prélevé ne présente aucune déchirure ni perforation. Après lavage, on observe néanmoins une forte contamination du géotextile par la végétation et le sol.

b) sur la déviation d'HYERES (Var) réalisée en 1976. Déblai sous limon, près de la nappe phréatique. Le géotextile assure un rôle anticontaminant de la couche de forme (grave en calcaire 50 $<$ D $<$ 80 mm, avec 9 % $<$ 80 microns) et un rôle de filtre dans les massifs drainants latéraux.
Le prélèvement a été effectué en Août 1979 dans la tranchée drainante, après une période de sécheresse.

c) sur la route Centre Europe Atlantique (LE MONTET). Réalisation en 1972. Le géotextile assure un rôle anticontaminant entre le sol naturel mou, argilo-sableux et la couche de forme. Le prélèvement a été effectué sous l'accotement où le géotextile est recouvert de 80 cm de graves alluvionnaires sableuses.

d) sur le barrage de MAUREPAS : réalisation en 1976. Le géotextile joue un rôle de filtre sur le parement amont du barrage. Trois prélèvements ont été effectués référencés de la manière suivante :

. M_2 : au niveau de l'eau sous enrochements - partie de géotextile recouverte de sable et d'alluvions.

- . M_3 : sous enrochements au niveau de l'eau - absence de sable sur le géotextile.
- . M_4 : extrémité gauche du parement - le géotextile a été découvert de ses enrochements par des promeneurs et a été exposé aux intempéries.
- . M_0 : témoin - échantillon SODOCA de l'époque conservé dans un garage à l'abri de la lumière.

Dans les tableaux de résultats qui suivent nous ferons apparaître essentiellement les modifications des propriétés mécaniques et de structure. Le colmatage et les propriétés hydrauliques résiduelles sont traitées par ailleurs (2).

En ce qui concerne les contrôles sur fibres des géotextiles AS 420 prélevés, ils ont été conduits, d'une part, sur des fibres de la face supérieure, et, d'autre part, sur celles de la face inférieure ; en outre, on a distingué dans chaque cas les petites fibres (3 dtex) des grosses fibres (12 dtex) puisque ces géotextiles posséderaient effectivement deux populations de fibres.

A l'analyse de ces résultats, on constate que 3 géotextiles accusent des pertes de 30 % et plus de leurs propriétés mécaniques.

. Pour l'échantillon M_4 du Barrage de MAUREPAS, les modifications s'expliquent très bien par le fait que le géotextile, non protégé par les enrochements, a été exposé pendant de longs mois à l'action des intempéries

Table 3. Propriétés mécaniques des échantillons SODOCA lavés (AS 420)

Echantillons	Test ST BRIEUC (1 essai)				Test CFG (1 essai)		Test AFNOR (5 essais)		Perte Mécanique	
	$\sigma\theta_r$ (daN/m)	$\varepsilon\theta_r$	ν	E	Résistance (daN/m)	Allong^t (%)	Résistance (daN/m)	Allong^t (%)	$\Delta R_{(*)}$ (%)	ΔL (%)
NOYALO	1470	0,517	0,38	25,8	1400	118	1220	104	44	49
HYERES	2420	0,752	0,36	28,7	2283	119	2070	109	9	8
LEMONTET	2290	0,979	0,44	19,4	1208	101	1547	130	51	26
MAUREPAS										
Témoin M_0	3000	1,02	-	-	2500	127	2225	137	-	-
M_2	2200	0,919	0,43	20,9	2368	123	1970	116	5	4
M_3	2290	1,036	0,40	19,2	2390	136	2023	116	4	-
M_4	1520	0,626	0,44	20,3	1800	96	1380	136	28	31

(*) Les pertes de caractéristiques mécaniques ont été calculées par rapport à l'échantillon témoin M_0 et à partir des résultats de l'essai CFG, excepté pour la perte d'allongement de NOYALO calculée à partir de l'essai ST BRIEUC.

Table 4. Caractéristiques des fibres prélevées dans les échantillons SODOCA lavés (AS 420)

Echantillons	Fibres face supér^{re}		Fibres face inférre		Indice cristallinité RX (%)	Orientation générale (Spectro IR)	Orientation cristalline (Spectro IR)	Indice viscosité limite	Taux Irganox 1076 ppm
	Résistance (cN)	Allong^t (%)	Résistance (cN)	Allong^t (%)					
NOYALO (1972)	4,2 (1) 25,7 (2)	72,6 138	6,03 28,2	137 147	44	3,02 ± 0,13 3,07 ± 0,11	6,41 ± 0,61 6,33 ± 0,59	121	460
HYERES (1976)	10 32	114 183	 33	 156	 54,4	4,51 ± 0,64 3,35 ± 0,56	19,18 ± 7,65 7,66 ± 2,74	119	450
LE MONTET (1972)	 19,9	 101	4,9 21	81 124	49,8	2,77 ± 0,26 2,89 ± 0,11	0,75 ± 0,02 0,75 ± 0,01	121	233
MAUREPAS (1976)									
Témoin M_0	8,6 35,4	122 168	- -	- -	53,5	2,65 ± 0,06	8,14 ± 0,42	109	280
M_2	6,8 30,8	79 135	7,3 31,4	126 156	50,6	2,57 ± 0,08	7,36 ± 0,65	112	190
M_3	8,5 34,2	145 152	8,1 33,9	130 150	42,7	2,95 ± 0,10	11,58 ± 1,43	107	280
M_4	5,3 23,1	77,4 54,7	6,4 29,6	159 116	43,1	2,77 ± 0,16	11,31 ± 1,4	116	180

(1) Petites fibres
(2) Grosses fibres

du batillage, de l'abrasion, etc... Il en a résulté une décristallisation du polymère fibreux, une chute importante de la barrière antioxydante (taux Irganox), un accroissement sensible de l'indice de viscosité, signe de réticulation du polymère ou d'extraction d'oligomère : autant d'évolutions de la structure qui peuvent rendre compte des diminutions enregistrées sur les propriétés mécaniques.

Table 5. Pertes des caractéristiques mécaniques des fibres (*)

Echantillons	Perte Résistance (%)		Perte Allongement (%)	
	grosses	petites	grosses	petites
NOYALO	23	42	25	18
HYERES	8	-	-	8
LE MONTET	43	44	56	41
MAUREPAS M_2	11	19	23	20
M_3	3	0	18	0
M_4	26	32	83	4

(*) Ces pertes sont calculées par rapport aux propriétés des fibres M_0 (Témoin)

Pour LE MONTET, il faut faire appel à l'analyse morphologique du géotextile in situ dans le sol pour mieux comprendre les modifications (2). Effectivement le géotextile apparaît très mal aiguilleté du fait d'une fabrication particulière ... mais plutôt d'une décohésion résultant certainement d'une exposition trop longue aux intempéries avant recouvrement par la terre d'apport sur l'accotement de la route. Cette hypothèse serait confortée par le fait que le taux d'additif d'Irganox qui devrait être comparable à celui trouvé dans l'échantillon contemporain de NOYALO est en fait 50 % plus faible, et également par le fait que les paramètres de cristallinité des fibres chutent. Pour cet échantillon, le test de ST BRIEUC ne fait pas apparaître la perte de résistance en traction décelée par le test grande largeur CFG. Le principe même du test sur manchon de ST BRIEUC peut expliquer ces différences. Le test triaxial restaurerait un peu de la cohésion entre les couches de fibres, cohésion perdue dans l'échantillon par la déficience aux ruptures de l'aiguilletage. Le test biaxial du CFG, au contraire, révèle cette déficience au lieu de la compenser en partie.

Dans le cas de NOYALO, pour expliquer les pertes de caractéristiques enregistrées par rapport au témoin choisi, il faut recourir aux observations et photographies faites au moment du prélèvement qui montrent que le géotextile en contact avec un sol naturel très mou, s'est déformé sous le poids du remblai et est affecté de très nombreuses empreintes dûes à la couche d'apport 20/40 (mais aucune perforation). Le géotextile a donc travaillé correctement sur le plan mécanique et ceci suffit à expliquer en partie les résultats des essais de traction sur le prélèvement. On peut penser que le choix d'un sol d'apport de granulométrie plus fine aurait certainement occasionné moins de déformations du textile.
En outre, le géotextile est relativement contaminé et colmaté comparativement aux autres prélèvements étudiés (2), cette pénétration intime du sol argilo-limoneux dans la structure du géotextile a certainement sollicité mécaniquement les fibres (fluage...). Les caractéristiques structurales des fibres témoignent de cette évolution : perte notable de cristallinité et augmentation de l'anisotropie (orientation générale) attribuables à un fluage.

Les autres échantillons HYERES, M_2, M_3, n'ont pratiquement pas travaillé mécaniquement et de ce fait les pertes de caractéristiques mécaniques enregistrées restent faibles ou nulles comparativement au témoin M_0 choisi.

Le fait d'analyser les caractéristiques physico-chimiques des fibres prélevées, soit à la face inférieure soit à la face supérieure, des géotextiles, n'apporte pas d'élément d'information supplémentaire. Par contre, la distinction entre grosses fibres et petites fibres permet de confirmer ce qu'on pouvait prévoir, à savoir :

- les petites fibres perdent en général plus de résistance que les grosses ;
- les grosses fibres perdent plus de capacité de déformation que les petites.

2.1.2 Géotextiles tissés :

Cas de deux échantillons ADITEX de 137 g/m2 (tissés de bandelettes de polypropylène) prélevés en 79:

a) sur la déviation de FOURCHON-SECLIN : réalisation en Novembre 1976. Le géotextile joue un rôle anti-contaminant en sol naturel (limon sur argile de Louvil) et une couche d'apport de 0,20 m de schiste rouge surmonté d'une couche de grave laitier 0/20 de 0,40 m et enfin d'une couche de cure. Au prélèvement, le géotextile présentait des empreintes de poinçonnement avec perforations sous l'effet des cailloux de schiste.

b) sur l'autoroute B71 (Déviation de THIERS) : réalisation Janvier 1973. Le géotextile a été placé sur un sol argileux sur lequel un remblai en graves tout-venant alluvionnaire de la Dore a été construit (hauteur maxi 5 m). Au prélèvement, le géotextile apparaît très plan, sans poinçonnements ni déchirures importants. A noter, que l'eau baignait le textile.

La Table 6 regroupe les résultats mécaniques obtenus sur ces éprouvettes.

Table 6. Caractéristiques mécaniques : ADITEX 137 g/m2

Echantillons	Essai AFNOR (5 essais)		Pertes de Propriétés
	Résistance daN/m	Allongement (%)	ΔR (%)
Témoin : chaîne	2800	21,5	
: trame	2300	13,4	
FOURCHON (1976)	2114	12,4	17
THIERS (1973)	1191	13	53

Indices de cristallinité du polypropylène :

FOURCHON : 47 % THIERS : 66 % (augmentation très nette de la cristallinité par rapport à l'échantillon de FOURCHON).

Pour l'échantillon de FOURCHON, les pertes de résistance sont certainement attribuables aux perforations et poinçonnements notés sur les éprouvettes. Pour THIERS, il faut admettre que l'échantillon a flué sous le poids du remblai pour expliquer les pertes de caractéristiques mécaniques.

2.2 Géotextiles en polyester

2.2.1 Géotextiles nontissés aiguilletés

Cas de plusieurs BIDIM non comparables entre eux.

2.2.1.1 BIDIM 300 g/m2 mis en place en 1969 et 1970 :

. Piste sur le Marais de REDON : l'utilisation du BIDIM a eu lieu dans deux zones différentes et dans des conditions différentes.
- 1ère zone : zone basse couverte d'eau sur laquelle des fascinages ont été posés et qui a reçu un remblai d'une épaisseur d'un mètre. Le remblai est constitué d'enrochements tout-venant de carrières de granulométrie comprise entre 0 et 300-1000. Le remblai nivelé a été recouvert d'une couche de sable

puis du BIDIM. Une couche dîte de roulement composée de tout-venant (granulométrie 0-200) de carrière de grès micacé relativement dur, a été déposée sur le BIDIM. Le BIDIM s'est comporté comme anticontaminant et drain, facilitant la consolidation des vases sous-jacentes; il a également un effet répartiteur de charge assurant la stabilité de la chaussée.

- 2ème zone : située au milieu du marais, zone couverte d'eau. Le BIDIM a été posé directement sur le marais enherbé. Remblayage sur BIDIM avec du tout-venant de carrières identique au précédent (épaisseur 0,70 m).

Aucune remontée de vase n'a été décelée dans les 2 zones. Sur cette piste, on peut estimer qu'il y a eu environ 5000 passages de camions pesant en charge 19 à 20 tonnes pendant une durée de 4 mois suivant sa construction (en période de pointe 2000 tonnes de remblai/jour).
Un premier examen du profil de la piste en 1970, a montré dans les 2 zones, un orniérage de 10 à 15 cm sous le passage des roues, quelques perforations du BIDIM, pas de colmatage et pas de contamination du remblai.

Prélèvements : en 1977, dans la zone 1 : réf. R_1
en 1980, dans la zone 1 : " R'_1
" dans la zone 2 : " R'_2
échantillon témoin : " R_0

L'échantillon témoin vieilli dans des conditions assez peu précises, sera situé lui-même par rapport aux données techniques de l'époque du producteur. Ces échantillons de REDON sont particulièrement intéressants, parce que prélevés à l'endroit de la piste ayant supporté un trafic intense.

. Chemin des grèves (CARENTAN) : réalisation 1971. BIDIM anticontaminant entre un limon cohérent très plastique et un matériau de recouvrement de grès quartzite concassé 0/30 mm (25 cm). Le géotextile prélevé en Décembre 1979 présente des perforations occasionnées par la grave concassée. Le sol en place apparaît très consolidé.

Les résultats sont regroupés dans les Tables 7 & 8. Les valeurs ΔR & ΔL sont calculées par rapport au test AFNOR.

Il est intéressant de noter que les échantillons de REDON n'ont pas perdu plus de 10% de leur résistance. Ils ont essentiellement perdu de la déformabilité. Les pertes de 20 à 30% enregistrées sur l'allongement à la rupture correspondent exactement aux déformations subies par le BIDIM du fait de l'orniérage constaté lors du prélèvement en 1980. Les 2/3 de cette déformation auraient été acquise dès les premiers mois de mise en service de l'ouvrage. Les essais sur fibres confirment à une échelle moindre la déformation supportée par BIDIM. L'indice d'orientation révèle une augmentation de l'anisotropie des fibres. Pas de dégradation chimique notable.
L'échantillon de CARENTAN apparaît plus dégradé et de toute évidence, a souffert essentiellement de poinçonnements et de perforations dûs à la couche de grave 0/30.

2.2.1.2 BIDIM 400 g/m2 mis en place entre 1970 et 1971 :

. RN 9 ST FLOUR : réalisation Automne 1970. Sol en place: Argile. Matériau d'apport : grave concassée 0/31,5. Prélèvement en bordure de chaussée : le géotextile présente un aspect sain.

. Chemin Dép. 77 de CHEVERNY : réalisation 1970. Zone très humide. Sol support : sable argileux. Matériau d'apport : grave sableuse de Loire, calcaire 0/20, terre végétale. Prélèvement Octobre 1979.

. Rectification de ST ANGEL-CAMBRESSOL : réalisation fin 1971. Sol support : argile + tourbe. Matériau d'apport: arène granitique (remblai de 3 m). Prélèvement en pied de talus.

Les évolutions de caractéristiques mécaniques ont été calculées par rapport à des échantillons témoins de BIDIM 400 g de 1973, conservés parfaitement stockés à l'abri de la lumière. Il a été vérifié préalablement, sur ces témoins, l'excellent accord avec les données techniques des producteurs de l'époque. Voir résultats Table 9.
Les pertes de résistance et d'allongement sont calculées à partir des tests AFNOR.

2.2.1.3 BIDIM U 34 :

. Accotement de la RN 28 (Neufchâtel en Bray) : Réalisation Septembre 1977. Géotextile anticontaminant prélevé en Septembre 1979.

Table 7. Caractéristiques mécaniques de prélèvements BIDIM 300 g/m2 lavés

Echantillons	Essai ST BRIEUC (1 éprouvette)				Essai CFG (1 éprouvette)		Essai AFNOR (5 éprouvettes)		Pertes Caractéristiques Mécaniques	
	$\sigma\theta_r$ (daN/m)	$\varepsilon\theta_r$	ν	E	Résistance (daN/m)	Allong^t (%)	Résistance (daN/m)	Allong^t (%)	ΔR (%)	ΔL (%)
Référence 1969	2200	0,33	-	-	-	-	1300	68	-	-
REDON : R_0	-	-	-	-	1234	52	1164	61	10	7
R_1	-	-	-	-	-	-	1250	48	4	20
R'_1	-	-	-	-	1660	29	1180	38	9	30
R_2	1560	0,31	0,36	47	1860	41	1464	47	-	21
CARENTAN 1971	-	-	-	-	-	-	480 (*)	26	63	42

(*) Les éprouvettes de traction présentent des perforations de 2 à 3 cm dans toute leur surface.

Table 8. Caractéristiques des Fibres prélevées dans les échantillons BIDIM 300 g/m2 lavés

Echantillons	Essais Mécaniques s/fibres		Indice Orientation (Spectro IR)	Indice Cristallinité (%)	Eq. COOH 10^6 g	Pertes Caractéristiques Mécaniq	
	Résistance (cN)	Allongement (%)				ΔR (%)	ΔL (%)
Référence 1969	25,5	35	-	63	46	-	-
REDON : R_0	23	40	2,47 ± 0,08	-	54	6	-
R'_1	24	32	2,76 ± 0,07	57	53	2	3
R'_2	23	30	2,65 ± 0,06	62	53	6	5
CARENTAN	18	23	2,85 ± 0,15	-	53,7	26	12

. RN 12 Créneau de Gandelain : Réalisation 1977. Le géotextile a pour fonction de conforter l'ouvrage et de drainer les eaux de ruissellement. Prélevé en 1979.

. Planches d'essais du CER Rouen : Réalisation 1974. Ouvrages avec géotextiles en contact avec un sol peu porteur, soumis à 230 passages d'essieux de 13 tonnes. Planches référencées respectivement 1, 2, 3, pour des matériaux de recouvrement du BIDIM : grave 0/40 semi-concassée, ballaste 20/60, tout-venant 0/100. Les pertes de caractéristiques mécaniques sont calculées par rapport aux résultats obtenus en 1973 par ITF sur du BIDIM U 34 (cf. Table 8).

Table 9. Evolution des caractéristiques mécaniques des BIDIM 400 g/m2 et U34 lavés

Echantillons		Pertes propriétés BIDIM		Pertes propriétés Fibres	
		ΔR (%)	ΔL (%)	ΔR (%)	ΔL (%)
400 g	ST FLOUR	36	18	46	31
	CHEVERNY	21	20	20	11
	CAMBRESSOL	22	8	12	15
U 34	NEUFCHATEL	23,6	22	0	4
	GANDELAIN	31	30	8	6
	CER-ROUEN 1	-	35	-	7
	CER-ROUEN 3	7	23	2	7

2.2.2 Géotextiles tissés

2.2.2.1 Stabilenka N99 & PX 99 (tissé de polyester et de polyamide)

. Planches d'essais du CER - ROUEN : 4 planches ont été réalisées dans des conditions identiques à celles décrites précédemment pour le BIDIM U34, à savoir :

Planche 4 : Stabilenka N99 utilisé comme anticontaminant sur sol peu porteur et recouvert de tout-venant 0/100. Le prélèvement est déchiré en maints endroits. Perte de résistance : 13 %. Perte d'allongement à la rupture : 2 points.

Planche 5 : Stabilenka N99 ; anticontaminant entre un sol peu porteur et un matériau de recouvrement (ballast). Le géotextile est déchiré. Perte de résistance 55%. Perte d'allongement à la rupture 7 points.

Planche 6 : Stabilenka N99; anticontaminant entre un sol peu porteur et un matériau de recouvrement (grève 0/40). Le géotextile est en assez bon état. Perte de résistance 63 %. Perte d'allongement 7 points.

Planche 7 : Stabilenka PX 99. Anticontaminant sur sol peu porteur. Matériau de recouvrement (grève 0/40). Le géotextile apparaît endommagé au prélèvement. Perte de résistance 63 %. Perte d'allongement 4 points.

Ces derniers résultats, comparés à ceux obtenus avec BIDIM U34, montrent très bien comment se comporte un nontissé aiguilleté par rapport à un tissé soumis à une même contrainte, en l'occurrence 230 passages d'essieux de 13 tonnes. Le nontissé se déforme en accompagnant et en consolidant le sol support. Le tissé, moins déformable, se déchire et se perfore sous l'effet des poinçonnements du sol de recouvrement, il est surtout affecté dans sa résistance à la rupture.

2.2.2.2 TRIX : tissé tridimensionnel en monofilament de polyester (diamètre 22/100 mm).

. RN 7 : Parachute de pierres (8 ans d'exposition aux intempéries) :
- perte de résistance des monofilaments : 5 %
- perte d'allongement à la rupture : 21 points
- perte de cristallinité du polyester : 33 %

Cette exposition prolongée à l'extérieur a donc entrainé une décristallisation poussée du polymère qui est surtout devenu fragile (peu déformable).

. Barrage de MARAVAL : 3 à 4 ans d'utilisation comme brise-chute sous le déversoir du Barrage.
- perte de résistance des monofilaments : 15 %
- perte d'allongement à la rupture : 1 point
- perte de cristallinité : 15 %

3. CONCLUSIONS

Toute l'information recueillie au cours de ce travail apparaît précieuse et permet d'apporter des éléments de réponse positifs aux questions que les prescripteurs se posent sur le vieillissement des géotextiles :

- tout d'abord, sur un plan très empirique, mais qui a son importance, les géotextiles peuvent être retrouvés dans les ouvrages, à la place où les ingénieurs les avaient placés 10-15 ans auparavant. Ils ont donc duré. Les constatations faites sur les sites des prélèvements indiquent qu'ils ont bien fonctionné et qu'ils peuvent encore fonctionner longtemps.

- ensuite, sur un plan déjà plus scientifique, les modifications de propriétés des géotextiles ont été mesurées, non seulement au niveau du géotextile lui-même, mais également à celui plus fin des filaments élémentaires constitutifs. Les propriétés mécaniques ont été considérées parallèlement aux modifications structurales du polymère. En ce qui concerne les géotextiles nontissés aiguilletés on peut dire que la plus grande partie des prélèvements n'accuse pas de perte de résistance supérieure à 30 %. Il existe d'ailleurs dans ces cas, une assez bonne corrélation entre modifications des propriétés mécaniques des géotextiles et celle des filaments élémentaires. Pour ces géotextiles, les modifications des propriétés résultent essentiellement de la synergie entre les actions d'une énergie mécanique (contraintes supportées lors de la réalisation ou du service de l'ouvrage) et de l'environnement. Les géotextiles ont donc travaillé mécaniquement, mais en cela ils ont rempli parfaitement leur mission : le cas du prélèvement sous la piste de REDON est remarquable à cet égard. Les changements physico-chimiques mesurés sur les fibres restent faibles, ils résultent des contraintes mécaniques supportées par le géotextile et s'interprètent assez bien de cette manière. Il est intéressant de situer parmi ces géotextiles, ceux qui ont subi les tests de vieillissement par enfouissement pendant plusieurs mois (1), ils n'ont pratiquement pas travaillé et ont supporté essentiellement les contraintes de manipulations à la mise en place et au prélèvement (ΔR < 25 %, ΔR Fibres < 10 %). Les géotextiles qui accusent plus de 30 % de perte de résistance ont été soit exposés au soleil longtemps, soit très perforés (tissés ou nontissés).

- Enfin, tous ces résultats ne révèlent pas de dommages rédhibitoires des géotextiles, sous des contraintes chimiques ou autres. En conséquence, si le géotextile est bien choisi en fonction des matériaux d'apport et du sol en place, sa durabilité devrait être supérieure à 100ans dans des conditions classiques et normales de mise en oeuvre.

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Geotextiles and Aging Tests
Les géotextiles et les tests de vieillissement accéléré

In this lecture will be recalled general problems linked to ageing of geotextiles through small-scale accelerated laboratory tests : chemical and photochemical tests especially.

The researchworker's position is incertain in front of the multitude of stresses and properties to take into account and also in front of the deficit of knowledge in the field of polymer ageing. As a result, it appears very difficult to simulate ageing of geotextiles with small-scale laboratory tests.
It is shown that simple chemical accelerated tests stand generaly on an severity level badly fitted with true modifications ascertained after natural ageing.
On the other hand photochemical tests have been proposed which gives results will fitted with changes of structure and properties of geotextiles after outdoor ageing.

At last, an outdoor ageing (3 months outdoor) of course weakens geotextiles but does not enhance effects of subsequent burying or those of subsequent chemical treatments (basic, acid, sea water).

Dans ce texte seront rappelés les problèmes généraux posés par l'étude du vieillissement des géotextiles à travers des tests de laboratoire : tests chimiques et photochimiques notamment.

L'incertitude où se trouve le chercheur devant la multiplicité des contraintes que peut supporter un géotextile en utilisation réelle, devant également la quantité des propriétés et caractéristiques à étudier, et enfin, en face du déficit de connaissances sur le vieillissement des polymères, fait qu'il lui apparaît très difficile de simuler au laboratoire le vieillissement des géotextiles. Il est montré que les tests chimiques simples et accélérés sont généralement d'un niveau de sévérité mal ajusté qui ne rend pas compte des modifications réelles constatées sur des géotextiles prélevés dans des ouvrages. Par contre, les tests de photovieillissement (UV et xénotest) proposés fournissent des résultats en assez bonne concordance avec les modifications de propriétés et de structure consécutives à un vieillissement à l'extérieur.

INTRODUCTION

Les géotextiles peuvent-ils durer 50 ans, 100 ans ou plus ?... Le colloque de Paris en 1977 a posé ces questions en termes très clairs. Depuis, toutes les instances qui, dans le monde, se préoccupent des géotextiles essayent d'apporter des éléments de réponse à ces interrogations, renseignements qu'attendent tous les prescripteurs et projeteurs tentés par l'utilisation des géotextiles dans des ouvrages permanents.

Empirisme ou Science ?... Quelles sont les attitudes possibles des chercheurs confrontés à un tel problème ?

- Démarche empirique d'abord, qui, pour beaucoup consiste à rechercher dans les ouvrages de génie civil, les géotextiles enfouis depuis plus de 10 ans. Dans cette approche, les géotextiles sont en général retrouvés à l'endroit où les ingénieurs les avaient placés. L'observation empirique permet alors de consigner les modifications macroscopiques éventuelles des prélèvements et dans la majorité des cas de constater que les géotextiles, non seulement existent toujours, qu'ils ont donc duré, mais qu'aussi, ils ont parfaitement assuré leur fonction et continuent à la tenir parfaitement. Mais alors, ont-ils vieilli et combien de temps peuvent-ils encore durer ?.. Pour essayer de répondre à ces nouvelles et légitimes interrogations, il faut alors pour le chercheur entreprendre une démarche plus scientifique. Tout d'abord, chiffrer les évolutions physico-chimiques du polymère dont est fait le géotextile. Ensuite, rechercher les causes qui sont à l'origine de ces modifications éventuelles. La corrélation entre l'évolution structurale du polymère et les propriétés physiques les plus intéressantes pour l'ingénieur permettra d'accéder à des données pratiques. Malheureusement, dans cette démarche, il faut bien reconnaître que le recul fournit par les premières utilisations des géotextiles est encore très faible et que le déficit des connaissances sur le vieillissement des polymères est grand, autant de causes qui ne permettent pas de tirer des conclusions radicales sur la durabilité des géotextiles. L'expérience conduite par le Comité Français des Géotextiles (CFG) sur des géotextiles prélevés in-situ sera décrite dans un autre article (1).

- Une autre démarche est souvent suivie, notamment, pour permettre l'acquisition rapide de résultats et qui consiste à faire appel à des méthodes de vieillissement accéléré et/ou simulé. Là encore, il faut recourir à un systématisme empirique et le déficit de connaissances impose une simulation extrêmement rigoureuse des conditions d'usage. La prise en compte de tous les paramètres possibles alourdit énormément le champ d'expérimentation. Pour illustrer ce fait, on peut citer en exemple la lourdeur des études conduites sur le vieillissement des parachutes qui, progressivement, ont amené les chercheurs à prendre en considération jusqu'à l'influence du "jus de sauterelles" sur la résistance des textiles... En outre, on peut craindre que toutes les méthodes de vieillissement accéléré, qui reposent sur la prise en compte d'un seul paramètre, que l'on intensifie pour concentrer l'échelle du temps, ne constituent que des demi-mesures. Cette démarche est néanmoins intéressante à suivre, ne serait-ce que pour tester la résistance relative au vieillissement de plusieurs types de géotextiles. Dans la suite de ce texte, nous décrirons l'expérience du CFG dans le domaine du vieillissement accéléré des géotextiles en polypropylène et polyester, dans laquelle ont été pris en compte des contraintes simples ou combinées (expositions extérieures et enfouissement ou actions chimiques). Revenons quelques instants aux problèmes posés par l'étude du vieillissement des géotextiles.

1 GENERALITES SUR L'ETUDE AU LABORATOIRE DU VIEILLISSEMENT DES GEOTEXTILES

Les questions qui se posent actuellement aux utilisateurs des géotextiles vont les conduire, à l'instar de ce qui se fait déjà dans le monde des producteurs de matière plastique, à la nécessité de prévoir les modifications de propriétés que subissent les produits en cours de stockage et d'utilisation. Ces modifications résultent de changement de la matière fibreuse qui compose le géotextile. Elles sont provoquées et/ou accélérées par les contraintes qu'il subit et qui sont les véritables causes du vieillissement.

L'incertitude dans laquelle se trouve le chercheur chargé d'étudier le vieillissement des géotextiles résulte de la complexité des facteurs mis en cause :
. qu'il s'agisse des conditions d'emploi et donc des contraintes réelles (mécaniques, chimiques, photochimiques, biologiques)
. ou bien du niveau minimal acceptable pour les propriétés physiques étudiées dont les variations serviront de critère relatif de vieillissement.

Les problèmes qui se posent pour les géotextiles sont communs à toute étude d'objet, il faut simplement considérer avec attention la finesse des fibres constitutives. Devant la multitude des propriétés, des contraintes, on a tendance à focaliser son attention uniquement sur la conservation et la dégradation des propriétés fondamentales (résistance) au risque de négliger d'autres aspects importants (allongement, fluage...). D'ailleurs, dans certaines conditions de vieillissement (hydrolyse en milieu humide), les caractéristiques mécaniques jugées importantes, tels que module, résistance, dureté, sont parmi les dernières propriétés à bouger, alors que d'autres (allongement de rupture) révèlent très vite l'évolution des polymères vers la fragilité. La connaissance des contraintes est certainement difficile et on oublie trop souvent que certaines contraintes subies par un géotextile dans un ouvrage ne sont perçues qu'à travers les insuffisances constatées à l'expérience (effet éducatif des échecs).

En outre, une intéraction s'établit d'une certaine façon, entre les propriétés et les contraintes, dans la mesure où les propriétés des matériaux déterminent le comportement des utilisateurs à leur égard. Des produits nouveaux comme les géotextiles se heurtent à des coutumes bien établies dans le domaine des travaux publics et il arrive que leur manipulation conduise à des maladresses, sinon emplois inadéquats. Néanmoins, avec de nouvelles habitudes, avec l'amélioration des géotextiles, les utilisateurs vont exiger plus des produits.

Le vieillissement et son étude posent d'autres problèmes relatifs à la durée d'application des contraintes à leur alternance ou simultanéité. Devant cette complexité on peut imaginer que l'accélération du processus de vieillissement au laboratoire pour la définition des propriétés à terme des géotextiles s'avère très difficile. Notamment, le facteur d'accélération pour chaque contrainte n'est pas le même : l'intensification des contraintes, l'absence de période de relaxation, peuvent conduire à des résultats irréalistes.

Par commodité, certains auteurs (2) ont situé les causes du vieillissement au confluent de l'action d'une énergie (apportée ou libérée) et de l'action d'un environnement :
. l'énergie peut être d'origine thermique, photochimique mécanique
. l'environnement peut agir sur la surface du géotextile et être à l'origine d'échange de matière dans le sens géotextile → environnement (agents protecteurs, fibre) ou environnement → géotextiles (oxygène, eau, agents chimiques, particules de sol, végétation).

Enfin, il faut toujours distinguer parmi les nombreuses méthodes d'étude du vieillissement accéléré :
. celles qui révèleront l'aptitude du géotextile à résister à une contrainte (enceinte de photovieillissement, étuve) et qui seront aptes notamment à faire des comparaisons de produits;
. celles qui permettront de suivre l'évolution physicochimique des fibres du géotextile sous l'effet des contraintes. Expériences plus difficiles, plus riches également, qui devraient permettre à terme l'établissement de corrélations structure-propriétés, directement exploitables sur le plan pratique.

Nous examinerons dans le chapitre suivant, les vieillissements des géotextiles BIDIM et SODOCA selon ces deux types de méthodes.

2 EFFET DES TESTS DE VIEILLISSEMENT ACCELERE SUR LES CARACTERISTIQUES DES FIBRES DES GEOTEXTILES

Dans cette phase de l'étude, nous avons soumis les géotextiles à des tests de vieillissement accéléré au laboratoire et nous avons volontairement suivi les modifications des caractéristiques des fibres élémentaires prélevées dans les géotextiles "vieillis".

Ces tests conduits sur du BIDIM U44 sont :
. immersion dans des solutions aqueuses d'HCl 6N, 115°C
. immersion dans des solutions aqueuses OHNa N, 105°C
  durées : de 15 mn à 2 h.
. exposition au xénotest 150 : durée de 2 h à 370 h soit des doses de 0,08 à 15 kilo Langley (KLy)
. exposition sous UV (lambda = 265 nm)
  durées : de 2 à 100 h.
  sous des tubes Philips 15 WT UV 57415 P/40

Les résultats d'essais mécaniques sur fibres prélevées dans les échantillons vieillis (50 fibres par échantillon) ont été décrits par ailleurs (3); ils sont simplement illustrés par les courbes de la Fig. 1.

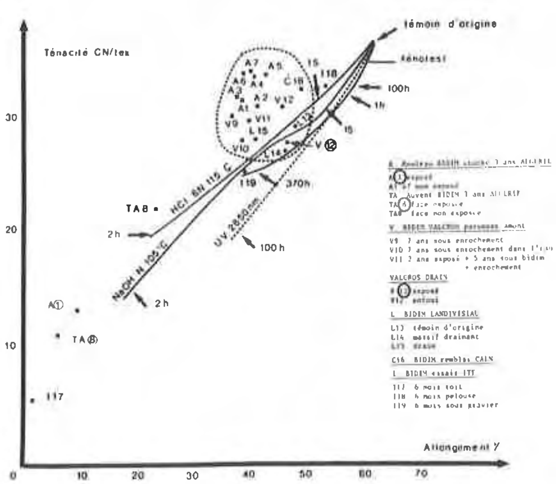


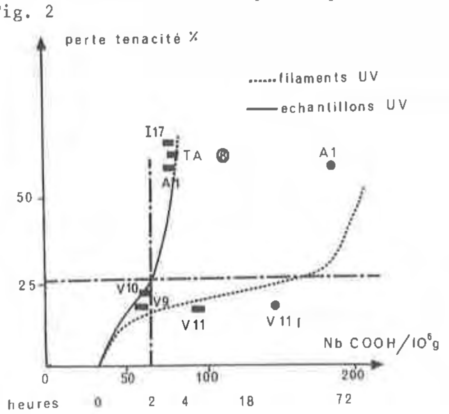
Fig. 1 Evolution des propriétés mécaniques des fibres après vieillissement.

Les courbes représentent l'évolution de la ténacité et de l'allongement de rupture des fibres après vieillissement. Leur allure traduit évidemment une dégradation des caractéristiques mécaniques. Mais, si on se réfère aux autres points significatifs du diagramme référencés A,V,L,I, (respectivement comme Algérie - stockage géotextile 3 ans - Valcros - géotextile parement barrage - Landivisiau - géotextile drainage - ITF - vieillissement géotextile à l'extérieur) qui sont relatifs eux, à des géotextiles vieillis en utilisation réelle, on ne peut pas dire que les tests de vieillissement artificiel accéléré soient très représentatifs de ce qui advient naturellement :

. d'une part, les tests chimiques basés sur une intensification de la concentration d'acide ou de base et une augmentation de la température, sont trop sévères et hydrolysent trop vite le polyester. En comparaison, les géotextiles après 3 ou 10 ans d'utilisation de stockage, ont tous leur point représentatif dans la zone cerclée du diagramme, donc se situent au-dessus des courbes de dégradation accélérée.

. d'autre part, les tests photochimiques ne sont pas assez sévères et ne reproduisent pas significativement les dégradations enregistrées sur les fibres de polyester après des expositions intempéries (lumière comprise) de plusieurs mois (points I_{17} - A_{1} - TA 8).

En cas d'exposition à la lumière d'un géotextile, seules les fibres de surface, donc une faible proportion de la masse du produit, peuvent être dégradées sous l'effet synergétique des photons, de l'oxygène, de l'humidité et de la température. Les tests de photovieillissement accéléré, notamment sous UV, ont été repris pour tenir compte de la modification des seules fibres de surface. Pour cela, des filaments ont été extraits du BIDIM, disposés parallèlement en une couche, et exposés aux lampes UV (Enceinte de 10 lampes Philips : lambda 265nm, température de l'enceinte régulée à 40°C, distance lampes-fibres réglée à 4 cm) : dans ce montage, les deux faces de la couche de fibres sont exposées aux UV. Simultanément, un échantillon de BIDIM intact est exposé aux UV dans les mêmes conditions, mais seule une face est irradiée comme dans une utilisation réelle. Après des durées d'exposition plus ou moins longues dans ces conditions, des essais mécaniques ont été conduits sur les fibres exposées et la dégradation photochimique a été estimée par dosages de groupes COOH apparus. On fera l'hypothèse que les fibres superficielles du BIDIM exposé ont subi des dégradations mécaniques identiques à celles des fibres élémentaires disposées en une couche sous UV. Le dosage des groupes COOH totaux sur BIDIM fournit évidemment des valeurs plus faibles que celles obtenues sur couches de fibres, puisque dans BIDIM seules les fibres de surface irradiées interviennent dans l'augmentation des groupes COOH. Les résultats sont exprimés par les courbes de la Fig. 2



La perte de tenacité des fibres après exposition augmente à mesure que le nombre de COOH s'accroît. Evidemment l'allure des courbes est différente, selon que l'on considère les COOH dosés sur les fibres ayant toutes été irradiées (courbe en pointillé) ou ceux dosés sur BIDIM irradié (courbe trait plein).

Il est très intéressant de noter que les mesures effectuées sur des échantillons de BIDIM ayant été exposés très longtemps, volontairement ou involontairement, aux intempéries (notamment lumière), permettent de situer les points représentatifs sur les courbes de la Fig. 2. Ce dernier résultat est fort important et précieux, car d'une certaine manière il va permettre d'étalonner les courbes de photovieillissement accéléré à l'aide de points repères. Il convient de rappeler que les échantillons V_{11}, I_{17}, A_{1}, TA 8 sont des BIDIM qui ont été exposés aux intempéries et qui ont reçu approximativement et respectivement 160, 100, 300, 300 KLangley et qui de ce fait ont tous été décohésionnés par rupture de fibres en surface, notamment au niveau des trous d'aiguilletage. En conséquence, on peut tenter de retenir pour apprécier la durabilité au soleil d'un tel géotextile, un test de 2 h d'exposition aux lampes UV dans les conditions décrites précédemment. Si, au terme de cet essai, le nombre d'équivalents COOH dosés sur le BIDIM excède $65/10^6$ g., on peut craindre que les filaments superficiels irradiés aient perdu en moyenne plus de 25 % de ténacité et que des points d'aiguilletage aient lâché. Une telle démarche pourrait être conduite pour tous les géotextiles, à condition de disposer de points de références pour étalonner les courbes, références correspondant à des géotextiles vieillis naturellement.

Par contre, si l'on considère, au lieu des pertes de ténacité des fibres, les pertes d'allongement après irradiation en fonction du nombre de COOH apparu, les courbes sont d'allure identique à celles de la Fig. 2 mais il n'est plus cette fois possible d'obtenir une bonne coïncidence avec les géotextiles vieillis à l'extérieur : les points représentatifs de ces géotextiles se situent très nettement en dehors des courbes (Fig. 3).

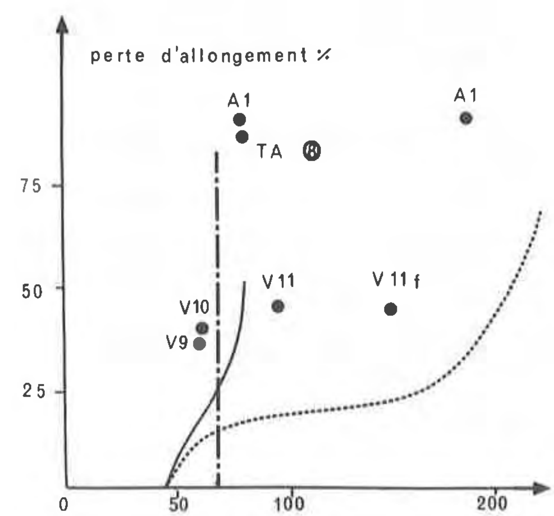


Fig. 3 Corrélations entre perte d'allongement des fibres et nombre de COOH.

← Fig. 2 Corrélations entre perte de ténacité des des fibres et nombre de COOH.

Si l'on suit le photovieillissement, non plus sous lampes UV, mais au xénotest 150, on enregistre également une perte de ténacité des fibres et un accroissement des groupes COOH en fonction de la durée d'exposition. Néanmoins, la sévérité du test est plus faible. La perte de ténacité obtenue après 300 h d'exposition au xénotest 150 est équivalente à celle obtenue après 25 h seulement sous UV. Corrélativement, le nombre de COOH apparu après le xénotest est beaucoup plus faible qu'après irradiation UV. Ces résultats sont illustrés par les courbes de la Fig. 4 qui montrent l'évolution de la ténacité des fibres de surface du BIDIM en fonction du nombre de COOH totaux sur BIDIM irradié. La coïncidence du vieillissement xénotest avec les points repères correspondant au BIDIM vieilli naturellement est nettement moins bon qu'avec le vieillissement UV.

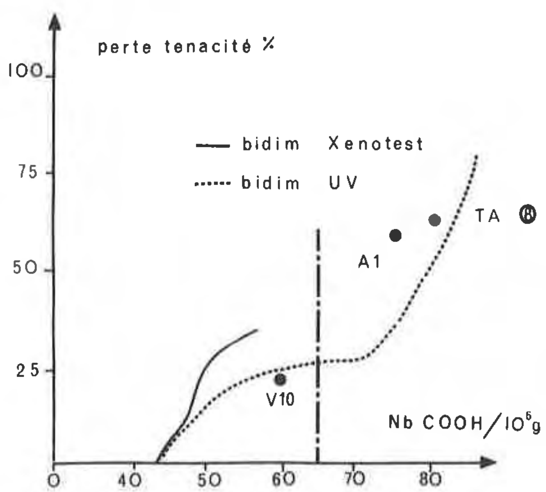


Fig. 4 Evolution de la perte de ténacité des fibres superficielles de BIDIM en fonction de COOH apparus après irradiation soit aux UV, soit au Xénotest 150.

3 EVOLUTION DES CARACTERISTIQUES DES GEOTEXTILES APRES DES TESTS DE VIEILLISSEMENT COMPLEXE

3.1 Description des tests et des échantillons

Suite aux premiers essais de vieillissement simple décrits précédemment, une autre expérimentation a été entreprise comparativement sur BIDIM U44 et comparativement sur SODOCA AS 250 et AS 420.

Les échantillons de géotextile ont été soumis à des tests de vieillissement chimique, moins sévères que ceux décrits au paragraphe 2, à savoir :

. solution HCl, pH = 3, 20°C, durée 15 j. à 19 mois
. solution OHNa, pH = 12, 20°C, -"- -"-
. eau de mer : solution aqueuse NaCl, 30 g./l., 20°C.

Pour apprécier l'effet éventuel d'une sensibilisation des géotextiles aux agressions chimiques, qui résulterait d'une exposition préalable aux intempéries, tous les essais ont été conduits simultanément sur les échantillons vierges et sur des échantillons ayant subi une exposition de 3 mois dans la région parisienne (Février-Mars 1979 : soit l'équivalent de 20 KLangley).

Enfin, des tests d'enfouissement dans du sol naturel ont été réalisés sur les échantillons vierges et pré-exposés 3 mois. Le creusement de la tranchée d'une profondeur comprise entre 50 et 80 cm, ainsi que le compactage et l'opération de fouille, ont été réalisés avec des engins de chantier. Les échantillons ont été prélevés régulièrement pour contrôle pendant 24 mois.

3.2 Résultats

Les propriétés mécaniques ont été contrôlés selon le test AFNOR (traction sur bandes de 5 cm). Sur les échantillons SODOCA en polypropylène les modifications chimiques ont été appréciées à travers les mesures d'indice de viscosité du polymère. Sur le BIDIM, les modifications chimiques ont été suivies par dosages de groupes COOH du polyester.

Nous rassemblons dans les tableaux suivants les principaux résultats de ces essais (cf. tableaux 1, 2,3).

Tous ces résultats indiquent que les géotextiles BIDIM et SODOCA ne subissent pas d'altération significative de leurs caractéristiques mécaniques par les traitements acide, alcalin, eau de mer, dans les conditions choisies et dans les limites de cette expérimentation. Certes, de légères modifications chimiques sont constatées par les dosages des groupes carboxyliques pour le polyester, et par l'augmentation de l'indice de viscosité pour le polypropylène. Ces modifications n'apparaissent pas corrélables avec des pertes de résistance. Elles traduisent une légère hydrolyse du polyester et des extractions de plastifiants, d'oligomères, sinon des réticulations pour le polypropylène.

Les résultats des tests combinés d'exposition-enfouissement révèlent essentiellement la diminution des propriétés mécaniques consécutive à l'action des intempéries (lumière comprise). Cette diminution est plus accentuée pour le polypropylène que pour le polyester, toutes choses égales par ailleurs.

L'enfouissement consécutif à une exposition n'entraîne pas d'augmentation des pertes de propriétés mécaniques. Ce qui signifie que les géotextiles ne sont pas sensibilisés par une exposition aux agressions ultérieures, ce qui pouvait être un risque à craindre. Les diminutions des propriétés mécaniques enregistrées sur les échantillons enfouis,mais non exposés préalablement, peuvent s'expliquer simplement par la prise en considération des contraintes de manipulation au moment de la mise en oeuvre et des prélèvements. Les échantillons exposés quant à eux, déjà manipulés pour les tests d'exposition à l'extérieur, seraient curieusement "moins affectés" par ces contraintes de mise en oeuvre et de prélèvements.

Enfin, il est intéressant de noter que les techniques de chromatographie en phase liquide peuvent être utilisées avec succès pour suivre la teneur des fibres de géotextiles en divers additifs tels que les barrières anti-oxydantes, anti-UV, stabilisants, plastifiants. Cette technique a été mise en oeuvre pour suivre la teneur en anti-oxydant (Irganox 1010) dans les fibres de polypropylène des échantillons de SODOCA. La courbe de la Fig. 5 montre la façon dont cet additif est détruit lors des tests de vieillissement complexe utilisés dans cette étude. Une exposition de 3 mois dans la région parisienne fait chuter la teneur en additif de 515 ppm à 74 ppm et ensuite à 52 ppm après un enfouissement subséquent de 6 mois (l'effet de lessivage dans les sols peut rendre compte de cette diminution de l'anti-oxydant).

Ces identifications et dosage des additifs par chromatographie liquide peuvent être très précieux pour l'optimisation des fibres et de leur protection contre les agressions photochimiques et thermiques. La permanence de ces additifs est peut-être une garantie de durabilité des géotextiles...

Table 1. Effet du vieillissement chimique et photochimique

BIDIM U44					
Echantillons	Résistance (daN/m)		Allong^t (%)		COOH 10^6 g.
Témoin	1950	(104)	80	(5)	43
Exposé 15 j. (Paris)	1950	(100)	80	(4,8)	43
Témoin + pH 12 (15 j.)	1747	(83)	81	(5)	49
Témoin + pH 12 (6 mois)	1952	(87)	72	(1,9)	56
Exposé + pH 12 (15 j.)	1771	(98)	76	(4,7)	52
Exposé + pH 12 (6 mois)	1871	(122)	66	(3,4)	56
Témoin + pH 3 (15 j.)	1754	(176)	79	(8,6)	46
Témoin + pH 3 (6 mois)	1945	(64)	78	(4,2)	50
Exposé + pH 3 (6 mois)	1918	(52)	82	(8,2)	55
Témoin + eau de mer (15 j.)	1767	(93,6)	81	(3,7)	53
Témoin + eau de mer (6mois)	1887	(108)	81	(3,3)	55
Exposé + eau de mer (6 mois)	1759	(146)	77	(4)	56

Les chiffres entre parenthèses correspondent à l'écart type.

Table 2. Effet du vieillissement chimique et photochimique

SODOCA AS 250					
Echantillons	Résistance (daN/m)		Allong^t (%)		Indice Visco.
Témoin	1810	(95)	101	(9,7)	109
Exposé 15 j. (Paris)	1821	(51)	104	(8,7)	122
Témoin + pH 12 (15 j.)	1819	(93)	94	(11)	141
Témoin + pH 12 (19 m.)	1766	(178)	95	(7)	-
Exposé + pH 12 (15 j.)	1764	(66)	97	(11)	128
Exposé + pH 12 (19 m.)	1891	(84)	100	(11)	-
Témoin + pH 3 (15 j.)	1717	(86)	98	(8)	130
Témoin + pH 3 (19 m.)	1909	(58)	94	(6)	-
Exposé + pH 3 (15 j.)	1870	(99)	101	(10)	139
Exposé + pH 3 (19 m.)	1863	(66)	100	(9)	-
Témoin + eau de mer { (15 j.)	1733	(102)	95	(6)	128
Témoin + eau de mer { (19 m.)	1894	(104)	99	(9)	-
Exposé + eau de mer { (15 j.)	1682	(92)	111	(9)	131
Exposé + eau de mer { (19 m.)	1848	(101)	100	(7)	-

Les chiffres entre parenthèses correspondent à l'écart type.

Table 3. Effets d'une exposition aux intempéries suivi d'un enfouissement

Temps d'exposition aux intempéries (Paris)										
		0			1 mois			8 mois		
	Temps d'enfouissement	Résistance (daN/m)	Allong^t (%)	COOH et (η)	Résistance (daN/m)	Allong^t (%)	COOH et (η)	Résistance (daN/m)	Allong^t (%)	COOH et (η)
BIDIM	0	1950	80	43	1795	64	54	1240	59	59
BIDIM	6 mois	1653	71	47	1648	59	53	1299	60	64
BIDIM	21 mois	1706	73	49	1704	60	54	1433	58	65
AS 420	0	2010	118	109	1529	106	122	1400	110	119
AS 420	6 mois	1456	126	129	1827	106	124	1222	84	113
AS 420	21 mois	1513	126	136	1729	105	132	142[illegible]	109	126

(η) = Indice de viscosité limite pour le polypropylène.

CONCLUSIONS

Le niveau de sévérité des tests simples de laboratoire pour étudier le vieillissement accéléré est en général mal ajusté pour simuler les contraintes réelles rencontrées par les géotextiles. Les tests de vieillissement accéléré permettront de classer des échantillons vis-à-vis d'une contrainte simple.

Si le niveau d'intensification du test laboratoire est abaissé (température ambiante par exemple), les géotextiles en polyester et polypropylène résistent très bien à ces tests, mais cette assertion n'est valable que pour la durée de l'expérimentation suivie et les résultats ne permettent pas de faire des prévisions de durabilité à long terme.

L'ajustement à des conditions d'essai réalistes, sera fonction de chaque type de géotextile et exigera de se référer à des gammes de géotextiles prélevés en ouvrages et ayant une histoire permettant d'identifier les causes des modifications de propriétés et de structure.

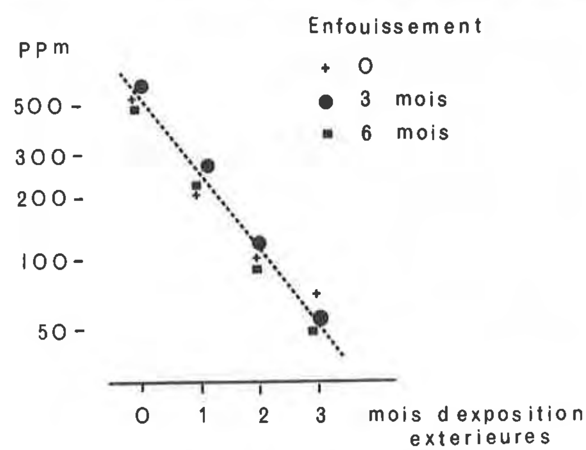


Fig. 5

Les géotextiles sont dégradés par des expositions prolongées aux intempéries (lumière comprise). Là encore des tests de vieillissement accélérés peuvent être adaptés pour mesurer la tenue des géotextiles à ce type de contrainte. L'approche n'est pas encore normalisée, plusieurs équipements et dispositifs sont utilisés pour l'exposition des géotextiles soit à des UV, soit à une lumière solaire artificielle. D'une manière générale la transposition des résultats à ces expositions artificielles, pour des prévisions de durabilité en exposition naturelle, est très délicate. Il faut toujours conduire de front et des expositions naturelles et des tests artificiels. Tous les producteurs de géotextiles, notamment ceux qui produisent des géotextiles en polypropylène, étudient très sérieusement ce problème. Certains (4), producteurs de gêotissus, ont déterminé à travers ces tests des temps de demi-durée de vie de leurs produits (temps nécessaire pour perdre 50 % d'une caractéristique, résistance à la rupture ou autre). Ces demi-durées de vie s'échelonnent de 300 à 1500 h pour des énergies d'irradiation de 18 à 75 Kly, soit respectivement 3 et 12 mois d'exposition extérieure dans des pays comme ceux de la France ou de l'Allemagne.

Le cas des géotextiles obtenu par filage direct (aiguilletés et thermosoudés) peut être étudié plus précisément. L'expérience a montré que des expositions prolongées, volontaires ou involontaires, aux intempéries entraînaient l'endommagement des géotextiles par ruptures des fibres et notamment des points de liage (point d'aiguilletage ou point de soudage) (cf. Fig. 6,7,8). Ces références permettent d'étalonner les tests de photovieillissement au laboratoire et de définir les limites de durées d'exposition extérieure à ne pas dépasser pour éviter les risques de tels ennuis pour ces types de géotextiles. Ces tests évidemment permettront aux producteurs d'optimiser leurs textiles en fonction d'exigences particulières dûes à un chantier ou à un climat.

Les essais d'enfouissement en terre végétale n'ont pas permis jusqu'ici de mettre en évidence des modifications particulières des propriétés des géotextiles étudiés : les modifications enregistrées semblent attribuables aux contraintes de manipulations au moment de la mise en oeuvre et du prélèvement, mais non à un quelconque vieillissement chimique ou biologique. Les pertes de résistance sont de l'ordre de 25 % maximum pour les géotextiles, et de 10 % maximum pour les fibres élémentaires. Pour interpréter ces différences, on peut suggérer que sous l'effet de l'enfouissement il s'est produit des modifications des intéractions fibres à fibres dans le géotextile aiguilleté par rapport à ce qu'elles étaient dans le témoin : les tests de traction sur géotextiles vieillis par enfouissement, révéleraient alors une moindre capacité de glissement des fibres les unes sur les autres...

Des expositions prolongées aux intempéries (3 mois) ne semblent pas sensibiliser particulièrement les géotextiles à des contraintes chimiques, biologiques, etc.. subséquentes, ceci dans les limites de durées de l'expérimentation réalisée (2ans).

Enfin, la tenue au fluage des géotextiles n'a pas été étudiée dans cet article. Certains producteurs font des études approfondies sur ce thème. Mais là encore, il apparaît difficile de concentrer l'échelle temps et d'accélérer le processus. Les résultats acquis sur 2 - 3 ans de tests, ne permettent pas d'extrapoler sans danger la tenue au fluage sur des dizaines, voire des centaines d'années. A partir d'un certain temps, les courbes de fluage qui possédaient une allure monotone peuvent parfois démarrer de nouveau pour des raisons certainement à rapprocher d'un vieillissement physique ou chimique du polymère fibreux.

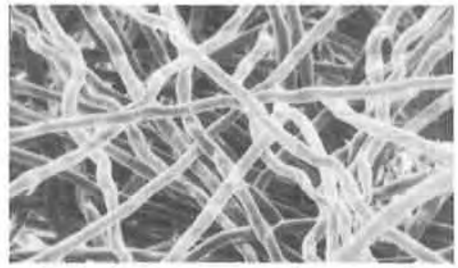


Fig. 6 Etat de surface d'un géotextile aiguilleté décohésionné par exposition extérieure prolongée.

Fig. 7 Rupture des points d'aiguilletage après exposition prolongée.

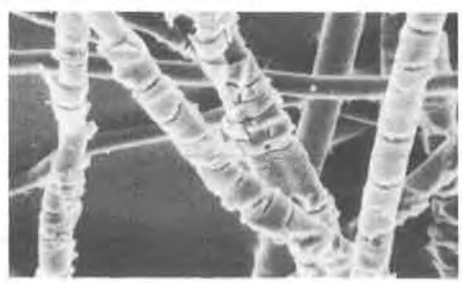


Fig. 8 Rupture des points de thermosoudage après exposition prolongée.

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APPENDIX

SECOND INTERNATIONAL CONFERENCE ON GEOTEXTILES
DEUXIEME CONGRES INTERNATIONAL DES GEOTEXTILES

LIST OF SYMBOLS
LISTE DE SYMBOLES (*)

1. GENERAL SYMBOLS
SYMBOLES GENERAUX

1.1 Dimension symbols
Symboles de dimension

Symbols used for the dimensions are:		Les symboles de dimension sont:
L :	length	longueur
M :	mass	masse
T :	time	temps
- :	dimensionless	sans dimension

1.2 Unit Symbols
Symboles d'unités

m	meter	mètre
m^2	square meter	mètre carré
m^3	cubic meter	mètre cube
mm	millimeter	millimètre
µm	micron	micron
g	gram	gramme
mg	milligram	milligramme
kg	kilogram	kilogramme
s	second	seconde
N	newton	newton
kN	kilonewton	kilonewton
Pa	pascal	pascal
kPa	kilopascal	kilopascal
MPa	megapascal	mégapascal
J	joule	joule
tex	tex	tex
L	liter	litre
^{o}	degree	degré
%	percent	pourcent
-	pure number	nombre pur

(*)This list of symbols has been prepared by J. P. Giroud.
Cette liste de symboles a été établie par J. P. Giroud.

The following relationships exist: Relations entre les unités ci-dessus:

1 Pa = 1 N/m^2

1 tex = 1 mg/m

1 J = 1 mN

1.3 Mathematical symbols
Symboles mathématiques

ln x	natural logarithm of x	logarithme naturel de x
lg x	logarithm of x base 10	logarithme décimal de x

1.4 Subscripts
Indices

Subindex / Indice	Applies to / Se rapporte à	
a	air or active (earth pressure)	air ou actif (poussée)
d	dry state	état sec
f	failure or final	rupture ou final
g	geotextile	géotextile
H	horizontal	horizontal
i	immediate or initial	immédiat ou initial
p	passive (earth pressure)	passif (butée)
r	radial	radial
s	solid particles	particules solides
sat	saturated	saturé
sec	secant	sécant
u	undrained conditions	conditions non drainées
V	vertical	vertical
w	water	eau
x, y	two orthogonal horizontal axes	deux axes orthogonaux horizontaux
z	vertical axis	axe vertical
o	at rest or initial conditions	au repos ou conditions initiales
1,2,3	principal directions	directions principales

1.5 Geometry and kinematics
Géométrie et cinématique

L	L	(m)	length	longueur
B	L	(m)	breadth	largeur
H	L	(m)	height, thickness	hauteur, épaisseur
D	L	(m)	depth	profondeur
z	L	(m)	vertical coordinate	abscisse verticale
d	L	(m)	diameter	diamètre

A	L^2	(m^2)	area	aire
V	L^3	(m^3)	volume	volume
t	T	(s)	time	temps
v	$L\ T^{-1}$	(m/s)	velocity	vitesse
g	$L\ T^{-2}$	(m/s^2)	acceleration due to gravity ($g = 9.81\ m/s^2$)	accélération de la pesanteur ($g = 9.81\ m/s^2$)

1.6 Stress and strain
Contraintes et déformations

u	$ML^{-1}T^{-2}$	(kPa)	pore pressure	pression interstitielle
			pressure of the fluid in the voids of a porous medium (soil, geotextile)	*pression du fluide dans les vides d'un milieu poreux (sol, géotextile)*
σ	$ML^{-1}T^{-2}$	(kPa)	normal stress	contrainte normale
σ'	$ML^{-1}T^{-2}$	(kPa)	effective normal stress	contrainte normale effective
			normal stress transmitted by intergranular contacts ($\sigma' = \sigma - u$ for saturated soils)	*contrainte normale transmise par contacts intergranulaires ($\sigma = \sigma - u$ pour les sols saturés)*
τ	$ML^{-1}T^{-2}$	(kPa)	shear stress	contrainte de cisaillement
$\sigma_1, \sigma_2, \sigma_3$	$ML^{-1}T^{-2}$	(kPa)	principal stresses	contraintes principales
ε	-	(-, %)	strain, élongation	déformation, élongation
			change in length per unit length in a given direction (called elongation if expressed in % and applied to a geotextile)	*variation relative de longueur dans une direction donnée (appelée élongation lorsqu'elle est exprimée en % et appliquée à un géotextile)*
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	-	(-,%)	principal strains	déformations principales

1.7 Hydraulic parameters
Paramètres hydrauliques

h	L	(m)	hydraulic head or potential	charge hydraulique ou potentiel hydraulique
			sum of pressure height (u / γ_w) and geometrical height (z) above a given reference level	*somme de la hauteur piézométrique (u / γ_w) et de la hauteur géométrique (z) au-dessus d'un niveau de référence*
q	L^3T^{-1}	(m^3/s)	rate of discharge	débit
			volume of water seeping through a given area per unit of time	*volume d'eau percolant à travers une section donnée d'un sol, par unité de temps*
v	$L\ T^{-1}$	(m/s)	discharge velocity	vitesse d'écoulement
			rate of discharge per total unit area perpendicular to direction of flow	*débit qui s'écoule à travers une section totale unitaire du milieu, perpendiculaire à la direction de l'écoulement*

i	-	(-)	hydraulic gradient	gradient hydraulique
			loss of hydraulic head per unit length in direction of flow	*perte de charge hydraulique par unité de longueur dans la direction de l'écoulement*
j	$ML^{-2}T^{-2}$	(kN/m^3)	seepage force per unit volume	force de filtration (ou d'écoulement) par unité de volume
			force per unit volume of a porous medium generated by action of the seeping fluid upon the solid elements of the porous medium ($j = i\gamma_w$)	*force volumique exercée sur les éléments solides d'un milieu poreux par un fluide en écoulement:* ($j = i.\gamma_w$)

2. PROPERTIES OF FLUIDS

PROPRIETES DES FLUIDES

ρ_w	ML^{-3}	(kg/m^3)	density of water	masse volumique de l'eau
γ_w	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of water	poids volumique de l'eau
η_w	$ML^{-1}T^{-1}$	(kg/ms)	dynamic viscosity of water	viscosité dynamique de l'eau

NOTE: Instead of w , use any other appropriate subscript of other fluids (e.g.: η_a, dynamic viscosity of air)

A la place de w, on peut utiliser d'autres indices relatifs à d'autres fluides que l'eau (par exemple: η_a, viscosité dynamique de l'air)

3. PROPERTIES OF SOILS
PROPRIETES DES SOLS

3.1 Solid particles and their distribution
Particules solides et leur distribution

ρ_s	ML^{-3}	(kg/m^3)	density of solid particles	masse volumique des particules solides
			ratio between mass and volume of solid particles	*quotient de la masse des particules solides par leur volume*
γ_s	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of solid particles	poids volumique des particules solides
			weight of solid particles per unit volume	*quotient du poids des particules solides par leur volume*
d	L	(mm)	particule diameter	diamètre de particule
			particle size as determined by sieve analysis or wet mechanical analysis	*taille de particule déterminée dans l'analyse granulométrique par tamisage ou sédimentométrie*
d_n	L	(mm)	n percent-diameter	diamètre à n pourcent
			diameter corresponding to n percent by weight of finer particles	*diamètre correspondant a un tamisat de n pourcent sur la courbe granulométrique (n% des particules ont des dimensions inférieures à ce diamètre)*

C_U	-	(-)	uniformity coefficient defined as : d_{60}/d_{10}	coefficient d'uniformité defini par : d_{60}/d_{10}

3.2 Physical properties of soils
Propriétés physiques des sols

ρ	ML^{-3}	(kg/m^3)	density of soil *ratio between total mass and total volume of soil*	masse volumique du sol *quotient de la masse totale du sol par son volume*
γ	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of soil *ratio between total weight and total volume of soil*	poids volumique du sol *quotient du poids total du sol par son volume*
ρ_d	ML^{-3}	(kg/m^3)	density of dry soil *ratio between mass of solid particles and total volume of soil*	masse volumique du sol sec *quotient de la masse des particules solides par le volume total du sol*
γ_d	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of dry soil *ratio between weight of solid particles and volume of soil*	poids volumique du sol sec *quotient du poids des particules solides par le volume total de sol*
ρ_{sat}	ML^{-3}	(kg/m^3)	density of saturated soil *ratio between total mass and total volume of completely saturated soil*	masse volumique du sol saturé *quotient de la masse totale du sol complètement saturé par son volume total*
γ_{sat}	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of saturated soil *ratio between total weight and total volume of completely saturated soil*	poids volumique du sol saturé *quotient du poids total du sol complètement saturé par son volume total*
ρ'	ML^{-3}	(kg/m^3)	density of submerged soil *difference between density of soil and density of water*	masse volumique du sol déjaugé *différence entre la masse volumique du sol et la masse volumique de l'eau*
γ'	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of submerged soil *difference between unit weight of soil and unit weight of water*	poids volumique du sol déjaugé *différence entre le poids volumique du sol et le poids volumique de l'eau*
e	-	(-)	void ratio *ratio between volume of voids and volume of solid particles*	indice des vides *rapport entre le volume des vides et le volume des particules solides*
n	-	(-. %)	porosity *ratio between volume of voids and total volume of soil*	porosité *rapport entre le volume des vides et le volume total du sol*

w	–	(-.%)	water content	teneur en eau
			ratio between weight of pore water and weight of solid particles (expressed in percentage)	*rapport entre le poids de l'eau interstitielle et le poids des grains solides (exprimé en pourcents)*
S_r	–	(-. %)	degree of saturation	degré de saturation
			ratio between volume of pore water and volume of voids	*rapport entre le volume de l'eau interstitielle et le volume des vides*

3.3 Permeability of soils
Perméabilité des sols

k	LT^{-1}	(m/s)	coefficient of permeability (or hydraulic conductivity)	coefficient de perméabilité (ou conductivité hydraulique)
			ratio between discharge velocity and corresponding hydraulic gradient (v / i)	*quotient de la vitesse d'écoulement par le gradient hydraulique correspondant (v / i)*

3.4 Mechanical properties of soils
Propriétés mécaniques des sols

E	$ML^{-1}T^{-2}$	(kPa)	deformation modulus	module de déformation
			ratio between a given normal stress change and the strain change in the same direction (all other stresses being constant)	*quotient de la variation d'une contrainte principale par la déformation obtenue dans la même direction, les autres contraintes restant inchangées.*
υ	–	(-)	Poisson's ratio	coefficient de Poisson
			ratio between strain changes perpendicular to and in the direction of a given uniaxial stress change	*rapport entre les deux déformations respectivement perpendiculaire et parallèle à la direction d'une sollicitation uniaxiale*
k_s	$ML^{-2}T^{-2}$	(kN/m^3)	modulus of subgrade reaction	module de réaction
			ratio between change of vertical stress on a rigid plate and corresponding change of vertical settlement of the plate	*quotient de la variation de la contrainte verticale sur une plaque rigide par la variation correspondante de tassement vertical de la plaque*
E_{oed}	$ML^{-1}T^{-2}$	(kPa)	oedometric modulus	module oedométrique
C_c	–	(-)	compression index	indice de compression
			slope of virgin compression curve in a semi-logarithmic plot "effective pressure-void ratio": $Cc = -\Delta e / \Delta lg\sigma'$	*pente de la courbe de compression vierge dans un diagramme semi-logarithmique "contrainte effective-indice des vides": $Cc = -\Delta e / \Delta lg\sigma'$*
c_v	L^2T^{-1}	(m^2/s)	coefficient of consolidation	coefficient de consolidation
T_v	–	(-)	time factor	facteur temps
			defined as $T_v = t\,c_v / d^2$, t being the time elapsed since application of a change in total normal stress	*défini par $T_v = t\,c_v / d^2$, t étant le temps écoulé depuis l'application d'une variation de contrainte normale totale*

τ_f	$ML^{-1}T^{-2}$	(kPa)	shear strength	résistance au cisaillement
			shear stress at failure in rupture plane through a given point	*contrainte de cisaillement, lors de la rupture, dans le plan de rupture en un point donné*
c'	$ML^{-1}T^{-2}$	(kPa)	effective cohesion	cohésion effective
ϕ'	-	(^{o})	effective angle of internal friction	angle de frottement effectif
			shear strength parameter with respect to effective stresses. Defined by the equation: $\tau_f = c' + \sigma' \tan \phi'$	*paramètre de résistance au cisaillement en contraintes effectives, défini par l'équation :* $\tau_f = c' + \sigma' \operatorname{tg} \phi'$
c_u	$ML^{-1}T^{-2}$	(kPa)	apparent cohesion	cohésion apparente
ϕ_u	-	(^{o})	apparent angle of internal friction	angle de frottement apparent
			shear strength parameters with respect to total stresses. Defined by the equation: $\tau_f = c_u + \sigma \tan \phi_u$. *In undrained situation, with saturated cohesion soils,* c_u *is also called undrained shear strength*	*paramètres de résistance au cisaillement en contraintes totales, défini par l'équation:* $\tau_f = c_u + \sigma \operatorname{tg} \phi_u$. Pour les sols *cohérents saturés en sollicitation non drainée,* c_u *est appelé également cohésion non drainée*
CBR	-	(-)	California Bearing Ratio	indice portant californien
q_c	$ML^{-1}T^{-2}$	(kPa)	static point resistance (or cone resistance)	résistance de pointe statique (ou résistance de cône)
			average pressure acting on the conical point in the standard static penetration test	*pression moyenne agissant sur la pointe conique dans l'essai normalisé de pénétration statique*
f_s	$ML^{-1}T^{-2}$	(kPa)	local side friction	frottement latéral unitaire
			average unit side friction acting on the friction sleeve in the standard static cone penetration test	*frottement latéral par unité de surface du manchon de frottement dans l'essai normalisé de pénétration statique au cône*
N	-	(-)	SPT blow count	nombre de coups SPT
			standardized result of the Standard Penetration Test (number of blows for 30 cm)	*résultat normalisé de l'essai SPT (nombre de coups pour 30 cm)*
p_l	$ML^{-1}T^{-2}$	(kPa)	pressuremeter limit pressure	pression limite pressiométrique
			limit pressure defined in the standard Ménard pressuremeter test	*pression limite définie dans l'essai pressiométrique normal Ménard*
E_M	$ML^{-1}T^{-2}$	(kPa)	pressuremeter modulus	module pressiométrique
			conventional modulus defined in the standard Ménard pressuremeter test.	*module conventionnel défini dans l'essai pressiométrique normal Ménard*

3.5 Consistency of soils
Consistance des sols

w_L	–	(-.%)	liquid limit	limite de liquidité
			water content of a remolded soil at transition between liquid and plastic states (determined by a standard laboratory test)	*teneur en eau d'un sol remanié au point de transition entre les états liquide et plastique (déterminée par un essai normalisé de laboratoire)*
w_p	–	(-, %)	plastic limit	limite de plasticité
			water content of a remolded soil at transition between plastic and semi-solid states (determined by a standard laboratory test)	*teneur en eau d'un sol remanié au point de transition entre les états plastique et solide avec retrait (déterminée par un essai normalisé de laboratoire)*
w_s	–	(-, %)	shrinkage limit	limite de retrait
			maximum water content at which a reduction of water content will not cause a decrease in volume of the soil mass	*teneur en eau maximale pour laquelle une réduction de teneur en eau ne cause plus de diminution de volume du sol*
I_p	–	(-,%)	plasticity index	indice de plasticité
			difference between liquid and plastic limits	*différence entre les limites de liquidité et de plasticité*
I_L	–	(-,%)	liquidity index	indice de liquidité
			defined as $(w - w_p) / I_p$	*défini par* $(w - w_p) / I_p$
I_C	–	(-,%)	consistency index	indice de consistance
			defined as $(w_L - w) / I_p$	*défini par* $(w_L - w) / I_p$
e_{max}	–	(-)	void ratio in loosest state	indice des vides dans l'état le plus lâche
			maximum void ratio obtainable by a standard laboratory procedure	*maximum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
e_{min}	–	(-)	void ratio in densest state	indice des vides dans l'état le plus dense
			maximum void ratio obtainable by a standard laboratory procedure	*minimum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
I_D	–	(-)	density index	indice de densité
			defined as $(e_{max} - e) / (e_{max} - e_{min})$	*défini par* $(e_{max} - e) / (e_{max} - e_{min})$

4. PROPERTIES OF GEOTEXTILES
PROPRIETES DES GEOTEXTILES

4.1 Solid elements and their distribution
Eléments solides et leur distribution

ρ_f	ML^{-3}	(kg/m^3)	density of fibers or filaments	masse volumique des fibres ou des filaments
d_f	L	(μm)	diameter of fibers or filaments	diamètre des fibres ou filaments

λ	ML^{-1}	(tex)	linear density of yarns, fibers or filaments *mass per unit length of yarns, fibers or filaments*	masse linéique (titre) des fils, fibres ou filaments *masse par unité de longueur des fils, fibres ou filaments.*
O_n	L	(mm, μm)	n-percent opening size *opening size corresponding to n-percent of finer openings (as measured by a test to be defined, such as sieving glass beads) (e.g. O_{95}, opening size corresponding to 95% of finer openings, sometimes called "Equivalent Opening Size")*	ouverture à n pourcent *dimension d'ouverture telle que n pourcent d'ouvertures soient plus petites (préciser l'essai utilisé, par exemple tamisage de billes de verre) (par exemple: O_{95}, dimension de l'ouverture telle que 95% des ouvertures soient plus petites, quelquefois appelée "Dimension d'Ouverture Equivalente")*

Note: Standard sieve numbers vary from one country to another. Opening sizes expressed by a standard sieve number would not be understood by most of the readers. Consequently, authors are strongly requested to express opening size in millimeters (mm) or microns (μm). Standard sieve numbers can be given in parenthesis after the value in mm or μm.

Note: Les numéros standards de tamis diffèrent d'un pays à l'autre. Les dimensions d'ouverture exprimées par un numéro standard de tamis ne signifieraient rien pour la plupart des lecteurs. En conséquence, il est fortement conseillé aux auteurs d'exprimer les dimensions d'ouverture en millimètres (mm) ou en microns (μm). Le numéro standard de tamis peut être donné entre parenthèses après la valeur en mm ou μm.

4.2 Physical properties of a geotextile
Propriétés physiques d'un géotextile

μ	ML^{-2}	(kg/m^2, g/m^2)	mass per unit area *ratio between mass and area of geotextile*	masse surfacique *masse par unité de surface d'un géotextile*
e	-	(-)	void ratio *ratio between volume of voids and volume of solid elements of a geotextile*	indice des vides *quotient du volume des vides par le volume des éléments solides d'un géotextile*
n	-	(-, %)	porosity *ratio between volume of voids and total volume of geotextiles*	porosité *quotient du volume des vides par le volume total d'un géotextile*
T_g	L	(mm)	thickness of geotextile	épaisseur d'un géotextile

4.3 Hydraulic properties of a geotextile
Propriétés hydrauliques d'un géotextile

k_n	LT^{-1}	(m/s)	coefficient of normal permeability of a geotextile	coefficient de perméabilité normale d'un géotextile
k_p	LT^{-1}	(m/s)	coefficient of permeability in the plane of a geotextile	coefficient de perméabilité dans le plan d'un géotextile
Ψ	T^{-1}	(s^{-1})	permittivity of a geotextile ($\Psi = k_n/H_g$)	permittivité d'un géotextile ($\Psi = k_n/H_g$)
θ	L^2T^{-1}	(m^2/s)	transmissivity of a geotextile ($\theta = k_p/H_g$)	transmissivité d'un géotextile ($\theta = k_p\ H_g$)

4.4 Mechanical properties of a geotextile
Propriétés mécaniques d'un géotextile

α_ε	MT^{-2}	(kN/m)	force per unit width of the geotextile at a given elongation ε (e.g. α_{30} is the force per unit width of the geotextile at 30% elongation)	force par unité de largeur du géotextile pour une élongation ε donnée (par exemple: α_{30} est la force par unité de largeur du géotextile pour une élongation de 30%)
α_f	MT^{-2}	(kN/m)	force per unit width of the geotextile at failure	force par unité de largeur du géotextile à la rupture
J	MT^{-2}	(kN/m)	"modulus" of the geotextile	"module" du géotextile
		Note:	The "modulus" of a geotextile is defined as a force per unit width while modulus is usually defined as a force per unit area.	Note: Le "module" d'un géotextile est défini comme une force par unité de largeur, alors qu'un module represente généralement une force par unité de surface.
J_i	MT^{-2}	(kN/m)	initial (tangent) "modulus" of the geotextile	"module" (tangent) initial du géotextile
$J_{t\varepsilon}$	MT^{-2}	(kN/m)	tangent "modulus" of the geotextile at elongation ε (e.g. J_{t30} is the tangent "modulus" of the geotextile at 30% elongation)	"module" tangent du géotextile pour une élongation ε donnée (par exemple J_{t30} est le "module" tangent du géotextile pour une élongation de 30%)
$J_{sec\ \varepsilon}$	MT^{-2}	(kN/m)	secant "modulus" of the geotextile between 0 and elongation ε (e.g. $J_{sec\ 30}$ is the secant "modulus" of geotextile between 0 and 30% elongation)	"module" sécant du géotextile entre 0 et l'élongation ε (par exemple, $J_{sec\ 30}$ est le "module" sécant du géotextile entre 0 et 30% d'élongation)
υ_g	-	(-)	Poisson's ratio of a geotextile	coefficient de Poisson d'un géotextile
ζ_t	M^2T^{-2}	(N/tex)	tenacity of a thread *quotient of the tensile force at failure of a thread to its linear density*	tenacité d'un fil *quotient de la force de traction à la rupture d'un fil par sa masse linéique*
ζ_g	M^2T^{-2}	(J/g)	tenacity of a geotextile *quotient of the force per unit width at failure of a geotextile to its mass per unit area*	tenacité d'un géotextile *quotient de la force par unité de largeur d'un géotextile à la rupture par sa masse surfacique*

(Note: 1 J/g = 1000 N/tex)

F_G	MLT^{-2}	(N,kN)	breaking force of geotextile as measured in grab test	résistance du géotextile dans l'essai d'arrachement
F_P	MLT^{-2}	(N, kN)	breaking force of geotextile in a puncture test (to be defined)	résistance du géotextile à la perforation (préciser l'essai utilisé)
F_T	MLT^{-2}	(N, kN)	breaking force of geotextile in a tear test (to be defined)	résistance du géotextile à la déchirure (préciser l'essai utilisé)
p_B	$ML^{-1}T^{-2}$	(kPa, MPa)	bursting pressure of a geotextile.	résistance du géotextile à l'éclatement

5. PRACTICAL PROBLEMS
PROBLEMES PRATIQUES

5.1 General
Généralités

FS	-	(-)	Factor of safety	Coefficient de sécurité

5.2 Earth pressure
Pression des terres

δ	-	(°)	angle of wall friction	angle de frottement sol-mur
			angle of friction between wall and adjacent soil	*angle de frottement entre le mur et le sol adjacent*
a	$ML^{-1}T^{-2}$	(kPa)	wall adhesion	adhésion sol-mur
			adhesion between wall and adjacent soil	*adhésion entre le mur et le sol adjacent*
K_a, K_p	-	(-)	active and passive earth pressure coefficients	coefficients de poussée et de butée des terres
			dimensionless coefficients used in expressions for active and passive earth pressure	*coefficients sans dimension intervenant dans les expressions de poussée et de butée*
K_o	-	(-)	coefficient of earth pressure at rest	coefficient de pression des terres au repos
			ratio of lateral to vertical effective principal stress in the case of no lateral strain and a horizontal ground surface	*rapport entre les contraintes effectives horizontale et verticale à déformation horizontale nulle et lorsque la surface libre du sol est horizontale*

5.3 Foundations and embankments
Fondations et remblais

B	L	(m)	breadth of foundation or embankment	largeur de la fondation ou du remblai
L	L	(m)	length of foundation or embankment	longueur de la fondation ou du remblai
D	L	(m)	depth of foundation or base of embankment beneath ground	profondeur de la fondation ou de la base du remblai au-dessous du niveau du terrain
s	L	(m)	settlement	tassement
U	-	(-, %)	degree of consolidation	degré de consolidation
			ratio of settlement at a given time to final settlement	*quotient du tassement à un temps donné par le tassement final*

5.4 Slopes
Pentes

H	L	(m)	vertical height of slope	hauteur verticale du talus
D	L	(m)	depth below toe of slope to hard stratum	profondeur du substratum rigide sous le pied du talus
β	-	(°)	angle of slope to horizontal	angle d'inclinaison du talus avec l'horizontale

Conversion factors
Facteurs de conversion

Conversion factors between the English system and SI are as follows:

Length, area, volume

1 mil	= 25.4 microns (μm)
1 inch	= 25.4 millimeters (mm)
1 foot	= 0.305 meter (m)
1 yard	= 0.914 meter (m)
1 square foot	= 9.29×10^{-2} square meters (m^2)
1 acre	= 4047 square meters (m^2)
1 cubic foot	= 2.83×10^{-2} cubic meters (m^3)
1 gallon	= 3.79×10^{-3} cubic meters (m^3)
1 gallon	= 3.79 liters (L)

Mass

1 ounce	= 28.3 grams (g)
1 pound	= 0.454 kilogram (kg)
1 ton (US)	= 907 kilograms (kg)
1 long ton (GB)	= 1016 kilograms (kg)

Linear Density

1 denier	= 0.111 milligram per meter (mg/m)
1 denier	= 0.111 tex (tex)

Mass per unit area

1 ounce per square yard	= 33.9 gram per square meter (g/m^2)
1 ounce per square yard	= 3.39×10^{-2} kilogram per square meter (kg/m^2)

Density

1 pound (mass) per cubic foot	= 16.0 kilograms per cubic meter (kg/m^3)

Force

1 pound	= 4.45 newtons (N)
1 kip	= 4.45 kilonewtons (kN)

Force per unit width or per unit length

1 pound per inch	= 175 newtons per meter (N/m)
1 pound per inch	= 0.175 kilonewton per meter (kN/m)

Stress, pressure

1 pound per square inch	= 6895 pascals (Pa)
1 pound per square inch	= 6.89 kilopascals (kPa)
1 pound per square foot	= 47.9 pascals (Pa)
1 kip per square foot	= 47.9 kilopascals (kPa)
1 ton (US) per square foot	= 95.8 kilopascals (kPa)

Unit weight or reaction modulus

1 pound (force) per cubic foot	= 157 newtons per cubic meter (N/m^3)
1 pound (force) per cubic foot	= 0.157 kilonewton per cubic meter (kN/m^3)

Velocity, Coefficient of Permeability

1 centimeter per second	= 0.010 meter per second (m/s)

Flow rate

1 gallon per minute	= 6.31×10^{-2} liter per second (L/s)
1 gallon per minute	= 6.31×10^{-5} cubic meter per second (m^3/s)
1 cubic foot per second	= 2.83×10^{-2} cubic meter per second (m^3/s)
1 cubic foot per minute	= 0.472 liter per second (L/s)
1 cubic foot per minute	= 4.72×10^{-4} cubic meter per second (m^3/s)
1 gallon per day	= 4.38×10^{-5} liter per second (L/s)

Dynamic viscosity

1 poise	= 0.100 kilogram per meter-second (kg/sm)
1 pound (force) second per square foot	= 0.478 kilogram per meter-second (kg/sm) (Note: 1 kg/sm = 1 Pa.s)

Gravity

g = 32.2 feet per square second	= 9.81 meters per square second (9.81 m/s^2)

U. S. Sieve Designation	Opening Size	
	mm	µm
200	0.075	75
170	0.090	90
140	0.106	106
120	0.125	125
100	0.150	150
80	0.180	180
70	0.212	212
60	0.250	250
50	0.300	300
45	0.355	355
40	0.425	425
35	0.500	500
30	0.600	600
25	0.710	710
20	0.850	850
18	1.000	
16	1.18	
14	1.40	
12	1.70	
10	2.00	
8	2.36	
7	2.80	
6	3.35	
5	4.00	
4	4.75	

Notes

Notes

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Le volume IV (à paraître en janvier 1983) comprend: les comptes rendus des sessions d'ouverture et de clôture, les transcriptions des questions et réponses, les errata et un index des auteurs et titres de communications.

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COMPTES-RENDUS

Second International Conference on Geotextiles

Deuxième Congrès International des Géotextiles

August 1–6, 1982
1–6 Août, 1982
MGM Grand Hotel
Las Vegas, Nevada, U.S.A.

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City of Southampton, Southampton, UK
JOHNSON, P. and RITSON, R.
Portsmouth Polytechnic, Portsmouth, UK
PETLEY, D.
University of Warwick, Warwick, UK

Behavior of Fabric Reinforced Soil Walls

Comportement des murs en terre armée avec des géotextiles

Metal is used extensively in the construction of reinforced soil walls, but concern about corrosion under certain environmental conditions has prompted the development of alternative reinforcement systems. One such alternative is the "Websol" System of soil reinforcement. The system is based on the use of "Paraweb" polyester fibre strips and "Terram" non-woven fabric. Instrumentation has been developed to monitor the behaviour of three soil walls constructed using the Websol System. In 1979 instruments were installed during the reconstruction of a river bank at Southampton, England. In 1980 a new harbour wall was instrumented in Jersey, Channel Islands. Monitoring of this wall is still continuing. In 1981-82 an experimental wall was constructed at Portsmouth, England and monitoring of the instruments is proceeding. In this paper details of the field instrumentation and observations to date are presented. The early results suggest that the behaviour of the structures is somewhat at variance with theoretical predictions.

Le métal s'emploie beaucoup pour la construction des murs en terre renforcés, mais à cause de la corrosion sous certaines conditions d'environnement, on a dévelopé des systèmes alternatifs de renforcer. Un projet de ce geuve c'est le système "Websol" de renforcer la terre. Ce système s'est basé sur l'emploi de bandes de fibres de Polyester et d'étoffe non-tissée. On a dévelopé l'instrumentation pour contrôler le fonctionnement des trois murs en terre construits selon le système Websol. En 1979 les instruments étaient installes pendant la reconstruction d'une rive à Southampton, Angleterre. En 1980 on a installé des instruments dans un nouveau mur de quai à Jersey, les Îles de la Manche. Le contrôle de ce mur continue toujours. En 1981-82 on a construit à titre d'essai un mur à Portsmouth, Angleterre et on est entrain de contrôler les instruments. Dans cet exposé on donne les détails de l'instrumentation sur le terrain et les observations des derniers progre's. Les premiers résultats suggèvent que le fonctionnement des structures est assez en désaccord des prédictions théoriques.

INTRODUCTION

The technique of reinforced soil has seen many applications over the past decade, but has primarily been used for the construction of retaining walls. Extensive use has been made of metal reinforcement in these walls, but this can lead to corrosion problems under certain environmental conditions. To overcome this difficulty alternative reinforcement systems have been developed. One such alternative is the "Websol" System of soil reinforcement. This system comprises reinforced concrete facing panels, "Paraweb" polyester fibre strips, "Terram" nonwoven fabric and suitable fill material (Fig. 1). Soil walls constructed with these types of reinforcing materials are likely to behave differently to those using metal reinforcement. To monitor the behaviour of fabric reinforced structures, instruments have been developed and installed in three walls, each constructed using the Websol System.

In 1979 a 4 m high wall was constructed in a tidal river environment at Southampton, England. Instruments were installed to monitor the forces at the connections of the reinforcement and the facing panels, and to measure the strain along the reinforcement. In 1980 a new harbour wall, 8 m high, was constructed in Jersey, Channel Islands. The instrumentation used was similar to that installed at Southampton with the addition of earth pressure cells and inclinometer tubes. In 1981/82 an experimental wall, 2½ m high, has been built at the Geotechnics Field Centre, Portsmouth Polytechnic, England. Several different reinforcement configurations are being monitored. Pull-out tests will be carried out and part of the wall will be tested to failure.

Fig. 1 Jersey wall under construction. Rock fill being placed over Paraweb and Terram

1. SOUTHAMPTON WALL

In 1979 a reinforced soil retaining wall was constructed to form part of an outfall structure on the bank of the River Itchen at Southampton. The outfall site is reclaimed land and the sub-soil consists of about 5 m of soft organic clay. The tidal range is about 4 m. The retaining wall was constructed on a 600 mm thick fabric reinforced raft foundation, consisting of three layers of gravel rejects separated by Terram 2000 sheet. The reinforced soil retaining wall, 4 m high and 20 m long, was constructed using the Websol System which was selected because of the environmental conditions. Single size 15 mm gravel aggregate was used as a free-draining fill material in the lower part of the wall. For the upper part graded crushed rock was specified. Reinforcement of the wall was provided by 6 m long Paraweb strips and Terram 1000 sheet.

In the Websol System the Paraweb reinforcing strips are attached to the facing panels using mild steel anchorage pins, or toggle bars. Each facing panel has a number of anchorage points, each consisting of two loops through which the horizontal toggle bar is located. In order to measure the force on the Paraweb connections at the facing panel some of the standard toggles were replaced by specially fabricated strain gauged toggle bars (Fig. 2). These special toggles were commercially produced shear transducers using foil strain gauges. Five were installed on one large (approximately 2½ m square) facing panel - one in each corner and one near the centre. The cables from the end of each special toggle bar were taken into a vertical plastic pipe, set close behind the facing panels, and thence to the surface.

Monitoring of the toggle loads took place for many months after construction. Although the average load remained fairly constant the distribution across the panel changed slowly with time. This trend continued for some time after construction (Fig. 3) and was most marked immediately after the first spring tides. Readings taken at high and low spring tides showed a small consistent difference, which was less marked during neap tides.

A magnet extensometer system has been developed to measure the strain distribution in the Paraweb strips.

Fig. 2 Strain gauged toggle bar and magnet extensometer used for Southampton wall

Small magnets are placed in a plastic housing and bolted through the strip prior to construction. Twelve magnets were bolted to each of eight Paraweb strips. A long 12 mm diameter plastic access pipe was placed over each strip and located through each magnet housing (Fig. 2). A special probe, consisting of multicore cable and twelve Reed switches, can then be inserted in each pipe in turn to locate the magnet positions. Typical results from the magnet extensometer system (Fig. 4) show a considerable amount of scatter.

To monitor the tidal draw-down effects within the wall vertical perforated standpipes were installed. Readings showed that the water level within the wall was always horizontal and at the same level as the tide.

Settlement was monitored by taking levels on top of the facing panels. The line of panels settled between 40 and 80 mm during the first six months, with most of this occurring in the initial three months.

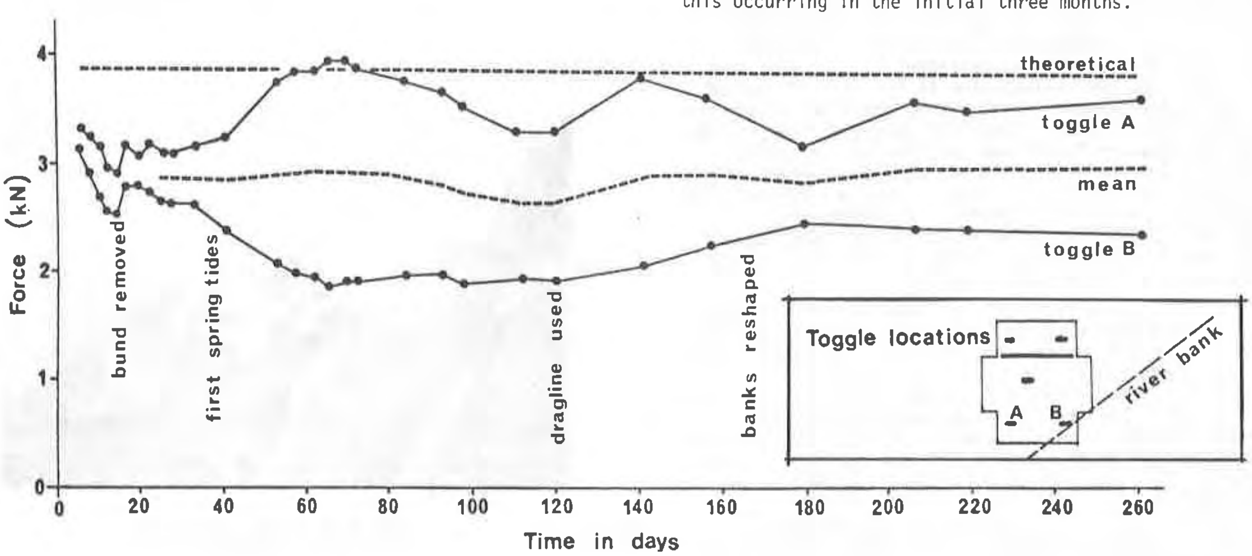

Fig. 3 Toggle loads at low tide, Southampton wall.

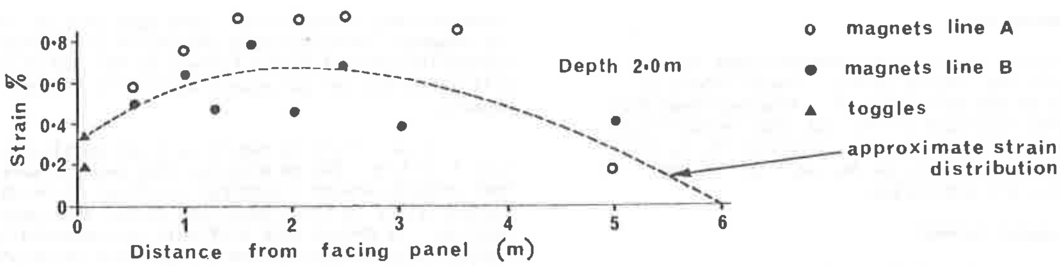

Fig. 4 Magnet extensometer results, Southampton wall

2. JERSEY WALL

In 1980 the opportunity arose to instrument a much larger wall at St. Helier, Jersey, Channel Islands. This new harbour wall is approximately 250 m long, 8 m high and varies in width from 7 m at the base to 5 m at the top. The tidal range here is about 11 m. The method of construction was the Websol System of soil reinforcement, which was offered as an alternative to the traditional rock fill bund usually used in the harbour. The wall is founded on an imported rock fill embankment. The same rock has been used as fill material in the reinforced soil wall. The seaward (front) face of the wall comprises reinforced concrete panels, and the landward (rear) face utilizes temporary plywood panels (Fig. 1). It is intended that landfill will abut the rear of the wall within a few years.

A complete vertical section of the wall was instrumented (Fig. 5). This section of the wall is 8 m (5 No. facing panels) in height and 2 m (1 No. panel) wide. Monitoring of the instruments has continued at regular intervals since installation and is still proceeding. The instrumentation was a development of that used at Southampton and consisted of the following:-

(i) Strain gauged toggle bars attached to the facing panels,
(ii) Earth pressure cells buried within the rock fill,
(iii) Inclinometer tubes sited within the fill,
(iv) Strain gauged Paraweb,
(v) Magnet extensometers bolted to Paraweb.

(i) Strain gauged toggle bars

In the Jersey Wall each facing panel has six anchorage points. In the instrumented section of 5 No. panels it was proposed that each of the 30 No. standard toggle bars would be replaced by specially fabricated strain gauged toggle bars. This would enable the loads on each toggle and facing panel to be assessed. The commercially produced toggle bars used in the Southampton wall were very expensive, and it was decided to fabricate similar special toggles in the workshops at Portsmouth Polytechnic. Each special toggle bar is 200 mm long and fabricated from 50 mm square bars. Each bar was carefully cut and drilled to provide mounting surfaces for strain gauges and associated wiring. The gauges and wiring were protected with a suitable rubber compound. The complete toggle bars were blast cleaned before being finally protected with the various coats of epoxy paint and polyurethane specified for the standard toggles. Although the original intention had been to install 30 No. special toggles, difficulties encountered on site meant that it was possible to install only 25 No. The cables from the end of each toggle bar were taken into a vertical 100 mm diameter plastic pipe, set close behind the facing panels. Monitoring of the strain gauged toggle bars has been carried out regularly during construction and at regular intervals since installation. At the present time 24 of the toggle bars are still functioning satisfactorily. The toggle bar results obtained to date, taken at various states of the tide, suggest that the performance of the structure is somewhat at variance with the predicted behaviour (Fig. 6).

(ii) Earth pressure cells

300 mm diameter oil filled earth pressure cells were installed within the rock fill to monitor pressures at various levels (Fig. 7). A group of 5 No. cells were placed towards the middle of the wall at a depth of about 7 m. A group of 3 No. cells were installed at a depth of about 4 m. Single cells were placed immediately behind the facing panels at depths of 4 m and 7 m. The cables from the cells were extended horizontally to a 75 mm diameter vertical plastic tube placed behind the facing panels, and thence to the surface. Although with one exception the pressure cells seem to be working satisfactorily the results obtained to date are not very encouraging.

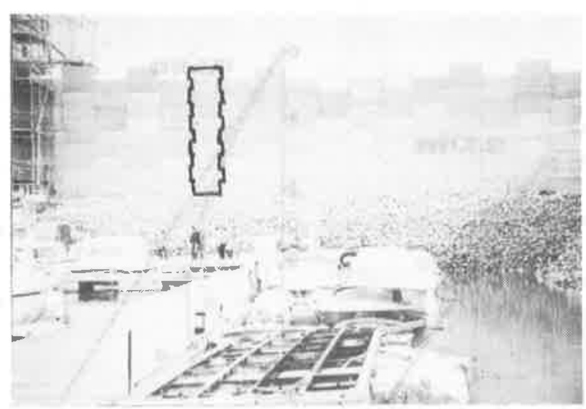

Fig. 5 Jersey wall showing instrumented section

(iii) Inclinometer tubes

2 No. square section inclinometer tubes were installed behind the facing panels. These extend to the full depth of the wall (Fig. 7). Readings have been taken at regular intervals during and since installation, using the Norwegian Geonor system. The results to date suggest that small movements of the wall are taking place. Readings are continuing.

(iv) Strain gauged Paraweb

It had been intended that some layers of the Paraweb strips within the test section should be strain gauged in order to monitor load distribution. However considerable problems were encountered in obtaining a suitable adhesive for bonding the gauges to the polyester fibre. Just prior to the start of site work an adhesive was located in Japan, but there was only time to strain gauge a short experimental length of Paraweb. The strain gauges are giving readings but the results are not encouraging. Even if it had proved possible to strain gauge long lengths of Paraweb it is unlikely that useful results could have been obtained. The placing of the rock fill caused considerable distortion to the Paraweb strips and may well have damaged the strain gauges.

(v) Magnet extensometers

Magnet extensometers were successfully used to monitor the extension of the Paraweb strips in the Southampton wall. In Jersey 3 No. of these magnets were incorporated in each of 10 layers of Paraweb. A 15 mm diameter plastic pipe was laid over each line of magnets and protruded through a small hole in the rear timber facing panels. It had been intended to insert a probe, incorporating a Reed Switch, into each pipe to locate the magnets. Unfortunately the severe and unexpected tortuosity created in the Paraweb by the use of rock fill prevented the subsequent monitoring of these magnets.

In an effort to record vertical settlement of the rock fill 5 No. 300 mm diameter ring magnets were placed horizontally around a vertical sectional 30 mm diameter plastic pipe, in three locations across the instrumented section. It proved very difficult to successfully install this system because of the plant being used to construct the wall. The rock fill appeared to be continuously moving as additional layers were placed. As a result two of the vertical pipes "walked" to the outer limits of the wall, and the magnets were displaced.

The work at Jersey has highlighted some of the considerable difficulties associated with carrying out scientific research on a working construction site. Even during the short period available for development and fabrication of instruments there were specification and design changes on the structure that caused last minute problems to the instrumentation programme. As a result some of the instruments have not performed as expected. On several occasions instruments were inadvertently damaged by plant on site after installation.

The experience gained during the Jersey work has proved very beneficial in the design, construction and instrumentation of an experimental wall recently completed at Portsmouth.

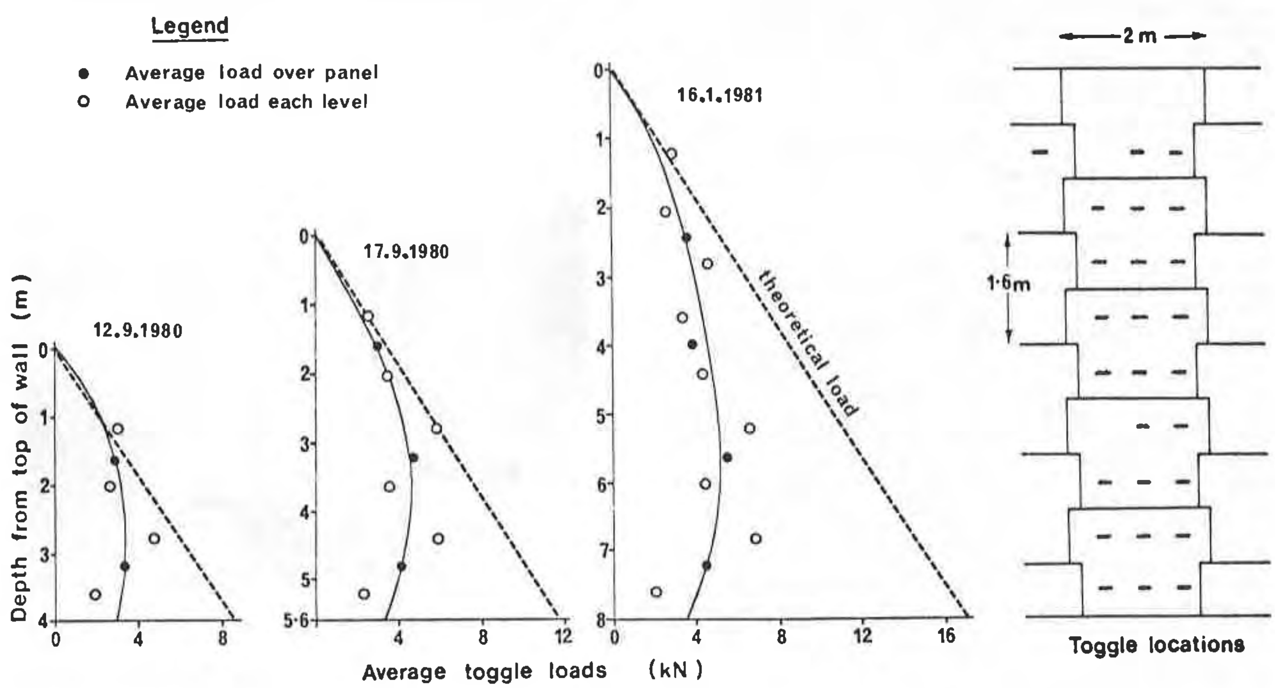

Fig. 6 Toggle Loads, Jersey Wall

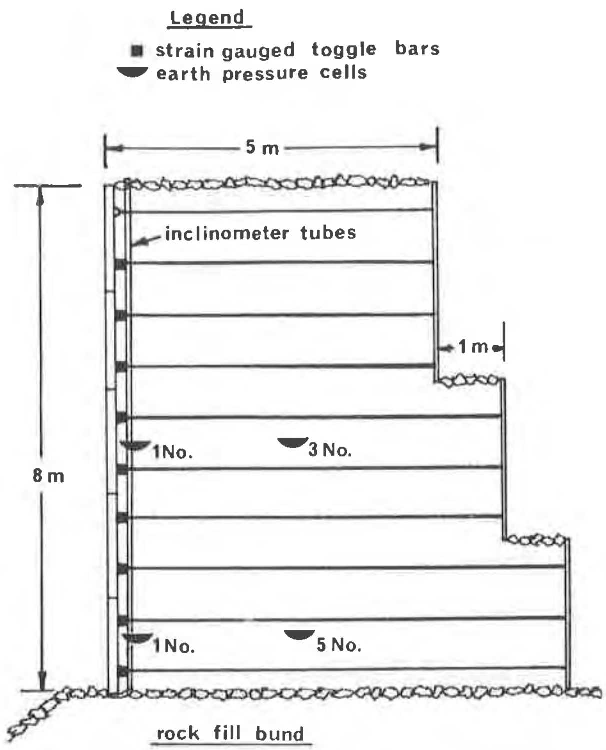

Fig. 7 Schematic of cross-section of Jersey wall showing instrument positions.

Fig. 8 Portsmouth wall nearing completion

3. PORTSMOUTH WALL

This experimental wall has been constructed at the Geotechnics Field Centre, Portsmouth Polytechnic (Fig. 8). The design is based on the Websol System and incorporates several different reinforcement configurations. The wall is approximately 20 m long, 5 m wide and 2½ m high. The facing panels are of timber construction and the fill material is single size 20 mm gravel aggregate.

Without the sea-water environment it was possible to develop a relatively simple strain gauged toggle bar, located outside the timber panels. Although fabrication and calibration was a major task 224 No. special toggles have been incorporated in the structure to monitor the various reinforcement configurations. The initial difficulties associated with attaching and waterproofing strain gauges on the Paraweb has largely been overcome and one section of the wall is instrumented in this way. Inclinometer tubes have been installed within the fill material at various locations. Terram sheet has been utilized in part of the wall and omitted from a directly comparable section. In this way it is hoped to more accurately assess the contribution of the Terram to the reinforcing function. Pull-out tests will be carried out on single and multiple Paraweb strips.

The northern end of the wall has been constructed with Paraweb strips crossing at right angles in a similar manner to that of a reinforced soil bridge abutment. In the southern end provision has been made to enable the Paraweb strips to be progressively shortened until failure is induced at that end of the wall.

Land survey marks have been incorporated in the facing panels, and when all the survey and instrument readings have stabilized the pull-out and failure tests will be carried out.

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FUKUOKA, M. and IMAMURA, Y.
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Fabric Retaining Walls

Mur de soutement de géotextile

The fabric retaining wall with multiple anchors was designed utilizing the merits of fabrics, light weight, ease of handling, and economy. Problems were earth pressure on the vertical wall, strength of fabrics, and method of construction. Results of a full scale model test revealed that the earth pressure estimated was satisfactory. The fabrics used were strong enough for this type of retaining wall. Problems related to construction were solved during construction. This type of retaining wall can be recommended for temporary construction work even soft ground without piling. Fabric gabions which were used for preventing the occurance of fault between bridge abutment and backfill, a retaining wall made of fabric sheets, and a large concrete block retaining wall with fabric at its back face are introduced here in brief.

La paroi de retenue structurée à ancres multiples a été réalisée en mettant en valeur des qualités remarquables, soit: une legerete de poids, une utilisation facile et un avantage économique. Le problème était la pression de terre contre la paroi verticale de face, c'est-à-dire la solisité de structure et la méthode de construction. Les résultats des examens sur des modèles de grandes dimensions ont révélé que sa résistance à la pression de terre satisfaisante. La structure utilisée avait une solidité suffisante pour ce type de parois. Les problèmes relatifs à la construction ont été resolus au cours de celle-ci. Une paroi de retenue de ce type peut etre utilisée sans pilier, meme aux sols mous et est recommandable pour les travaux de constructions temporaires. La gabions de structure destinés à la prévention des différences de niveaux qui peuvent survenir entre les parois de retnue des butées et des remlais des butées des points, les parois de retenue fabriquées avec des feuilles structurées, etc. seront presentes brièvement.

INTRODUCTION

Professor Fukuoka has conducted research work on retaining walls collaborating with many engineers for about 20 years. Field tests, laboratory testings, and theoretical analysis have been carried out on cantilever walls, concrete block walls, inverted Y type walls, concrete frame walls, steel walls, and multiple anchored walls. He has been studying fabric retaining walls for about 10 years. Here are four examples.

(1) Cylindrical, fabric sand packs (gabions), were placed horizontally inside the backfill of a cantilever retaining wall (Fig. 1). Prior to this experiment with the prototype retaining wall, small scale model tests were performed in order to compare the effectiveness of sand pack and fabric sheets. According to the test results, a large fault between the concrete wall and the settled backfill did not appear due to the function of the sand packs. A car running at high speed on pavement laid on the backfill will run into the abutment at this large fault. The sand packs will serve as an anchor and drain in the backfill. Very strong fiberglass was used for the sand packs so that they would not break or tear.

(2) Fig. 2 shows a retaining wall made of fabric sheets and steel wire meshes with steel plates. This experiment was performed by Yamada and Sakaguchi of the Taisei Corporation as suggested by Prof. Fukuoka. The steel meshes and the fabric sheets were laid horizontally. The tail of the mesh is inserted into the backside slope as an anchor, and the front of the retaining wall felt with seeds is inserted between the steel mesh and vinyl net.

(3) The retaining wall in Fig. 17 is composed of heavy concrete blocks weighing 10, 20, and 40 kN. Fablic is placed behind the back of the retaining wall in order to let water drain out without bringing sand in the backfill. The fabric should be permeable, strong enough not to be torn even by the strong impact of rocks, and durable without aging. It is important for design to know earth pressure on the back side of the concrete blocks.

(4) Prof. Fukuoka invented the multiple anchored retaining wall, and used fabric for the front wall. The following sections 1-7 are a report of the testing.

1 DESIGN OF FABRIC RETAINING WALL

The test retaing wall is 5m high. It has a row of columns, fabric stretched between them, and steel rod anchors. It is necessary to estimate earth pressure on the vertical wall and forces acting on the anchors. With reference to past case records, the following were assumed---unit weight 13.5 kN/m^3, coefficient of earth pressure on the assumed vertical wall at about 5m behind the vertical front wall 0.5, diameter of steel rod 19mm, frictional force on the steel rod 2 kN/m, frictional stress between the base ground and the backfill 3.5 kN/m^2. Fig. 3 shows earth pressures and forces acting on members of the retaining wall and backfill used for design. The cross section of the concrete column is square (20cm x 20cm) and 4 stages of steel bars (diameter 19mm) are inserted. Concrete anchor plates are square in shape, and their size is 40cm x 40cm x 10 cm. An anchor should be strong enough to resist pulling out force and small enough in order not to disturb the movement of a bulldozer. Two kinds of fabrics are used, namely net type and sheet type. The net type fabrics are easy to stretch, and therefore they were fixed flat

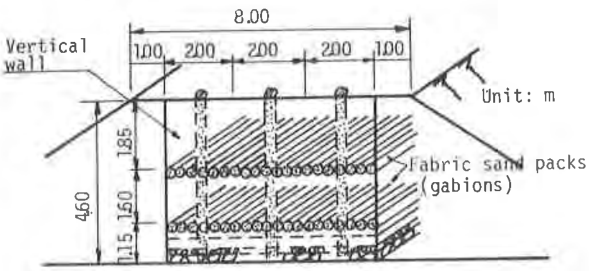


Fig.1 Front view of gabions placed in backfill behind vertical wall of cantilever retaining wall.

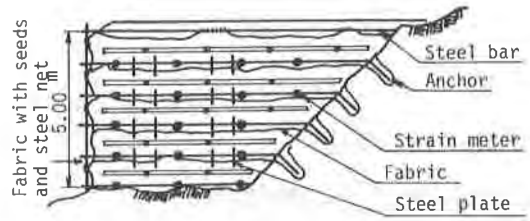


Fig. 2 Cross section of fabric retaining wall.

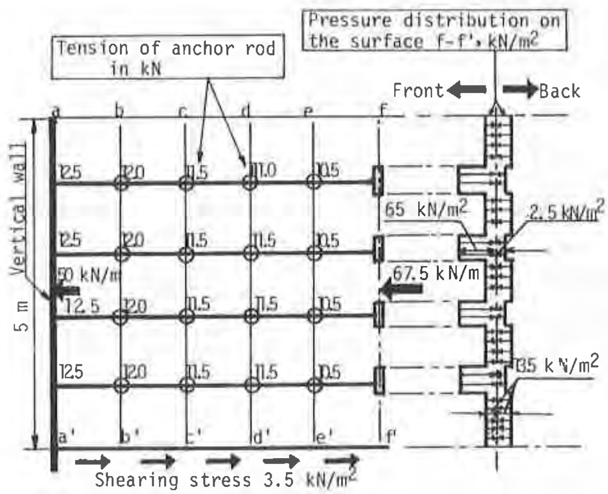


Fig.3 Assumed forces and stresses used for design.

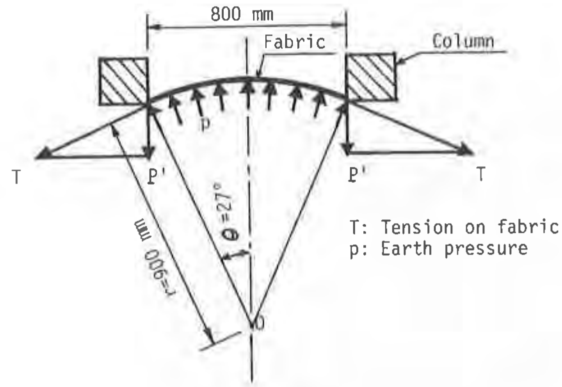


Fig. 4 Tension on fabric.

Table 1. Test results of fabrics.

Name	Testing	Direction	Before construction (A)	After one year(B)	B/A in %
Sheet type fabrics	Tensile strength in kN/m	Longitudinal	72.0	66.0	92
		Transversal	74.0	53.0	72
	Elongation in %	Longitudinal	20.0	15.8	79
		Transversal	18.0	9.0	50
Neet type fabrics	Tensile strength in kN/m	Longitudinal	61.0	57.0	93
		Transversal	62.0	55.0	89
	Elongation in %	Longitudinal	27.5	25.3	92
		Transversal	26.8	24.7	92

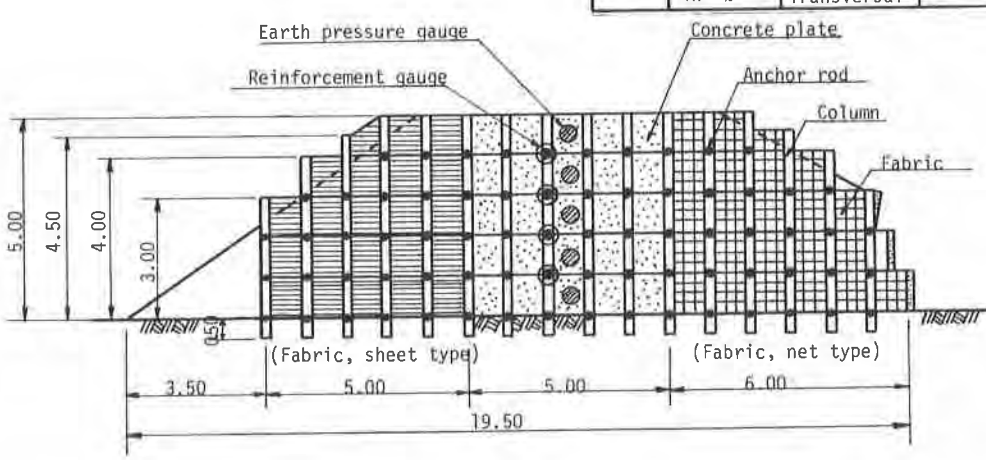


Fig.5 Front view of retaining wall (unit:m).

to the concrete columns without any slack. The sheet type fabrics were fixed to the concrete columns with slack. Both of them are expected to become a semi circle as shown in Fig. 4. The earth pressure on the front wall was assumed to be 12.5 kN/m^2, but the design earth pressure for the fabric wall was taken as 20kN/m^2, considering the scattering of earth pressure. The fabric used was 1.25m wide and its tensile strength was 60-72 kN/m (Table 1). The tensile force acting on the fabric was computed as 18 kN/m, which was much less than its tensile strength. Fig. 5 shows the front view of the test retaining wall. Concrete slabs were used at the middle part of it for measuring earth pressure with earth pressure gauges.

2 PLAN OF MEASUREMENT

Earth pressure gauges were attached to the front and rear faces of the concrete anchor plates. Wire strain gauges were pasted to the 4 steel rods. Earth pressure gauges were attached to the back faces of the concrete plates placed just behind the concrete columns. Fig. 6 shows a picture of the instrumentation arrangement. Displacement of columns and settlement of the backfill were measured.

3 TEST RESULTS OF FABRIC AND SOIL

Water content, unit weight, and static cone resistance were measured during construction. Table 2 shows the results of soil tests. Table 3 shows the properties of the fabrics.

4 CONSTRUCTION OF RETAINING WALL

First, the holes were excavated, and the concrete columns were erected. The cross section of a hole was 0.4 m x 0.4 m and the depth was 0.5 m. The size of the column was 0.2 m x 0.2 m x 5.5 m. It was about 5.5 kN in weight. The column had 5 holes with a diameter of 0.3 m at the heights of 1.0, 2.0, 3.0, 4.0 m from the ground surface. The steel anchor rod had two long bolts (diameter 19 mm, length 400 mm) at each end. The concrete anchor plate had a hole for the anchor rod. The threaded long bolts were used instead of turnbuckles. The concrete plates attached behind the columns were 1 m x 1 m x 0.15 m in size and had no reinforcement. The concrete columns erected vertically at 1 m intervals (Fig. 5). The lowest stage of anchors were set on the ground. The concrete plates and fabrics were attached behind the columns. The net type fabric was stretched on the right as one faced the wall, and the plate type fabric was attached on the left with slack. Both ends of the fabrics were wound around wooden square beams, and fixed to the concrete columns with steel wires. The fabrics were fastened lightly to the columns with wires. A small bulldozer weighing 4 tons was used for the filling operation. Dry density of the backfill was rather low as indicated in Table 1, and it was 79-85 % of the maximum dry density by laboratory compaction test. The second stage anchors were laid on the fill surface 1 m high. The second stage fabrics and concrete plates were fixed to the concrete columns. The backfill was raised up to 2 m. Similar operations were repeated until the fill height reached the level of 5 m. A light steel frame work was installed to support the concrete columns from the beginning of construction work till the completion of the second layer of 2 m, in order to prevent the columns from inclining. If the frame work had been used from the beginning to the end of construction, the deformation of the retaining wall would have been much smaller. Steel pipes of 80 mm in diameter could have been used instead of the heavy concrete columns which required a crane for erection.

5 RESULTS OF MEASUREMENT

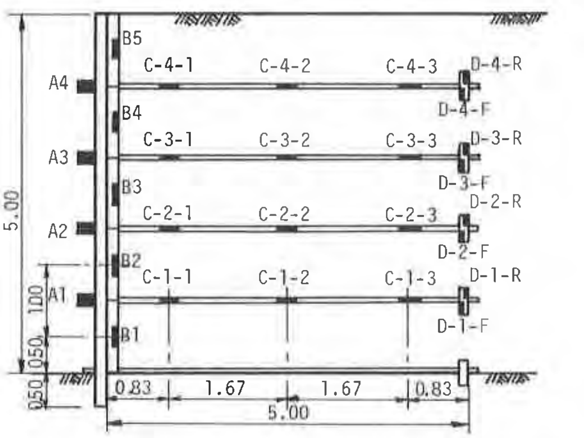
Fig. 7 shows the earth pressures on the back side of the vertical wall and the back and front faces of the anchor plates, and the tensions of the anchor rods. The measurements have been continued for more than one year after the completion of construction, and earthquake forces were recorded. Fig. 8 shows displacement of the concrete column. Unit weight, water content, and cone resistance, were measured as 14.5 kN/m^3 (12.3-15.4), 71.0 % (55.0-94.9), and 40 kN/m^2 (20-100), respectively. Fig. 9 shows the cone resistance with depths. The settlement plates were placed on the ground surface (EL 0), and at the levels of EL 0.782 m, EL 2.424 m, EL 3.795 m respectively. The amounts of settlement were measured as 0.082, 0.087, 0.142, and 0.075 m respectively. A reinforcement gauge and three sets of wire strain gauges were attached to the steel anchor rod. Fig. 10 shows the results of these measurements. The tensile forces measured with the reinforcement gauges were lower than those of the wire strain gauges. Friction between the rod and the column may be the reason for this difference. The tensile force of the anchor rod is caused by the relative displacement of the anchor plate. The relationship between the relative displacement and tensile force of rod can be obtained as shown in Fig. 11. The anchors at the levels of 1 and 2 m (C-1,C-2) have a resistance of about 10 kN corresponding to displacement 25 mm, but the resistance of the anchors at the level of 3 and 4 m (C-3, C-4) is 5 kN for the same 25 mm displacement. The reason for this displacement may be soil properties which are represented as the cone resistance of the backfill as shown in Fig. 9. Frictional force acting around the rods C-2,3, and 4 is small, and that of C-1 is 2-3 kN. Earth pressure on the backside of the vertical wall was measured with earth pressure gauges. If the fabric had been fixed without any slack, and stretched out only by by the earth pressure, acting earth pressure on the fabric could have been back calculated. However the acting earth pressure could not be back calculated because the fabric was fixed to the columns with slack. The shapes of the fabric between the columns, bent by the earth pressure, were measured. Fig. 12 shows one example of the lower part of the wall, where the earth pressure is the highest. Tensile strength of the double fabrics was 142 kN/m at the end of construction, and so the fabrics will be safe for many years even if they lose their strength by aging. Fig. 13 shows earth pressure obtained by the earth pressure gauges installed in the anchor plates.

6 COMPARISON OF MEASURED VALUES WITH PREDICTED ONES

The design of the retaining wall was made based on assumed earth pressure on the vertical wall and anchor plates, and assumed tensile forces acting on the anchor rods. Fig. 3 shows the assumption used for design. Comparing the measured values with the assumed ones, the following conclusions may be reached.

(1) The earth pressure on the vertical wall is shown on Fig. 14. The distribution of earth pressure was assumed to be similar to that on sheeting of open cut. In this case the earth pressure diagram has a triangular shape, but a trapezoidal shape was assumed for design. Total horizontal earth pressure was 65 kN/m, and this is much larger than the design earth pressure of 50 kN/m. The coefficient of earth pressure 0.36 is plotted on Fig. 15 showing inclination of walls versus coefficients of earth pressure. Earth pressure on the assumed vertical wall ff' is 82.5 kN/m, which is much larger than the predicted one of 67.6 kN/m, and the coefficient of earth pressure is 0.45 contrary to the expected one of 0.4.

(2) The tensile force on each rod was assumed to be



A : Reinfocement gauge. B : Earth pressure gauge on front wall.
C : Wire strain gauge.
D : Earth pressure gauge on anchor plate.

Fig.6 Arrangement of gauges (unit in m).

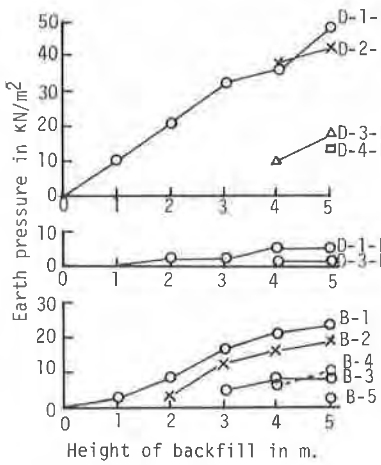


Fig.7 Earth pressure on vertical wall versus height of backfill.

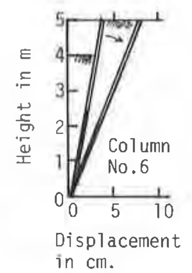


Fig.8 Displacement of concrete column.

Table 2. Results of soil test.

	Backfill	Foundation	Remarks
Natural water content w %	73.0	85.0	
Unit weight γ kN/m^3	14.6	15.1	
Specific gravity Gs	2.61	2.71	
Liquid limit w_L %	85.2	156.5	
Plastic limit w_P %	66.9	52.7	
Cohesion Cu kPa	1.1 2.6	1.1	Triaxial test
Angle of inter friction Φu	15	5	
Optimum moisture content w_{opt} %	62.0		JIS 1-1-b
Maximum dry unit weight γ_{dmax} kN/m^3	9.42		
Optimum moisture content w_{opt} %	71.0		JIS 1-1-a
Maximum dry unit weight γ_{dmax} kN/m^3	8.70		

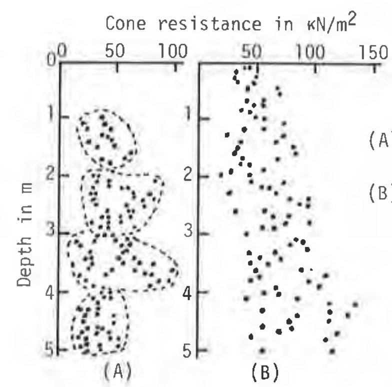


Fig.9 Cone resistance.

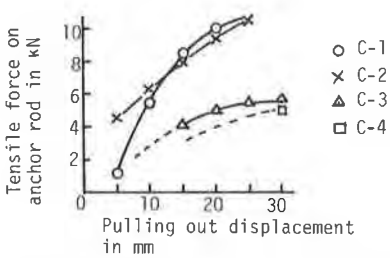


Fig.11 Pulling out displacement versus tensile force of anchor.

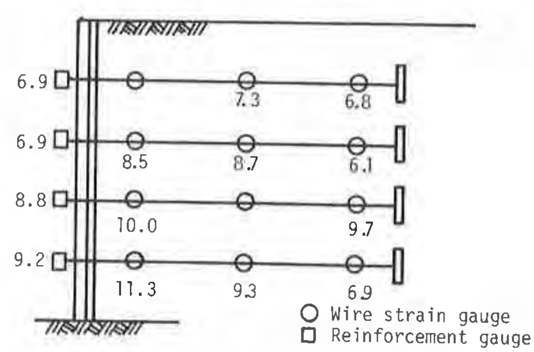


Fig.10 Tension on anchor rod in kN.

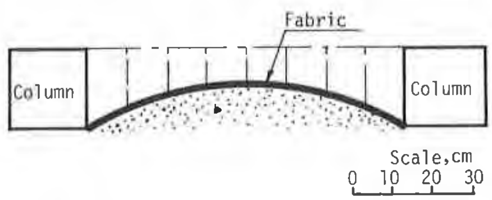


Fig.12 Lower part of fabric wall.

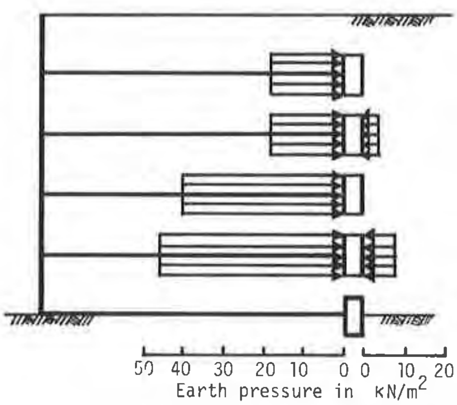


Fig.13 Earth pressure on anchor plate.

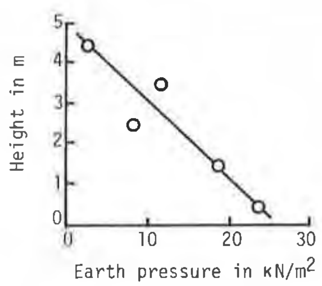
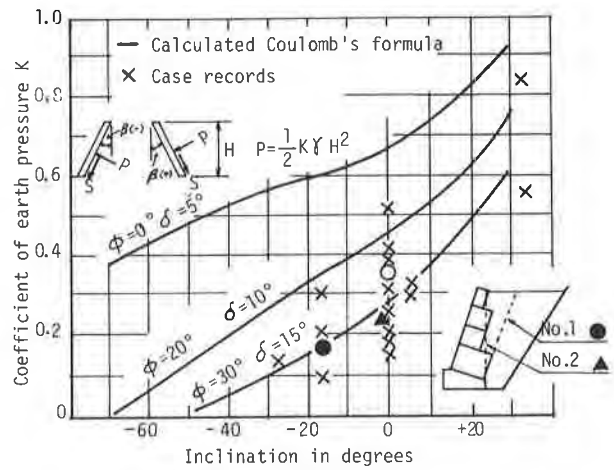


Fig.14 Earth pressure on vertical wall.

Table 3. Fabric used for large concrete block retaining wall.

	Longitudinal	Transversal
Maximum tensile strength (kN/m)	6.08	9.87
Elongation at failure (%)	180	89
Tearing strength (N)	255	665
Weight:300 g/m^2, Thickness:4 mm Coefficient of permeability:3 × 10^{-2}cm/s		



φ: Angle of internal friction
δ: Angle of wall friction
○: Fabric retaining wall with mulitiple anchors

Fig.15 Coefficient of eath pressure and inclination of retaining wall.

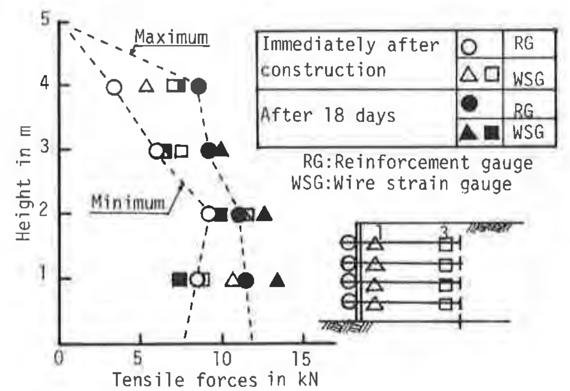


Fig.16 Maximum and minimum tensile forces measured.

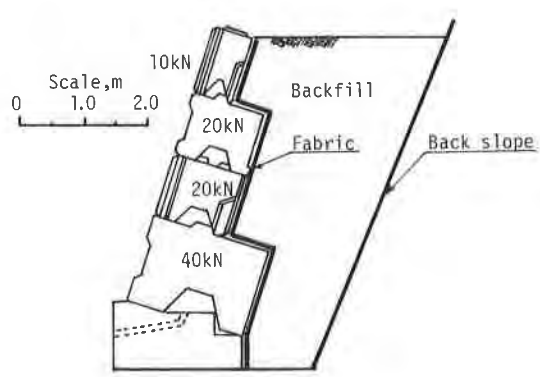


Fig.17 Large concrete block retaining wall with fabric sheet backside.

12.5 kN near the vertical wall and 10 kN near the anchor plate. The assumed values are rather smaller than the predicted ones, with a maximum 11.3 kN and minimum 6.8 kN. The friction on rods was assumed to be uniform and 0.2 kN/m, but actually ranged from -0.1 to +1.6 kN/m, and their magnitudes and directions were not the same. Only the anchor 1 m high showed high frictional force.
(3) Earth pressures on the front and rear sides of the anchor plates were assumed to be 65 kN/m^2 and 2.5 kN/m^2, respectively. The earth pressures on the other plates measured were smaller than the assumed ones. The reason for this difference seems to be the difference of earth pressure distribution, and non uniform tensile forces on anchor rods.
(3) The assumption, as shown in Fig. 4, used for designing fabrics seems to have been quite satisfactory.

7 RESULT OF LONG TERM OBSERVATION

The retaining wall has been left for about one year, and earth pressure, deformation, and its behavior during heavy rains and earthquakes have been observed. The test was performed after about one year with the fabrics to study the effects of aging.
(1) The earth pressure has changed with time. Fig. 16 shows the maximum and minimum limits of earth pressures. The effects of earthquakes and heavy rains were relatively small compared with the changes at ordinary times.
(2) It has been said that a retaining wall with cohesive soils as backfill moves forward with time, but this retaining wall has shown no appreciable movement since its completion.
(3) Increment of earth pressure during heavy rains were on the order of 0.1 kN with the C-1 anchor. There were several earthquakes recorded. The tensile force on C-1 anchor was 0.3 kN during the earthquake, accelaration of of which was 16 gals at the ground, and 51 at the top. The tensile force was generated by inertia force on the column.
(4) Pieces of the fabrics were cut out and tested. The results are written in Table 1. Judging from the test results, the fabrics may be used for 5 to 10 years.

8 TEST ON LARGE CONCRETE BLOCK RETAINING WALL WITH FABRIC SHEET BACKSIDE

Large concrete blocks weighing 10-60 kN are piled up and an inclined retaining wall is constructed with mixture of coarse gravels and sands with silts as backfill (Fig. 17). The concrete blocks have cavities, through which water in the backfill can be drained. Fabrics are placed on the backside of the retaining wall in order to prevent spilling of fine materials in the backfill. The fabric should be permeable. As large rocks are thrown into the backfill space, the the fabric should be strong enough to withstand the shock of rocks. The life of the fabric should be sufficiently long. Table 3 shows the characteristics of the fabrics used. A spilling test with fine sands 0.105 - 0.25 mm in diameter was performed. No sign of spilling of sands was noticed. Good results were obtained in a shock test with gravel and a laboratory aging test. The prototype model test was performed to study earth pressure on the retaining wall at ordinary times. The increment of earth pressure was very small when artificial heavy rain was applied. Fig. 17 shows a cross section of the retaining wall. The panel type pressure gauges were installed behind the blocks. This particular retaining wall has fabrics on the back face, and has steps too. It makes it quite difficult to estimate earth pressure on the retaining wall. Fig. 18 shows measured earth pressure by arrows. Coefficients of earth pressures on the assumed surfaces No. 1 and No.2 on Fig. 15 are K=0.18 and 0.25 respectively. Those coefficients are plotted on the figure. Angles of wall friction δ are as large as 47 and 36 degrees for the assumed surface No. 1 and 2 respectively. Those values are much larger than what would be commonly thought.

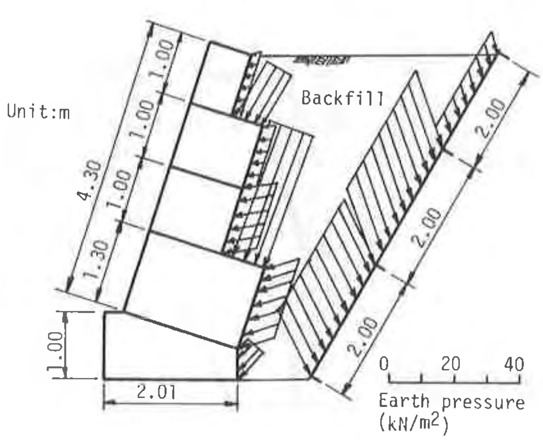


Fig.18 Earth pressure measured.

CONCLUSIONS

(1) Fabric retaining walls with multiple anchors were tested. It was revealed that a retaining wall of this type can be constructed easily even by unskilled workers, that speed of construction is very high, that relatively soft foundations can be used without piling, and more over that it is very economical. Therefore, it can be used for temporary construction of 5-10 years.
(2) The large concrete block retaining wall has been used at mountainsides. The back side of the retaining wall has steps and fabric. The panel type earth pressure measuring system was used to get accurate earth pressure on this retaining wall. It was found that a large angle of wall friction appeared on the back face. The use of fabrics was justified.

ACKNOWLEDGEMENTS

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Practical Construction Techniques for Retaining Structures Using Fabrics and Geogrids

Techniques pratiques de construction pour les structures de soutement, utilisant des textiles et des géogrids

Early examples of geotextile structures generally had a mis-shapen form and it was often suggested that the role of geotextiles in retaining structures and soil reinforcement was limited to temporary works or to applications in which appearance was of secondary importance. Coupled with the early trial structures has been the fear, expressed by some authors, that the relatively low elastic modulus of geotextiles (compared with steel) would theoretically result in distortion of any retaining structure.

The paper illustrates, using case histories of both full scale trials and completed retaining walls and bridge abutments, that geotextiles can be used on the most demanding sites to produce economic structures of high aesthetic appeal with no defects or distortion. It has been found that the critical element to the successful development of geotextile structures is the construction technique and that the theoretical reservations are groundless.

Les premiers exemples de structures geotextiles avaient généralement une forme peu satisfaisante et on suggérait souvent que le rôle des géotextiles dans les structures de soutenement et le renforcement due sol était limité à des travaux temporaires ou à des applications ou l'apparence etait d'importance secondaire. Les premiers essais de structure out aussi engendré la crainte exprimée par certains auteurs, que le coefficient d'elasticité relativement faible des géotextiles (comparé à l'acier) entraînerait théoriquement une deformation de toutes les structures de soutènement.

Cette communication illustre, à l'aide de cas à la fois d'essais à grande échelle et de murs de soutènement et de culées de ponts terminés, que les géotextiles peuvent etre utilises sur les sites les plus difficiles, pour produire des structures économiques d'un grand attrait esthétique, sans defáut ni deformation. On a constaté que l'élement critique à la réussite du developpement de structure géotextiles est la technique de construction et que les réserves théoriques ne sont pas fondées.

INTRODUCTION

Early examples of geotextile structures generally have a mis-shapen form and it has been suggested that the role of geotextiles in retaining structures and soil reinforcement should be limited to temporary works or to applications where appearance is of secondary importance. Coupled with the early trial structures has been the oft expressed fear that the relatively low elastic modulus of geotextiles would result in distortion of any retaining structure using fabrics.

Al-Hussaini (1977)(1), in a field trial, conducted at the US Army Engineer Waterways Experiment Station (WES), of a wall reinforced with nylon fabric in the form of a strip, postulated that the collapse of the structure was the result of excessive deformation, even though the safety criteria for reinforcement tie breakage and tie pullout were satisfied. In a separate trial conducted in Oregon Bell and Steward (1977)(2), found no evidence of creep in a fabric retained wall and concluded that creep may not be as major a problem with this form of structure as originally anticipated.

From a cursory inspection, the results of these two field trials are completely contradictory. However, there were differences between the two trials, perhaps the most significant being the form of the reinforcement used. The WES trial used 100mm(4") strip reinforcement laid at 1200m (4') centres, giving a reinforced ratio in a horizontal plane of 1:12. The strain to failure of the reinforcement appears to have been approximately 20%; however, the strain in the reinforcement at failure can be calculated as varying from 2.8% at the foot of the wall to 5% at the top.

In contrast the fabric reinforcement used by Bell and Steward (3) had a reported failure strain varying between (60-166)%. However, complete layers of reinforcement were used, and, the reinforcement ratio in the horizontal plane was (1:1). Although significantly greater areas of reinforcement were used by Bell and Steward this does not satisfactorily account for the different conclusions reached in the two trials.

Andrawes and McGown (1978)(4), have shown that the mechanisms originally used to describe the reinforcement of soils are simplistic, and argue that rather than referring to materials placed in soils are 'reinforcements', they should be termed as 'Inclusions', as their influence may not always be one of strengthening soil structures. For solid reinforcement such as steel, and also perhaps some particular generic forms of reinforcement such as strips formed from any materials, the insitu and in-isolation properties are likely to be the same. However, for materials having a composite construction such as fabrics and geogrides, and which are laid in such a manner that sheet and abutment effects are evident, their insitu properties are known to be different to those in-isolation. Moreover, the insitu properties will vary depending upon the particular application and the reinforcement-soil ratio. In some applications, it can be shown that low deformation characteristics, low hysteresis and low creep are important. In other conditions the need for low creep and good hysteresis is not significant. Where an "Inclusion' has a higher or lower insitu rupture deformation within the soil determines, to some extent, the character of the soil-inclusion interaction.

A comprehensive understanding of the behaviour of earth reinforced structures has not been developed and the designer of fabric reinforced structures has few texts for assistance. The majority of current design methods for earth reinforced structures takes no account of the magnitude of the strains induced in the tensile members as these have, in the past, been manufactured from high modulus materials such as steel, glass fibre, or the higher range of molecular orientated polyolefins. Murray (1980)(5), using force-equilibrium considerations, has produced design equations relating to fabric reinforced earth walls, but these rely upon in-isolation material properties and, in the Author's opinion, grossly over estimate the probable strains. However, in the absence of other information, they form a guide.

Trial Structures

An alternative to theoretical systems is to use empirical methods based upon experience. In order to use this approach, general experience, including insitu material behaviour, is required of those applications where fabric or goegrid structures can be economically employed. This experience can be obtained through trial applications.

To study the use of fabric structures in an urban environment, conventional low retaining walls were identified as worthy of construction. The majority of these are gravity structures constructed in brick or masonry. The principle adopted with the fabric wall was to replace the mass of the wall using a fabric stabilised granular fill, whilst retaining the conventional facing, Fig 1.

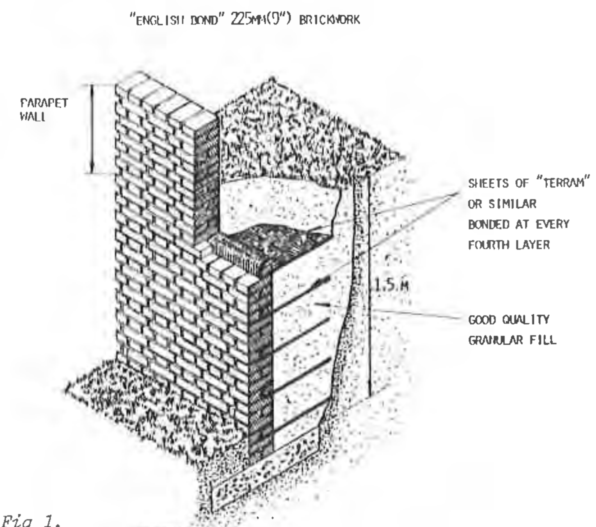

Fig 1.
FABRIC REINFORCED BRICK RETAINING WALL

A full scale trial has been conducted in West Yorkshire using conventional materials. The trial consisted of two fabric reinforced walls built alongside an unreinforced wall, Fig 2. The reinforcement used in the trial was ICI 'Terram 100' the properties of which are detailed in Table 1. The length of the reinforcement was 1m and the height of the walls 2m. All three structures were backfilled in 150m(6") layers, using crusher run stone and hand ramming. On completion of filling, all sections of the wall remained standing including the 112mm(4½") wall formed from a single skin of brickwork and having no reinforcement. After 6 months the unreinforced wall collapsed by overturning about the toe. After 12 months the reinforced earth walls were still showing no signs of distress and accordingly, the 112mm(4½") reinforced wall was surcharged with 1m of unbonded building bricks.

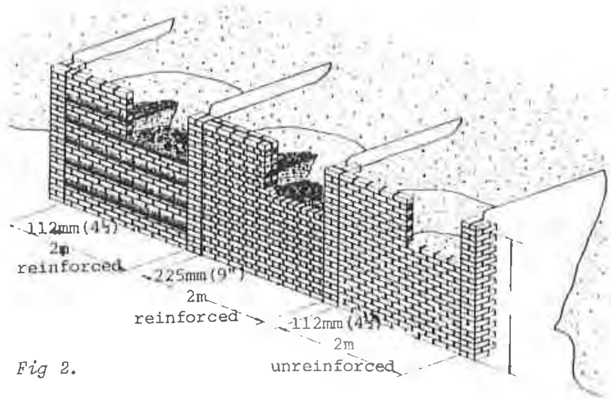

Fig 2.

The movement of the walls is recorded in Fig 3.

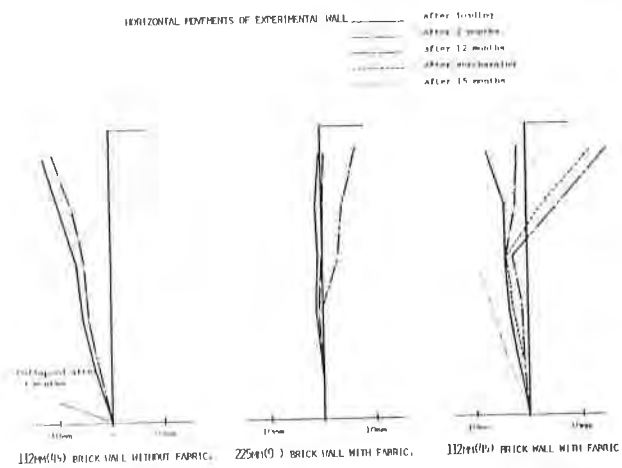

Fig 3.

This indicates that the reinforced earth walls have rotated backwards in the manner postulated by Jones and Edwards (1981)(6), There is no evidence of large extensions of the reinforcement having taken place, although Table 1 shows that the fabric is very extensible (an expansion of 45% at full load would result in a movement of 225mm(9") in 500mm).

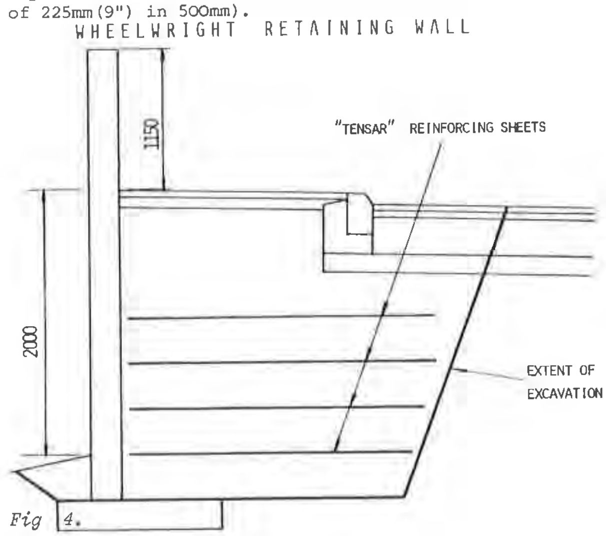

Fig 4.

TABLE 1. MECHANICAL PROPERTIES OF 'TERRAM'

	"Terram" Product	1000
200mm Plane strain test	Tensile Strength	
	Max load Newtons/200mm	1700
	Extension at Max. load -%	45
	Load at 5% ext'n - Newtons	500
	Rupture Energy - Joules	100
25mm Grab test	Max load Newtons	850
	Ext'n to Max load -%	80
	Load at 5% ext'n - Newtons	160
	Tear Strength Wing - Newtons	250
Burst test	Bursting Load Newtons/cm^2	110
	Distension at Burst - mm	15

In order to use this form of structure for permanent works a satisfactory appearance is essential. This can be obtained using a construction technique sympathetic with the material and material behaviour.

From a serviceability approach, the walls are perfectly adequate, although the existing theories would not predict that the reinforced earth walls would have behaved as recorded. Similar measurements on a second brick retaining wall have been undertaken; in this case the reinforcement was a molecular orientated polymer geogrid (Tensar). The properties of the reinforcement are shown in Table 2. The wall was built in an urban environment to support a small road following the collapse of a gravity structure. The structure consists of 224mm(9") brickwork with the reinforcement laid between courses at 300mm(12") levels, Fig 4. The wall has been monitored for a period of 2 years, during which no movements in excess of 2mm have been recorded.

Practical Construction Techniques

The above trials indicate that post construction distortion does not occur and that design for overall stability

TABLE 2. TENSAR

STRUCTURAL CHARACTERISTICS	SR1	SR2
Width (m)	1.0	1.0
Weight (gms/m^2)	872	938
Grid pitch (mm)	11x54	23x108
Colour	Black	Black
MECHANICAL CHARACTERISTICS		
Tensile strength - max (kN/m)	84.0	79.0
Extension @ max load (%)	12.3	12
Extension @ 40% load (%)	3.5	3.0
Modulus in Tension (N/m^2)	$5.2x10^9$	$4.1x10^9$
Thermal Stability	Stable over temperature range of -60°C to 80°C	
POLYMER CHARACTERISTICS - SR1 and SR2		
Raw material	: High Density Polythene	
Chemical Resistance	: Resistant to all natural occuring alkaline and acidic soil conditions (i.e. < pH)	
Biological Resistance	: Resistant to attack by Bacteria fungi and vermin	
Sunlight Resistance	: Resistant to UV attack	

* The properties of this material have been improved since this

may be the most critical factor, Jones and Edwards (1980) However, previous constructions built using fabrics have been mis-shapen and their use, particularly in an urban environment, has not been seriously contemplated. In order to use this form of structure for permanent works a satisfactory appearance is essential. This can be obtained using a construction technique sympathetic with the material and material behaviour.

It has been found that control of the distortion of the face can be easily achieved by introducing a regular shaped backing block around which this fabric can be wrapped and against which the compaction plant can operate. Suitable materials for this backing block have been found to be redundant kerbstones, although no fines masonry blockwork is the easiest to use and produces the most regular structures. It has been found that little distortion of the fabric reinforcement occurs during construction and that what does occur can be easily accommodated in the facing skin which is applied following the construction of the fabric elements of the structure, Fig 5.

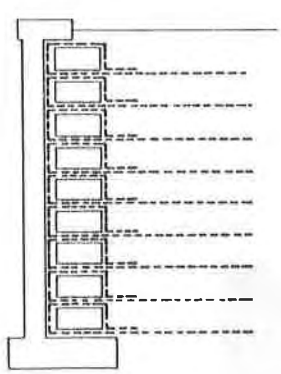

Fig 5.

Fig 6.

This system has also been adopted for the construction of bridge abutments in an area of mining subsidence. Conventional structures such as masonry arches cannot accommodate the ground strains resulting from mining. In the Yorkshire Coalfield these strains typically amount to settlements in excess of 1m and compressive and tensile strains in excess of 3mm per meter. An arrangement for a fabric reinforced abutment is shown in Fig 7(a).

Fig 7(a)

NEWTON LANE BRIDGE

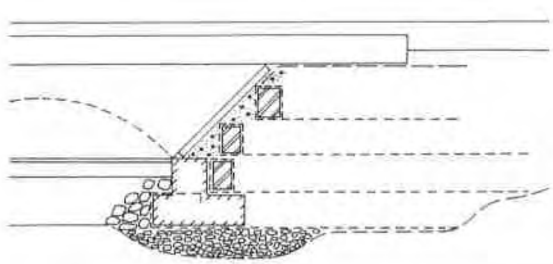

Fig 7(b)

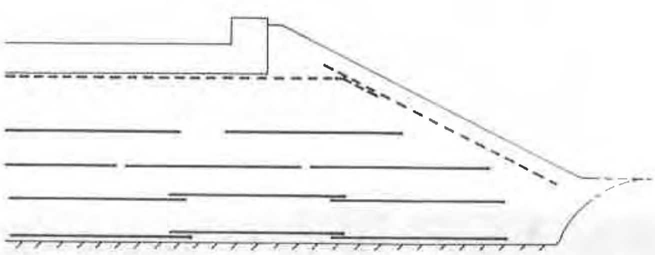

Fig 7(b) shows that an element of curtailment of a reinforcement towards the top of the abutment can be obtained. Geogrids can also be used to form successful structures, particularly where subsoil conditions are very poor. Fig 8, shows a reinforced earth structure constructed using cohesive colliery shale as fill reinforced with Tensar geogrid. The facing is formed from prestressed concrete panels.

Since construction the movement of the facing has been recorded, Fig 9. The measurements show little movement even though the structure is heavily surcharged every few minutes during a 24 hour period. The critical element of the construction of this particular structure is that the facing is propped and held in position during back filling. In addition, during the filling, the reinforcement is held taut. Upon removal of the props from the face, a small degree of readjustment is possible within the structure in order that an equilibrium state can be obtained. The bold architectural treatment of the double Tee facing effectively disguises this movement. The theoretical considerations of this application, which involves the reinforcing of a cohesive soil, have been described elsewhere, Jewell and Jones (1981) (6).

CONCLUSIONS

The use of fabrics and geogrids in practical works is attractive because of the low cost structures which can be constructed. It has been found that using simple construction techniques which permit some adjustment of the facing during construction, constructions free from any distortion problems can be produced. Fears based upon theoretical studies that these structures will distort after construction appear groundless. Provided that the internal design is adopted from current procedures using a reinforcement soil ratio close to (1:1) in the horizontal plane, the dominant design criteria appears to be that of overall stability.

Fig 8.

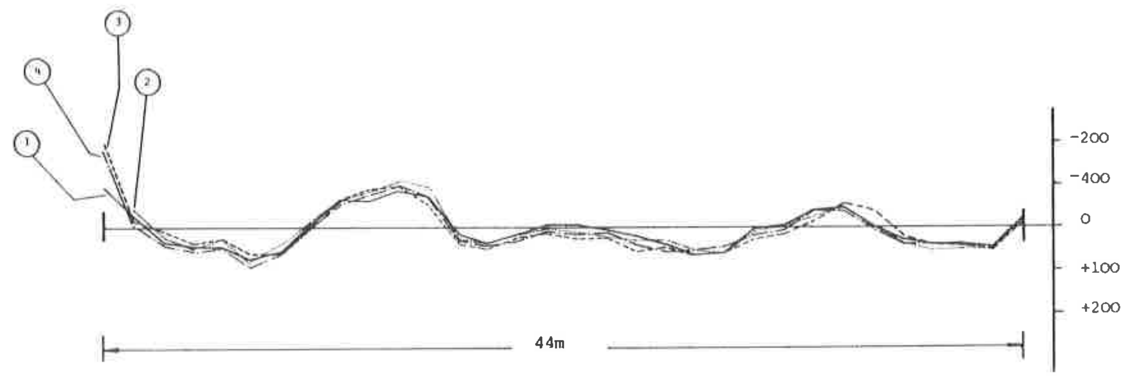

Fig 9. Note: negative records represent forward movement
positive records represent movement into fill

(1) measurements 3.IV.80
(2) " 25.IV.80
(3) " 20. V.80
(4) " 3.VI.80

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Analytical and Laboratory Investigations of Reinforced Clay

Recherches analytiques et en laboratoire sur l'argile renforcée

The current design theories for reinforced granular soils employ a discrete analysis in which soil and reinforcement are considered separately. This design philosophy could in principle be extended to reinforced clays where the short term stability might be assessed in terms of Cu the undrained shear strength of the clay. A novel deviation from this approach involves the concept of a composite theory in which the effects of the reinforcement are assumed to impart an equivalent undrained shear strength Cu'. Ideally structures could then be designed using existing total stress theories with Cu' substituted for Cu. To assess this possibility simple theories are developed to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and finally a reinforced clay wall. Following this a series of model tests are described and test data presented to allow comparison with the theories. Such a comparison shows sufficiently reasonable agreement overall to warrant further research with the aim of defining and calibrating this potentially simple design technique.

Les théories actuelles d'étude des sols granuleux renforcés utilisent une analyse discrète suivant laquelle on examine séparément le sol et le renforcement.Cette philosophie de l'étude pourrait en principe être étendue aux argiles renforcées où la stabilité à court terme peut être évaluée en fonction du facteur Cu (la résistance à la déformation de l'argile non asséchée).Une déviation nouvelle à partir de cette approche implique le concept d' une théorie composite selon laquelle on suppose que les effets du renforcement transmettent une résistance à la déformation non asséchée équivalente Cu'.Dans l'idéal on pourrait céer des structures utilisant les théories existantes de tension totale en substituant Cu' à Cu. Pour juger de cette possibilité on développe des théories simples pour reproduire la compression sous tension plane d'un cube en argile renforcée,de fondations en argile renforcée et enfin d'un mur en argile renforcée.Après cela on décrit une série de tests modèles et on présente des données expérimentales pour permettre de les comparer aux théories.

INTRODUCTION

In designing reinforced soil structures employing non-cohesive fill the strength components of the soil and reinforcement are considered separately using discrete theory. Since drained conditions are deemed to prevail both during construction and the service life of the structure design is based on effective strength parameters. The same approach could in principle be adopted for the design of structures employing cohesive fill but this would only be relevant to the assessment of long term stability. Obviously in the short term, that is during and immediately after construction, design would need to be based on total stress analysis,employing amongst other parameters, the undrained shear strength of the fill Cu. Basic design philosophy would remain unchanged with the reinforcement being considered separately from the fill. A novel deviation from this discrete approach involves the concept of a composite theory in which the effects of reinforcement are assumed to impart an equivalent undrained shear strength Cu'. To quantify this possibility a simple theory was derived based on the plane-strain compression of clay between adhesive plattens. Subsequently plane-strain laboratory tests were carried out to provide test data for comparison with the theory with a view to extending this approach to other applications.

COMPOSITE THEORY

The basic theory developed relates to the plane-strain compression of clay between a pair of rigid adhesive plattens as depicted in Figure 1 which shows plattens of width B bounding an element of clay of thickness S.

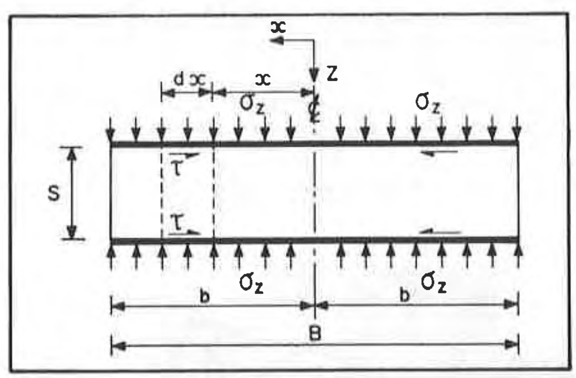

Fig1 Compression between Adhesive Plattens

Compression of these plattens under a vertical normal stress σ_z will ultimately lead to yielding of the clay of undrained shear strength Cu, which will attempt to displace laterally. This movement will mobilise restraining shear stresses between the clay and plattens. The magnitude of these shear stresses may be related to Cu by an adhesion factor α. Consideration of the equilibrium of an element of clay of width dx, Figure 1,leads to equation (1)

$$2\alpha Cudx \; + \; Sd\sigma_z \; = \; 0 \qquad (1)$$

At the free edges of the plattens the vertical stress σ_z may be assumed to locally equal the compressive strength of the clay 2Cu. Thus integrating equation (1) and applying the boundary conditions $x=B/2, \sigma_z=2Cu$ leads to equation (2)

$$\sigma_z = 2Cu\left[1 + \frac{\alpha(B/2-x)}{S}\right] \quad (2)$$

The expression in equation (2) represents local values of σ_z. Further integration leads to the mean value given in equation (3) which is deemed to represent the compressive strength, p, of the composite.

$$p = 2Cu\ (1 + \alpha B/4S) \quad (3)$$

Extension of the normal convention for unconfined loading which defines compressive strength p=2Cu leads to the proposition that equation (3) may be used to represent Cu' the equivalent undrained shear strength of the reinforced clay composite, equation (4)

$$Cu' = Cu\ (1 + \alpha B/4S) \quad (4)$$

2 PLANE-STRAIN TESTS AND RESULTS

To provide test data to compare with the theoretical analysis a number of plane-strain tests were carried out on reinforced and unreinforced remoulded London Clay using a specially constructed 150mm plane-strain apparatus. The basic apparatus, shown in Figure 2, comprises a concrete cube mould with two opposite sides welded to a large rigid base plate and stiffened by heavy welded webs. During preparation of the sample the two remaining sides of the mould are bolted firmly in position. The London Clay employed was thoroughly remoulded by drying, pulverizing in a jaw crusher and subsequently a mechanical grinder, following which water was added to achieve an undrained shear strength of approximately 45kN/m². The clay was prepared well in advance of testing and double sealed in plastic bags to allow for equalisation of moisture content. Reinforcement, which was cut to the exact internal dimensions of the mould, was in the form of a plastic geogrid with a 6mm diamond shape mesh and a total structure depth of 4mm. With the four sides of the mould in position alternate layers of reinforcement and London Clay were placed and compacted to a bulk density consistent with full saturation. Each sample was constructed with a layer of reinforcement top and bottom with the intervening reinforcement layers being at a constant spacing within any one sample. With the sample formed two sides of the mould were unbolted and carefully removed following which a rigid 150mm square loading platten was positioned on top of the sample. The whole assembly was then located in a compression machine and loaded, through the platten, at a vertical rate of strain of 2% per minute. A record was made of the applied vertical pressure, p, and vertical axial strain. To assess the true undrained shear strength of the clay in each reinforced sample an unreinforced 150mm cube sample of clay was prepared from the same clay and at the same time as the reinforced sample. Sets of 76mm x 38mm diameter samples were subsequently taken from the unreinforced sample and tested over a range of cell pressures compatible with the range of vertical pressure p to take account of any lack of saturation. Although the inner surfaces of the plane-strain cell were highly polished and lubricated to obviate any strength enhancement induced by side friction this was no guarantee of compatibility between unreinforced strengths measured using this apparatus and 38mm diameter samples tested in the conventional triaxial apparatus. To quantify any difference the first two tests using the plane-strain apparatus were carried out using no reinforcement. When the resulting drained shear strengths were compared with those obtained using the triaxial test they were found to be only 8% higher and thus for the purpose of the investigation confirmed the compatibility of the two test methods. Following this four tests were carried out with reinforcement at nominal spacings of 19mm, 25mm, 38mm and 50mm. The resulting measured values of p at failure were normalised by dividing by 2Cu the corresponding unreinforced compressive strength measured using the triaxial apparatus. Reinforcement spacing, S, was also rendered dimensionless by dividing by platten width B. These test results are shown in Figure 3 together with the theoretical line obtained from equation (3). The adhesion factor of 0.89 employed in the evaluation of equation (3) was obtained by direct measurement using a 60mm square shear box. Since the comparison of theoretical and test data showed promise it was decided to extend the basic composite theory to the specific cases of reinforced clay foundations and walls with a view to conducting laboratory tests to model these applications and provide test data for comparison.

3 FOUNDATION THEORY

The generation of a composite theory for reinforced clay foundations deviates from that of the previous simple theory in two fundamental points. Firstly since there is diminution of applied vertical stress with depth the

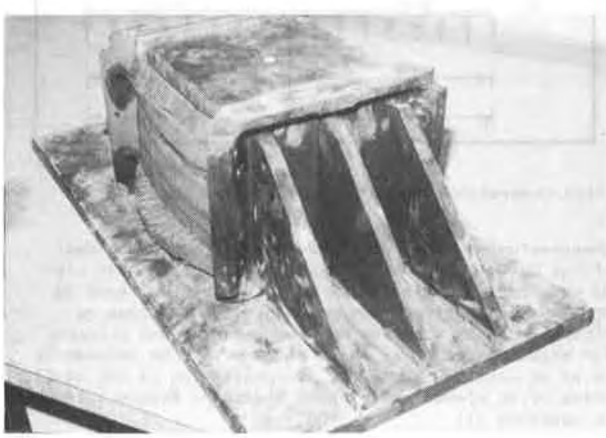

Fig. 2 Plane Strain Apparatus

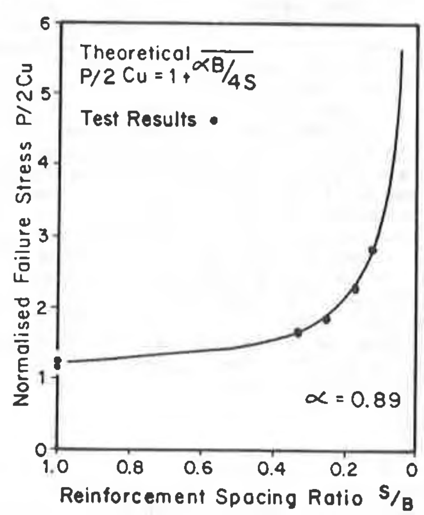

Fig. 3 Plane Strain Test Results

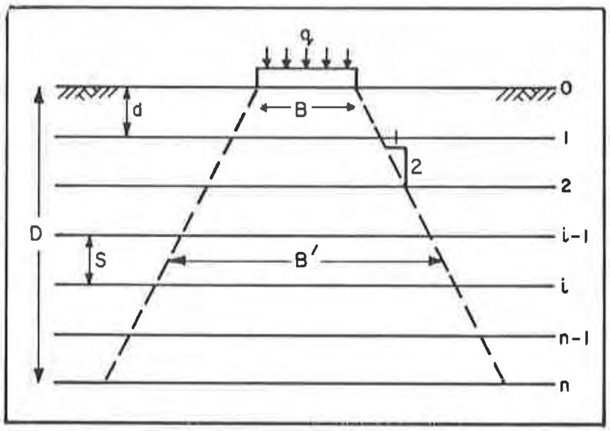

Fig. 4 Diagrammatic Representation of Model Footing

stress intensity at each level of reinforcement will be different. Secondly, since the width of the reinforcement in the clay is large compared with the width of the footing the clay cannot be considered to be unconfined at any horizontal boundary. The first of these problems can be overcome by making an assumption regarding the vertical stress distribution. In the theory presented a very simple 1:2 spread has been taken, Figure 4, which shows that the effect of assuming this, or indeed any other distribution, is to effectively increase the platten width with depth. Consequently it is necessary to define a mean effective platten width over the depth of the reinforced zone D. It follows from Figure 4 that the effective platten width B' between the (i-1)th and ith reinforcing layer is:

$$B' = B + (i - \tfrac{1}{2})S$$

For n layers of reinforcement the mean effective platten width is then given by equation (5)

$$\bar{B} = B + \frac{1}{n}\sum_{1}^{n}(i-\tfrac{1}{2})S = B + \tfrac{1}{2}nS \qquad (5)$$

Obviously the magnitude of n is known, since for a reinforced zone of depth D with a regular reinforcement spacing S it follows that n = D/S.

Having defined a notional platten width $\bar{B}$ consideration may be given to the internal equilibrium of an element such as that shown in Figure 1. This leads to the same generic equation, namely equation (1), which can again be integrated, however, the boundary condition $x=\bar{B}/2$, $\sigma_z=2Cu$ will not apply since in the case of a foundation the clay is confined at the boundary $x=\bar{B}/2$. If the general boundary condition $x=\bar{B}/2$, $\sigma_z=(2+\beta)Cu$, is substituted then equation (1) can be integrated as before to render the mean value of σ_z. This results in equation (6) which is of similar form to equation (3)

$$q = 2Cu\ (1 + \alpha\bar{B}/4S + \beta/2) \qquad (6)$$

Now if the foundation soil were unreinforced equation (6) would reduce to $q=(2+\beta)Cu$ whence taking a direct correspondence between this and the classical expression $q=(2+\pi)Cu$ leads to $\beta=\pi$ and equation (7)

$$q = 2Cu\ (1 + \alpha\bar{B}/4S + \pi/2) \qquad (7)$$

Applying the classical bearing capacity coefficient $N_c=(2+\pi)$ to the equivalent undrained shear strength leads to equation (8)

$$q = N_c Cu' = 2Cu(1 + \alpha\bar{B}/4S + \pi/2) \qquad (8)$$

Evoking equation (5) to allow substitution for $\bar{B}$ and

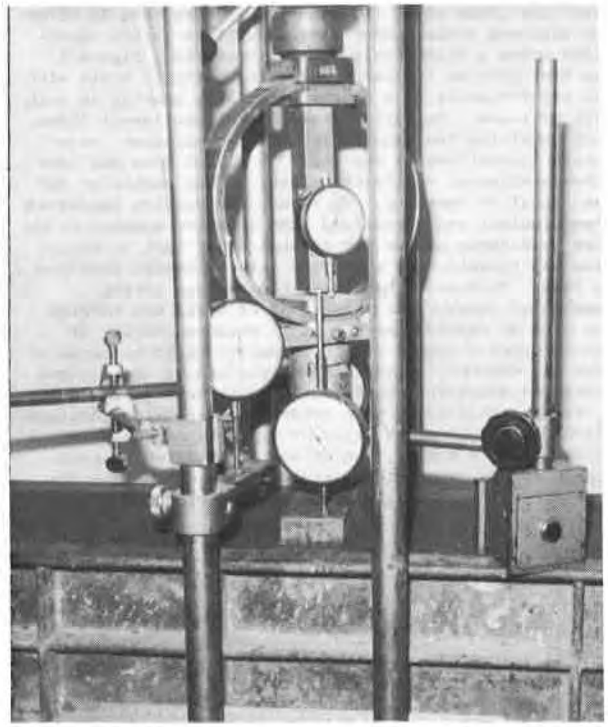

Fig. 5 General View of Model Footing Apparatus

remembering that n=D/S allows equation (8) to be re-arranged to form an expression for equivalent undrained shear strength

$$Cu' = Cu\left[1 + \frac{\alpha(B+D/2)}{2S(2+\pi)}\right] \qquad (9)$$

4 FOUNDATION TESTS AND RESULTS

Model footing tests were conducted using the same clay and geogrid reinforcement employed in the earlier plane strain cell tests. The apparatus consisted of a rigid

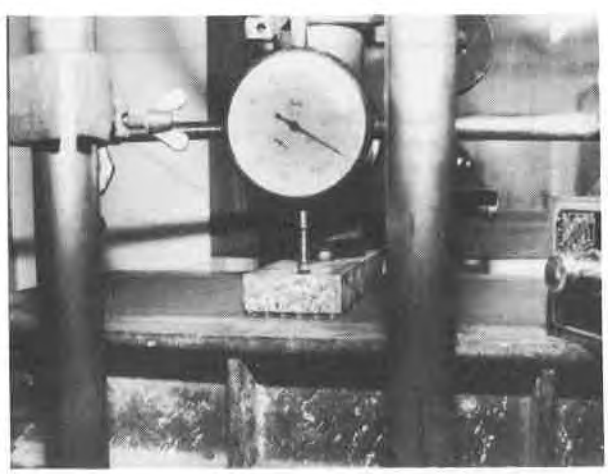

Fig. 6 Detail of Model Footing

steel box 150mm wide, 150mm deep and 710mm long in which the clay was loaded under undrained plane-strain conditions using a rigid strip footing 50mm wide, Figure 5. The test program involved conducting pairs of tests with the reinforcement at a constant vertical spacing in each pair of tests. Employing sheets of reinforcement 325mm long permitted the construction of two adjacent reinforced foundations at one time. A gap of 60mm was left between adjacent reinforced zones, at the centre of the box, to allow sampling. To render the results completely dimensionless and normalised 38mm diameter samples of the clay were taken at the end of each model test to determine any variation in undrained shear strength from test to test. To ensure that the apparatus was giving meaningful results the first pair of tests was carried out with no reinforcement so that measured values of unreinforced ultimate bearing capacity could be compared with the theoretical value N_cCu obtained using measured undrained shear strengths and the classical value of $N_c=(2+\pi)$. This comparison rendered an average undrained bearing capacity coefficient of 5.0 with measured values in the range 4.9 to 5.1 which was in acceptable agreement with the theoretical value of 5.14. Subsequently four pairs of reinforced tests were carried out with reinforcement at constant spacings of 19mm, 25mm, and 50mm. As well as using reinforcement at the prescribed vertical spacings within the clay a layer of reinforcement was attached to the underside of the footing to render a rough base, Figure 6.

For comparison with the theory summarised in equation (9) the measured failure stress q was normalised by dividing by N_cCu using measured values of Cu for each test. The corresponding theoretical plot of q/N_cCu versus S/B is given in Figure 7 together with the normalised test results. As can be seen the agreement between theory and test data is reasonable. The results were further analysed by plotting the normalised bearing capacity ratio, R, this being the ratio of reinforced to unreinforced bearing capacity, against number of reinforcing layers, n, Figure 8, and depth ratio d/B, where d is the depth to the top reinforcement, Figure 9. Consideration of Figure 8 shows bearing capacity ratio generally increasing with number of reinforcing layers as might be expected, however, at low settlement ratios, namely 5%, for n<5 the reinforcement appears to weaken the foundation as indicated by bearing capacity ratios less than unity. This tendency is repeated in Figure 9 which shows R<1 for d/B>0.65 and ρ/B =5%. A more significant

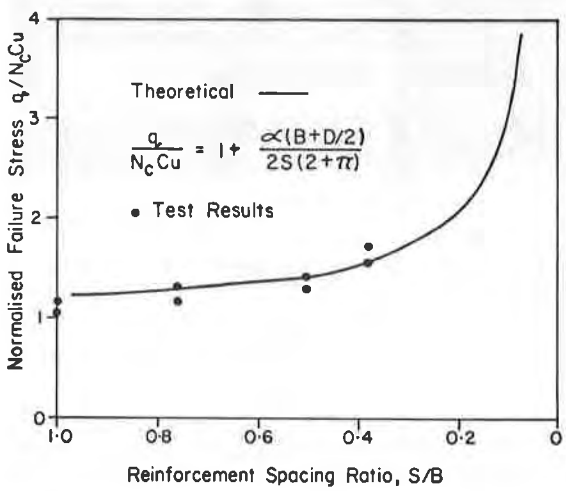

Fig. 7 Model Footing Test Results(q/N_cCu vs. S/B)

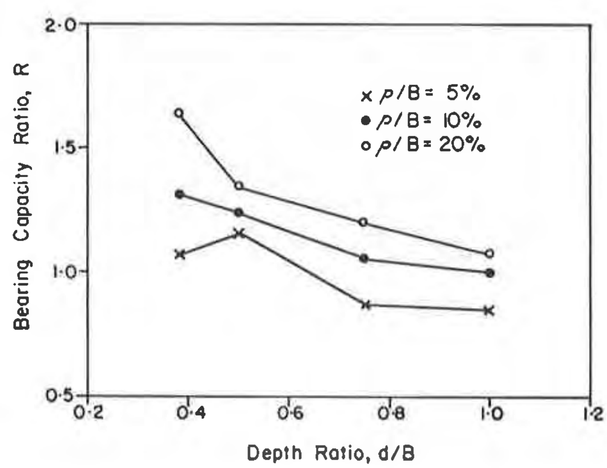

Fig. 9 Model Footing Test Results (R vs. d/B)

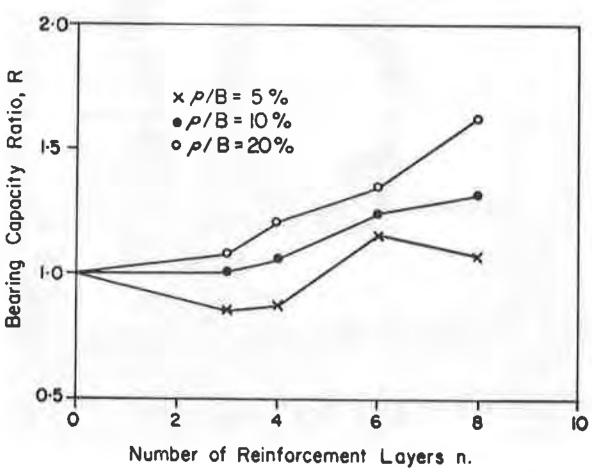

Fig. 8 Model Footing Test Results (R vs. n)

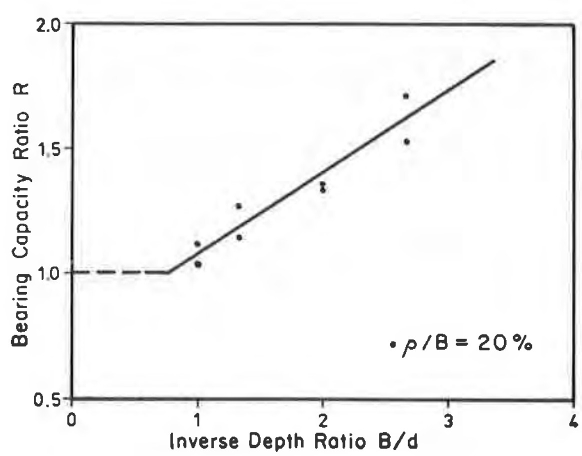

Fig. 10 Model Footing Test Results (R vs. B/d)

Fig. 11 Detail of Failure Mode for n = 8

relationship emerges from Figure 10 which shows a plot of bearing capacity ratio against B/d for ρ/B=20%. Accepting that at this settlement ratio R would not be less than unity Figure 10 indicates that for B/d<0.75 the reinforcement has no effect, however, as B/d increases, that is the top reinforcement rises nearer to the foundation, there is an approximately linear increase in bearing capacity ratio. Since B/d appears to be of major significance the theory presented earlier would need to be extended to allow for d$\neq$S. Additionally there is a need to optimise the number of reinforcing layers. This is evident from Figure 11 which shows the failure mode for eight layers of reinforcement. As can be seen the lowest three or four layers of reinforcement do not appear to have contributed to enhancing bearing capacity. In this case the effective depth, D, of the reinforced zone would be less than assumed in the theoretical evaluation. If D was decreased this would have the effect of slightly lowering the theoretical line shown in Figure 7.

5 WALL THEORIES AND TEST RESULTS

Since the wall theories developed here are, to an extent, a function of the test method subsequently described it is appropriate to present the two simultaneously. In order to put composite theory in clearer focus it is presented in parallel with conventional, or discrete, theory with both of these theories being compared with test results obtained. To assess the theories a series of simulated wall tests was carried out using Kaolin clay again reinforced with plastic geogrids. Due to the impracticality of bringing a laboratory model to failure by self weight only the simulated walls were failed under the application of a vertical surcharge Figure 12. To minimise the vagaries of subsequent total stress analyses the surcharge was applied using a rigid platten that has the effect of inducing failure along a preselected plane. The justification for this and detailed description of the apparatus is given elsewhere, Ingold (1). In essence the objective of the tests conducted was to observe the surcharge intensity, p kN/m², required to cause failure in walls with and without reinforcement. The walls, which were formed in a long, rigid, open ended box to maintain plane-strain conditions, were 150mm high and reinforced at 19mm, 25mm, 38mm or 75mm vertical centres.

One approach to predicting the magnitude of surcharge intensity to cause failure is to apply conventional theory which considers the restoring forces in a reinforced clay wall to be the sum of two discrete forces namely those due to the clay fill and those developed by the reinforcement. To demonstrate this Figure 12 shows a wall of height H with n reinforcing layers at a vertical spacing S. For the particular series of tests reported the reinforcement strength and geometry were selected to obviate tensile failure or pull-out from Zone II, Figure 12. Remembering that the wall has no facing units the remaining mode of failure involves pull-out of the reinforcement from Zone I. Assuming the generation of soil-reinforcement adhesion αCu the pull-out resistance of the ith layer is represented by equation (10)

$$T_i = 2(L-L_i)\,\alpha Cu = 2H\cot\theta\,\alpha Cu[(2i-1)/2n] \qquad (10)$$

The total horizontal restoring force T developed by n layers of reinforcement is obtained by summing equation (10) over n terms

$$T = nH\cot\theta\,\alpha Cu \qquad (11)$$

For the particular platten width employed, b=100mm, $\theta=45°+\phi'/2$ for which equation (11) reduces to equation (12)

$$T = nH\sqrt{K_a}\,\alpha Cu \qquad (12)$$

A theoretical value of p, the surcharge intensity at failure, can be obtained by a simple Coulomb total stress analysis incorporting T, equation (13)

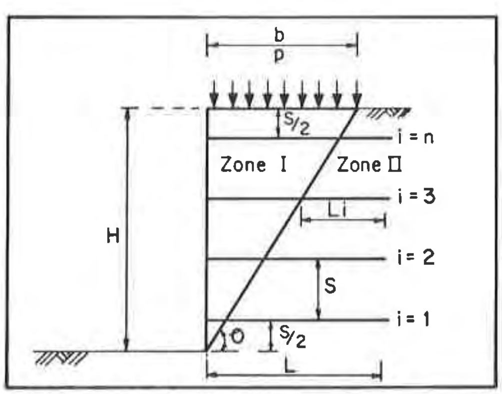

Fig. 12 Model Wall-Arrangement of Reinforcement

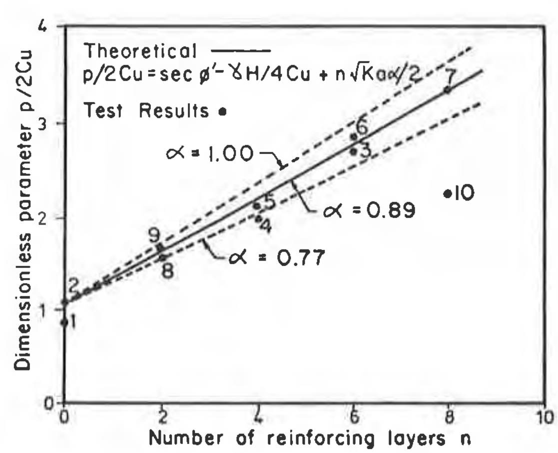

Fig. 13 Model Wall Test Results (p/2Cu vs. n)

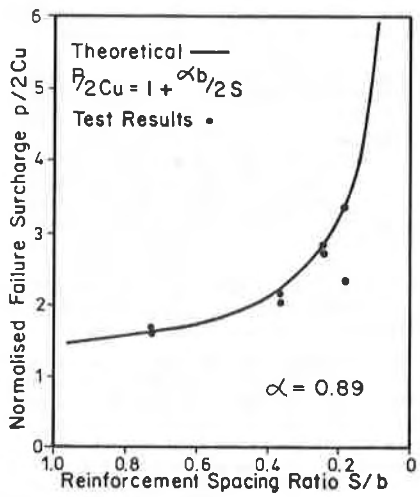

Fig. 14 Model Wall Test Results(p/2Cu vs. S/b)

$$p = 2Cu\sec\phi' - \tfrac{1}{2}\gamma H + n\sqrt{K_a}\,\alpha\,Cu \qquad (13)$$

This equation is rendered dimensionless by dividing throughout by 2Cu and is compared with test data in Figure 13. The range of adhesion factors cited are the maximum, mean and minimum values obtained from shear box tests.

A possible alternative to the above conventional or discrete analysis is composite theory which defines an equivalent undrained shear strength Cu'. Now for an un-renforced wall simple total stress theory predicts a failure surcharge intensity defined by equation (14)

$$p = 2Cu - \tfrac{1}{2}\gamma H \qquad (14)$$

From consideration of internal stability similar to that shown in Figure 1 it is possible to derive equation (15) which gives an expression for equivalent undrained shear strength

$$Cu' = Cu\,(1 + \alpha b/2S) \qquad (15)$$

Substitution of Cu' for Cu into equation (14) leads to equation (16)

$$p = 2Cu\,(1 + \alpha b/2S) - \tfrac{1}{2}\gamma H \qquad (16)$$

For the particular wall tests conducted the $\frac{1}{2}\gamma H$ term was approximately 1 kN/m^2 and was therefore neglected. The resulting theoretical expression is compared with the test results in Figure 14 and shows generally good agreement save for one test where the reinforcement became contaminated with grease thus causing premature failure.

A final reflection on the vagary of total stress analysis concerns the apparent compatibility between the discrete and composite theory if failure is considered to occur along a plane at 45° to the horizontal. In this case, which is equivalent to assuming a surcharge width of b=H as opposed to $b=\sqrt{K_a}H$, the discrete theory predicts a surcharge intensity at failure defined by equation (17)

$$p = 2Cu - \tfrac{1}{2}\gamma H + n\alpha Cu \qquad (17)$$

For a platten width b=H the composite theory may be used to define an equivalent undrained shear strength Cu' for the reinforced clay, equation (18)

$$Cu' = Cu(1 + \alpha H/2S) \qquad (18)$$

In the case of an unreinforced wall the surcharge p is defined by equation (14) as before. Remembering that H/S=n substitution of Cu' from equation (18) into equation (14) leads to an expression identical to that for the discrete theory, equation (17).

6 CONCLUSIONS

Simple theories have been developed to model plane-strain compression of a reinforced clay cube, reinforced clay foundations and finally a reinforced clay wall. Following this a series of model tests were described and test data presented to allow comparison with the theories. Although there are obvious shortcomings the comparisons in general show sufficiently reasonable agreement to warrant further research with the aim of developing and calibrating this potentially simple design technique.

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The Behavior of Geotextile Reinforced Clay Subject to Undrained Loading

Le comportement de l'argile renforcée au géotextile soumise à une charge non asséchée

In the rapid loading of clay soils there is often no opportunity for dissipation of the porewater pressure generated. For reinforced clay this short term, or undrained condition might be associated with instability. Amongst other things the response of reinforced clay to undrained loading will be a function of the nature of the reinforcement. To investigate possible responses three different geotextiles were installed in cylindrical clay samples subject to either rapid shear or shear at constant volume, both of these regimes being consistent with the notion of undrained loading. It was found that even permeable geotextile reinforcement caused a consistent and substantial decrease in strength compared to that of an unreinforced sample, however, as reinforcement spacing decreased the strength of the soil improved and ultimately exceeded that of the unreinforced clay. This response reflects the fact that rapid loading is not necessarily associated with undrained loading if reinforcement, which also acts as a drain, is installed at a sufficiently small spacing.

Dans la charge rapide des sols argileux il n'y a souvent aucune chance que l'eau produite par la pression se dissipe.Pour l'argile reinforcée cette courte période,ou état de non assèchement peut être associée à l'instabilité.La réaction de l'argile renforcée à une charge non asséchée sera fonction,entre,autres choses,de la nature du renforcement. Pour examiner les réactions possibles trois géotextiles différents ont été installés dans des échantillons d'argile cylindriques soumis soit à une déformation rapide soit à une déformation constante en volume,ces deux régimes étant conformes à la notion de charge non asséchée.On a trouvé que même le renforcement géotextile perméable causait une diminution régulière et substantielle de la résistance par rapport à un échantillon non renforcé;cependant,avec la réduction de l'espacement du renforcement la résistance du sol s'améliorait et finalement dépassait celle de l'argile non renforcée.Cette réaction fait ressortir le fait que la charge rapide n'est pas nécessairement associée à la charge non asséchée si le renforcement,qui agit en facteur de drainage,est mis en place avec un espacement suffisamment réduit.

INTRODUCTION

Reinforced soil exhibits several advantages not least of which is speed of construction. In the case of free draining granular backfill rate of construction is of little consequence since even for rapid loading the fully drained condition may be deemed to prevail. This is not necessarily the case for low permeability cohesive fill where rapid construction is likely to be associated with undrained loading. To investigate the possible effects of rapid loading several series of triaxial compression tests were carried out on cylindrical samples of clay reinforced with discs of various geotextiles. Three geotextiles were used namely porous sintered polythene, a thick needle punched felt, and a melt-bonded fabric. Three clay soils were used, however, these all had very similar strength, deformation and consolidation characteristics.

A simple test procedure was followed involving the determination of the strength of the unreinforced clay and the apparent strength of the same clay reinforced with geotextile discs at various spacings. The change in strength caused by the reinforcement is quantified by the introduction of a strength ratio which is simply the measured deviator stress at failure in the reinforced sample divided by the deviator stress at failure in the unreinforced sample. By varying the spacing of a given geotextile in a given soil type it is possible to observe the effect of this parameter on strength ratio. Since all the clays employed exhibited similar properties it is possible to make valid comparisons of the effects of different reinforcing materials. All tests were carried out under conditions of undrained shear,however several of the tests were sheared at a rate consistent with porewater pressure equalisation thus permitting measurement of porewater pressure.

1. SOIL AND REINFORCEMENT PROPERTIES

Three clay soils were used in the investigation,Kaolin clay, boulder clay and London clay. All of these clays were thoroughly remoulded, following which cylindrical samples were formed using a hydraulic press. This process was followed by saturation under back pressure and subsequent consolidation under an isotropic stress regime. The relevant properties of the three clays are summarised in Table 1 which shows the plasticity and effective strength parameters to be similar.

To give some indication of compressibility initial undrained deformation moduli are also given in Table 1. Since the deformation modulus is, amongst other things,a function of consolidation pressure a range of moduli and corresponding consolidation pressures is indicated. A range of geotextile reinforcement was employed as indicated in Table 2.

All of the geotextiles used have permeability, normal to the plane of the material, several orders of magnitude higher than those of the clay. Due to the structures of the synthetic reinforcement it is not possible to define a deformation modulus in the strict sense,however, to permit some comparison with moduli for the soils, Table 2, gives values for a notional initial modulus. This is derived from plane strain tensile test data and the nominal thickness of the structure which allows calculation of a notional stressed area. The structure thickness, especially in the non-woven fabric and the

Table 1. Soil Data.

Soil	Liquid Limit %	Plastic Limit %	ϕ'	Initial Modulus MN/m^2	Consolidation Pressure kN/m^2
Kaolin	57	33	24.3	13	250
Remoulded London Clay	77	31	19.8	7-28	100-400
Remoulded Boulder Clay	68	39	20.5	7-28	50-200

Table 2. Reinforcement Data.

Reinforcement	Material	Structure	Weight g/m^2	Thickness mm	Initial Modulus MN/m^2		Tensile Strength kN/m	
					Min.	Max.	Min.	Max.
Porous Plastic	Polythene	Sintered	3280	4.75	75	110	18.0	18.6
Non-woven Fabric	Polypropylene/ Nylon	Melt Bonded	280	0.70	30	42	8.1	9.0
Felt	Polypropylene	Needle Punched	250	3.20	2	7	5.5	10.9

felt will tend to decrease due to confinement by the soil and tensile loading of the reinforcement. Thus the values given in Table 2 would represent a lower bound. Comparison of the soil and reinforcement moduli show that with the possible exception of the felt the reinforcement deformation moduli are higher than those for the soil.

2. TESTS AND TEST RESULTS

A total of four series of tests are reported. Sample preparation involved thoroughly remoulding the clay, followed by hydraulic pressing of standard 204mmx102mm diameter samples at moisture contents and dry densities theoretically consistent with full saturation. All reinforcement was cut to form discs having the same diameter as the soil sample. To allow introduction of the reinforcement each sample was cut into a number of thick discs, of equal height, using a cheese wire. Each soil sample was then reassembled with a disc of reinforcing material being introduced between each soil disc so producing a multi-reinforced sample. All test results are reported in the same format namely the failure strength ratio, R, and inverse aspect ratio μ. The strength ratio is defined as the ratio of measured compressive strength, or failure deviator stress, of the reinforced sample to that of an unreinforced sample with the latter being either measured directly for an unreinforced control sample or derived from some other measured soil property which had been previously related to the unreinforced strength. The inverse aspect ratio is simply defined as d/h where d is the sample diameter and h is the height of an individual soil cell within the multi-reinforced sample.

In the first series of tests the effects of a permeable reinforcement were investigated using a porous plastic reinforcement within multi-reinforced 102mm diameter Kaolin samples. All samples were sheared unconfined at a rate of strain of 2% per minute with unreinforced undrained shear strengths being determined from a previously derived moisture content-undrained shear strength relationship, Ingold (1). On this occasion eight pairs of reinforced samples were tested with a wide range of inverse aspect ratios. The results, which are summarised in Table 3, show strength ratios falling below unity for small inverse aspect ratios. To widen the scope of the investigation the next two series of tests were carried out using multi-stage consolidated undrained tests with porewater pressure measurement on remoulded boulder clay reinforced with geotextiles in the form of a non-woven fabric, Test Series No. 2, and a needle punched felt, Test Series No.3. Results for these two series are given in Table 3. Sample preparation methods for the final test series deviated considerably from those previously described. Although use was again made of Kaolin clay and porous plastic reinforcement, the clay was consolidated from a slurry to produce an isotropic sample whose unreinforced strength could be related to effective consolidation pressure. In place of the multi-reinforced sample a single unit cell of soil was employed this being bound top and bottom by a porous plastic reinforcing disc. To ensure compatibility of stress regimes developed within unit cell and multi-reinforced samples it was necessary to minimise friction between the outer faces of the reinforcing discs and the

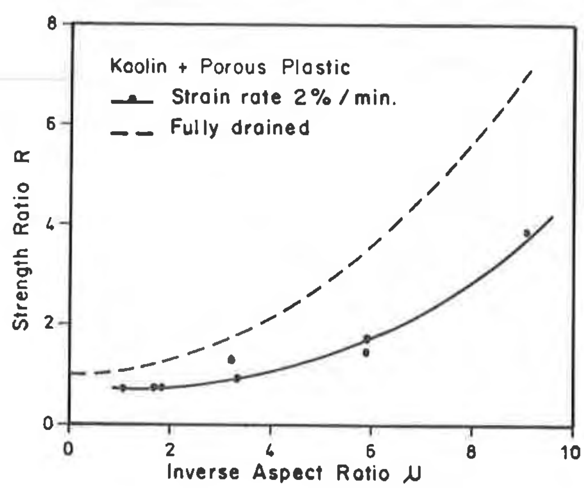

Fig. 1. Results of Test Series No. 1.

Table 3. Summary of Test Results.

Test Series	Test No.	Test Type	Reinforcement	Soil	Inverse Aspect Ratio	Strength Ratio
1	1	UCUD*	Porous Plastic	Kaolin	1.09	0.71
	2	"	" "	"	1.72	0.79
	3	"	" "	"	1.79	0.79
	4	"	" "	"	3.23	1.30
	5	"	" "	"	3.33	0.94
	6	"	" "	"	5.88	1.45
	7	"	" "	"	5.88	1.70
	8	"	" "	"	9.09	3.92
2	9a	CUD+PWP[+]	Non-woven Fabric	Remoulded Boulder Clay	4.17	1.08
	9b	"	" "	"	4.43	1.15
	9c	"	" "	"	4.72	1.17
3	10a	"	Felt	"	4.29	1.22
	10b	"	"	"	4.74	1.30
	10c	"	"	"	5.18	1.32
	11a	"	"	"	4.30	1.13
	11b	"	"	"	4.74	1.18
	11c	"	"	"	5.18	1.23
4	12	CUD+PWP°	Porous Plastic	Kaolin	3.41	1.57
	13	"	" "	"	4.65	1.98
	14	"	" "	"	6.99	2.50
	15	"	" "	"	13.89	3.21

* Unconsolidated unconfined undrained tests - strain rate 2% per minute.

[+] Multi-reinforced multi-stage consolidated - undrained with porewater pressure measurement.

° Unit reinforced cell consolidated - undrained with porewater pressure measurement.

pedestal and top cap of the triaxial apparatus. This was achieved by the introduction of thin lubricated rubber end plattens, the lower platten being provided with a central opening to allow the measurement of porewater pressure via the conventional porous stone mounted on the pedestal. Using this arrangement four consolidated undrained tests with porewater pressure measurement were carried out at different inverse aspect ratios with the results being summarised in Table 3, Test Series No. 4.

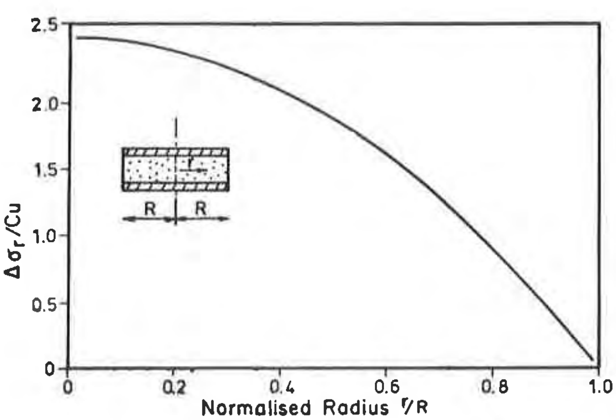

Fig. 2. Radial Variation of $\Delta\sigma_r$/Cu

3. ANALYSIS OF TEST RESULTS

The results of Test Series No. 1, which are plotted in Figure 1, were analysed using a second order regression analysis which rendered a coefficient of correlation of 0.983 . This is shown as a solid line in Figure 1 together with the results of a similar analysis, shown in broken line, for fully drained test results reported elsewhere, Ingold (1). Reference to the solid line shows the strength ratio dropping below unity for inverse aspect ratios less than approximately 4. Since such values of μ are associated with long vertical drainage paths the rapid rate of shear of 2% per minute is associated with truly undrained loading in which case premature failure is induced. It can be shown from earlier work, Yang (2), Ingold (3), that the introduction of reinforcement leads to an enhancement of radial total stress, $\Delta\sigma_r$, and a corresponding increase in global total stress $\Delta\sigma_3$. For a saturated soil this would be associated with a change in porewater pressure of equal magnitude and therefore no change in effective stress, Skempton (4). If, however, consideration is given to the radial distribution of $\Delta\sigma_r$ and therefore by implication $\Delta\sigma_3$ it can be seen that this is not uniform with $\Delta\sigma_r$ decreasing radially, Figure 2. As a consequence of this the highest porewater pressures, which are continuously generated near the centre of the sample during the initial stages of loading, migrate radially so causing a net decrease in minor principal effective stress throughout the sample. For a soil obeying the Mohr-Coulomb failure criterion a change in minor principal effective stress is associated with a change in strength. Since in this case the minor principal stress decreases there would be a corresponding decrease in strength. For values of μ>4 the vertical drainage paths

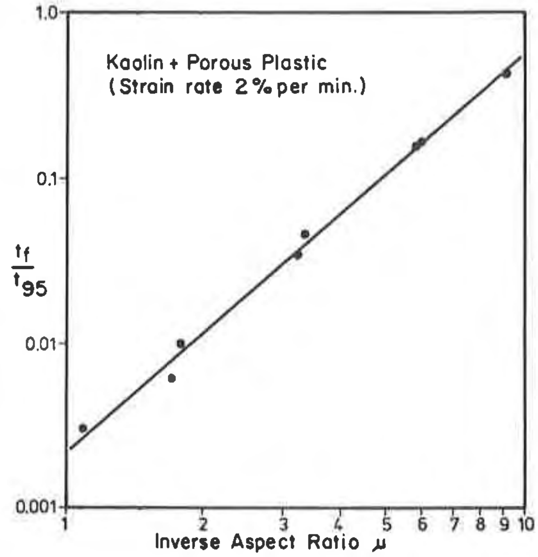

Fig. 3. Variation of t_f/t_{95} with μ.

become shorter in which case even this rapid rate of shear is associated with partial drainage. In consequence of this the deleterious porewater pressures associated with an increase in total stress, $\Delta\sigma_3$, are allowed to partially dissipate in which case there is an increase in effective stress, $\Delta\sigma_3'$, leading to an enhanced strength. For the highest value of $\mu \cong 9$, the regression analysis is almost parallel to that for the fully drained condition shown by the broken line in Figure 1. That increasing μ is associated with increasing degree of drainage becomes evident from Figure 3 which shows a plot of actual failure time, t_f, divided by theoretical failure time for 95% drainage, t_{95}, as a function of μ. As can be seen values of t_f/t_{95} approach unity, that is the fully drained condition as μ becomes progressively larger. The logical implication of this is that if porous reinforcement is installed at sufficiently close centres then fully drained conditions can be made to prevail under rates of loading, or construction, that would normally be associated with undrained loading.

The effects of other geotextile reinforcements were investigated in Test Series Nos.2 and 3. The results are given in Table 3 which shows strength ratios varying between 1.08 and 1.32 for inverse aspect ratios in the range 4.17 to 5.18. Remembering that these tests were executed under truly undrained conditions it is remarkable that there was any strength increase at all since an improvement in strength must be associated with an increase in minor principal effective stress, $\Delta\sigma_3'$. To explore this phenomenon further Test Series No. 4 was carried out using Kaolin clay, consolidated from a slurry and the porous plastic reinforcement employed in earlier tests. As for the non-woven geotextiles the test regime was consolidated-undrained with porewater pressure measurement, however, tests were conducted on unit cells, rather than multi-reinforced cells to avoid any ambiguity in porewater pressure measurement. Results from these tests given in Table 3 confirm that the strength ratio does in fact increase with increasing inverse aspect ratio. To probe the possible mechanism of this strength increase it is necessary to attempt a more detailed analysis of the results.

Earlier radiographic analyses, Ingold (1), indicated that although there is substantial rotation of principal stress axes the measured vertical effective stress in the reinforced sample can be approximated to the major principal effective stress. If it is assumed that failure within the reinforced soil occurs at the same principal effective stress ratio, K, as unreinforced soil, then a mean value of minor principal effective stress, $K\sigma_1'$, can be backfigured from the measured value of major principal stress, σ_1'. The difference between this derived value of mean minor principal stress acting within the sample and that applied at the boundary of the sample σ_3' may be taken to be the mean increase, $\Delta\sigma_3'$, induced by the reinforcement. This condition is represented by equation (1):

$$\Delta\sigma_3' = K\sigma_1' - \sigma_3' \qquad (1)$$

Calculated values for an unreinforced control sample and four reinforced samples are shown in Table 4 together with the corresponding values of inverse aspect ratio μ and strength ratio R.

As can be seen increasing R is associated with increasing $\Delta\sigma_3'$ and μ as expected. What was not expected was that the measured porewater pressure at failure, u, also increased with μ, Table 4. Such an increase in porewater pressure would be expected to be associated with a decrease in strength. Thus the results on cursory examination lead to the apparent inconsistency of an increase in effective stress, $\Delta\sigma_3'$, associated with an enhanced porewater pressure at failure. The only explanation for this is that there must be an increase in total stress, $\Delta\sigma_3$ which is greater than the excess porewater pressure at failure, Δu by an amount $\Delta\sigma_3'$. Inspection of equation (2) shows that this requirement is consistent with the laws of effective stress:

$$\Delta\sigma_3' = \Delta\sigma_3 - \Delta u \qquad (2)$$

A possible mechanism controlling these apparently conflicting requirements can be deduced from Skempton's general porewater pressure equation which can be applied to the shear stage of the reinforced samples, equation (3):

$$A = \frac{u - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \qquad (3)$$

The resulting calculated values of the porewater pressure parameter A are shown in Table 4, as the values of the

Table 4. Analysis of Test Results Test Series No. 4.

Test No.	μ	R	$\Delta\sigma_3'$ kN/m²	u kN/m²	Δu kN/m²	$\Delta\sigma_3$ kN/m²	A
Control	0.50	1.00	0	124	0	0	0.70
12	3.41	1.57	20	180	56	76	0.51
13	4.65	1.98	45	192	68	113	0.33
14	6.99	2.50	71	221	98	169	0.19
15	13.89	3.21	118	237	113	231	0.02

porewater parameter that would be required for the corresponding values of $\Delta\sigma_3'$ to be realized. As can be seen these values, which are for the reinforced composite, are lower than the measured A value of 0.70 obtained for the soil alone from the unreinforced control sample. Since the A value of the soil alone may be taken as a constant of the soil for a particular stress history and stress level it follows that any modification of the A value of the reinforced composite must stem from the reinforcement. To investigate this a sample of reinforcement of the same diameter and same mean height as that used in the reinforced soil sample, was tested to measure its A value directly. This was found to be 0.08. A similar measurement for the felt reinforcement rendered a mean value of 0.27 compared to 0.75 for the soil alone. The consequence of these low A values in the reinforcement is that the high porewater pressures generated in the soil by $\Delta\sigma_3$ migrate towards the reinforcement which does not exhibit such a high porewater pressure response. Obviously since the porewater pressure in the body of the soil is reduced there is an increase in the effective stress $\Delta\sigma_3'$ and a corresponding increase in strength ratio. As the reinforcement spacing decreases the magnitude of $\Delta\sigma_3$ and therefore porewater pressure increases. Since the ability of the reinforcement to reduce porewater pressure is limited its effects begin to become suppressed and the strength ratio does not increase at the same rate as it might in a drained loading situation. This is shown clearly in Figure 4 and

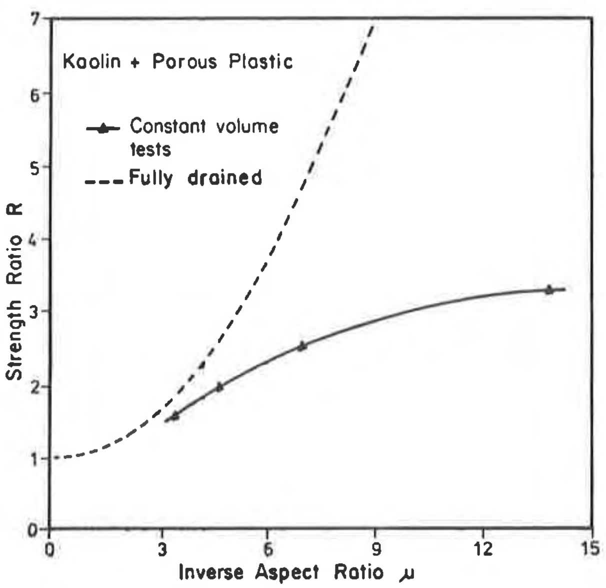

Fig. 4. Results of Test Series No. 4.

follows from Table 4 which shows the "required" A value decreasing with increasing μ. At the highest inverse aspect ratio tested the required A value of 0.02 is slightly less than that of the reinforcement alone therefore the implied increase of $\Delta\sigma_3'$ of 118kN/m^2 represents the maximum that could ever be developed by the particular reinforcement and test conditions employed.

4. CONCLUSION

A total of fifteen triaxial compression tests were carried out on samples of very similar clay soils using three different reinforcing materials. All the tests were ostensibly sheared undrained either by virtue of rapid rate of shear or, more positively by ensuring shear at constant volume. The phenomenon of strength reduction under rapid loading was observed in clay reinforced with porous plastic, however, this reduction in strength was only observed at large reinforcement spacing where a rapid rate of shearing is consistent with undrained loading. With decreasing reinforcement spacing and consequently decreasing vertical drainage path length there is a partial dissipation of porewater pressure permitting an apparent increase in minor principal effective stress and therefore compressive strength. The logical implication of these results is that permeable reinforcement placed at sufficiently close centres might allow fully drained conditions to prevail at rates of construction normally associated with undrained loading.

On investigating the non-woven fabric and felt reinforced samples under truly undrained conditions it was found that they enhanced compressive strength. This finding appeared anomalous since a gain in strength is associated with an increase in effective stress, however, the previous tests indicated that under undrained conditions there is likely to be a decrease in strength. Further tests, involving unit cells of clay reinforced with porous plastic showed that although there was a high porewater pressure induced this appears to migrate to the reinforcement which exhibits a much lower pore pressure parameter A than the soil alone. In consequence of this there would be a reduction in pore water pressure in the soil resulting in an increase in minor principal effective stress and hence compressive strength. In the particular tests reported the strength enhancement was limited, at high inverse aspect ratios, by the finite ability of the reinforcement to depress induced porewater pressure. Finally it must be remembered that the data presented relates to laboratory tests carried out on saturated clay loaded under an axisymmetric stress regime. In all cases failure was by bond as opposed to tensile failure of the reinforcement which was applied in the form of a continuous disc. These conditions deviate considerably from those likely to prevail on site where the stress regime would normally be plane-strain and the soil partly saturated.

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Bearing Capacity Tests on Fiber-Reinforced Soil

Essais de force portante sur des sols renforcés par des fibres

The use of synthetic and other man-made fabrics for soil improvement is oftentimes not within common reach in developing countries due to inavailability of high grade material and high cost of suitable imported fabrics. Work is currently being undertaken aimed at utilising locally available material to achieve similar objectives. The results of laboratory model tests presented are on bearing capacity experiments on a dry sand reinforced with a locally obtainable rope fiber material and also layers of crushed rock. Parameters studied were the influence on the bearing capacity of a square footing of vertical spacings of reinforcement layers, depths below the footing of the first layer and number of layers. A comparative study of the two materials used showed that although bearing capacities with crushed rock layers were improved by a factor of about two, experiments with the rope fiber material indicated that improvement factors could be as high as three. This was shown to be due to the tensile strength of the rope material as against zero in the case of the crushed rock.

L'utilisation de textiles synthétiques n'est généralement pas à la portée des pays en voie de développement du fait du coût élevé des textiles importés. Le but de ce travail est d'utiliser à la place des matériaux locaux. Les essais présentés sont relatifs à du sable sec renforcé avec des fibres locales, ainsi que des couches de gravier concassé. On a étudié l'influence des paramètres suivants sur la force portante d'une semelle carrée: espacement entre les couches de renforcement, profondeur de la première couche de renforcement sous la semelle et nombre de couches. Une étude comparative des deux matériaux montre que bien que les forces portantes avec des couches de gravier concassé aient été améliorées par un facteur de deux, les essais faits avec les fibres ont montré que le facteur d'amélioration pouvait aller jusqu'à trois. On a pu montrer que ceci était dû à la résistance à la traction des fibres, comparée à zéro dans le cas du gravier concassé.

INTRODUCTION

The application of the reinforced earth technology is fast becoming very popular among designers of retaining structures in developed countries. A lot of encouragement however still needs to be given to the application of the technique in the less developed countries as the benefits of lower cost of construction in both time and money is more relevant in these countries. Incidentally, however, the use of reinforced earth in construction is very native in the local construction of some of those countries. In Nigeria as in many developing countries, the use is commonplace in many village constructions. Rope fibers and bamboo sheets are used to strengthen rural road bases and the soil below low-cost low-rise buildings. Vertically arranged rectangular grids of bamboo sheets and stalks of palm branches are also used as the central core around which mud walls are built because of the resulting higher strengths and resistance to cracks and crack propagation that such composite walls possess.

Since the pioneering work of Vidal (14, 15), much work has been done by many researchers (3 - 13) on the analysis, design, testing, and construction of reinforced earth structures. Although much of this work relates to earth retaining structures, it has been shown (3, 5, 6) that the reinforced soil technology is also applicable to bearing capacity problems. The need to channel local technology in developing countries to the design and construction of low-cost highway and housing projects cannot be overemphasised (2) and an earlier work of the authors (3) is an attempt in this direction. It was mainly research into the use of local rope fibers to reinforce granular soils. The problem with the use of such fibers is that they are biodegradable and therefore are prone to being unsuitable in housing or road design projects. As the water table in the soil rises above the level of the strip reinforcements, the strips become weakened. Also, they are prone to termite attack. This paper aims at exploring the suitability of an alternative type of reinforcement that would be non-biodegradable and yet be low-cost. In place of rope fibers therefore, crushed rock has been used and a comparison is being made of its efficiency as compared with the rope materials.

LABORATORY - SCALE MODEL EXPERIMENTS

The model tests were performed in a square-based wooden box 1.0m wide with a depth of 0.7m. The soil material was unformly graded dry sand passing through British Standard (B.S.) No. 14 sieve and retained on B.S. No. 200 sieve; D_{50} = 0.43mm and D_{10} = 0.14mm; specific gravity = 2.78. The method of sand compaction resulted in a constant density of 1,700 kg/m^3 and a friction angle of 38°. For the crushed rock material, the gradation was also uniform, the partide sizes varying between 5mm and 20mm. The gradation curves are shown in Fig. 1

The rope reinforcement material has been described elsewhere (3). It was used in strips that were 10mm wide and 0.03mm thick, the tensile strength being 80N/mm^2 and the angle of soil-tie friction 12°. The model footing was a 100mm square-sided steel plate 13mm thick. The geometry of the test arrangements for

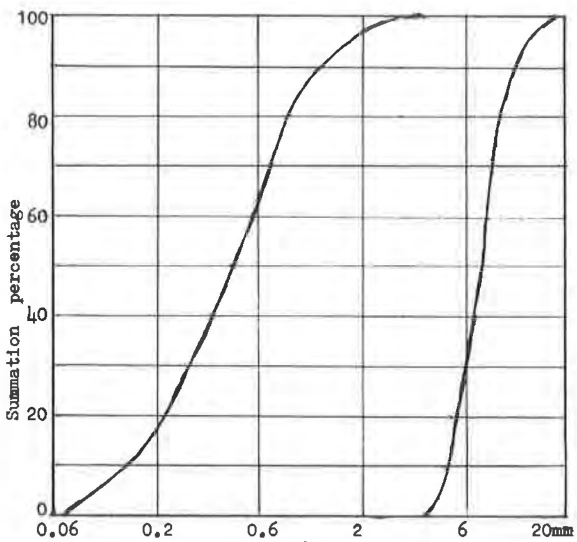

Fig. 1

both types of tests - with rope fiber and crushed rock reinforcements - are shown in Fig. 2. The method of soil placement and the loading arrangements have been described elsewhere (3) and so also is the method for estimating the ultimate bearing capacity in each test (1).

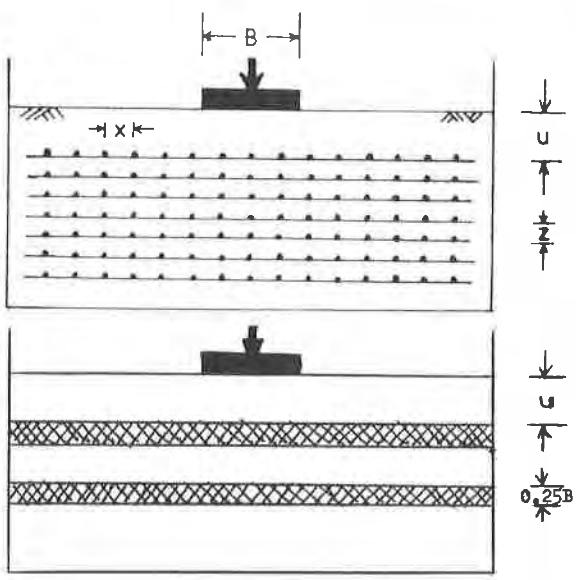

Fig. 2 Geometry of the bearing capacity test series.

The reinforcing strips were almost as long as the 1.0m sides of the smooth bed of compacted sand arranged in horizontal square grids in directions parallel to the sides of the footing and the box. In each test, all the strips in one direction were first laid and those in the other direction were then laid on top. The middle strip in each direction of each layer always passed directly under the center of the footing. In the case of reinforcement with crushed rock, the gravels were spread evenly at predetermined horizontal locations within the soil mass ensuring that the thickness of each gravel layer was 25mm, that is 0.25B, B being the width of the footing. The ultimate bearing capacity q_o of the unreinforced soil was found to be 91 kN/m^2. This was used as comparator for all subsequent tests. A bearing capacity efficiency was defined for the purpose as the ratio of q, the ultimate bearing capacity of the reinforced soil, to q_o. The tests performed were to determine how the ratio q/q_o was affected by the various parameters being examined.

EXPERIMENTAL RESULTS

Crushed Rock Reinforcements

Three series of tests were performed to examine how the bearing capacity efficiency is influenced by the depth of the topmost layer below the footing base, the vertical spacings between layers and the number of layers.

(a) Depth of top layer below footing:- It has been shown before (3) that irrespective of the number of layers involved, the trend of the effect of depth of top layer below the footing base is qualitatively the same. The influence of this parameter on the capacity efficiency is shown in the typical graph in Fig. 3 for N = 1. The peak of the curve was obtained for the situation when the gravel layer was on top of the sand mass and directly below the footing (u = 0). The efficiency is generally high up to u = 0.25B. (Note that the thickness of gravel layer effectively makes the surface of the soil below the layer an extra 0.25B below the footing.)

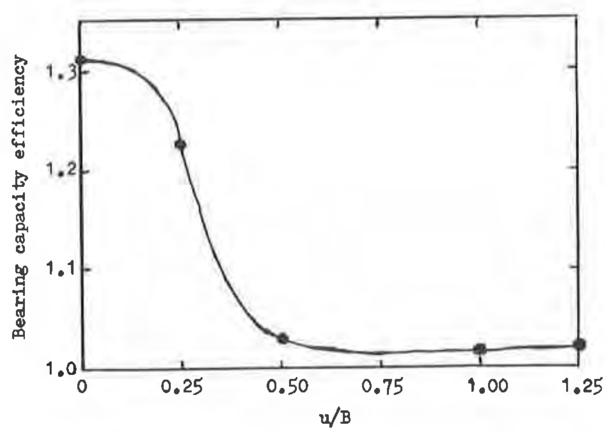

Fig. 3 Efficiency variation with depth of first layer (N = 1)

It was observed that the failure patterns of the soil-gravel mass below the footing were different for u = 0 and for other u - values. With the gravel in direct contract with the footing, failure was by punching of the gravel immediately below the footing into the sand below. With other u - values, there was squeezing of the sand immediately below the footing and above the gravel layer. The resulting effect of these on the settlement characteristics (not shown) was that footing settlement - to - failure for sand in contact with the footing was much compared with the u = 0 case. In this latter case, there was an initially high increase in the load supported with very little settlement. The post-yield behavior was however that of a brittle medium, the ultimate bearing capacity being little more than that at the end of the linear (elastic) portion of the load-displacement curve.

(b) Vertical spacings between layers:- The influence of vertical spacings between layers of reinforcement on bearing capacities is shown in the typical curve in Fig. 4 for N = 3 and u = 0.25B. For z - values up to 0.25B, the efficiency was approximately constant showing that under the first layer of reinforcement, block action took place with the sand and gravels acting as a single entity. With higher z-values, the layers became

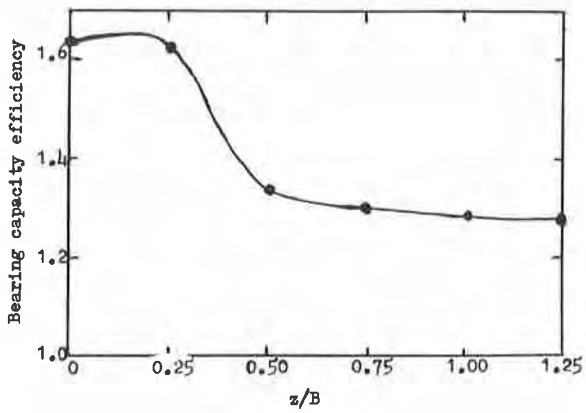

Fig. 4 Efficiency variation with vertical spacing between layers (u = 0.25B, N=3)

sufficiently far apart to enable the shearing of the sand in-between layers. The behavior of the composite soil thus approached that with only one layer of reinforcement as the lower layers became farther and farther away from the zone of influence of the footing. Thus, with z increasing infinitely, the efficiency approached the level of that of the footing when supported on one layer (Fig. 3).

(c) Number of layers:- The influence of the number of layers on bearing capacity efficiency in turn depends on the location of the topmost layer, as described above. Thus, two sets of tests were performed in this series, with u = 0 and u = 0.25B. The results are shown together in Fig. 5. It is clear from these curves that the u = 0 condition gives higher efficiencies than u > 0.

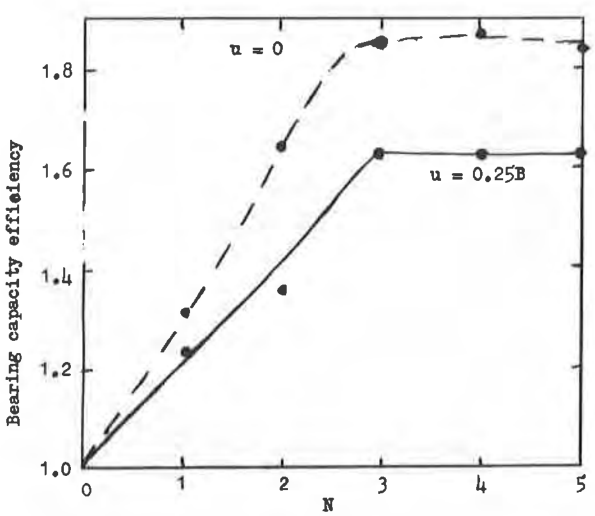

Fig. 5 Efficiency variation with number of layers (u = 0 and u = 0.25B)

However, irrespective of the value of u, it is apparent that little change occured in efficiency after the use of three layers indicating that the addition of more layers of reinforcement after the third does not contribute to bearing capacity improvement.

Rope Fiber Reinforcements

Along the same lines as the crushed rock reinforcement tests, the results of three series of tests are reported herein.

(a) Depth of top layer below footing:- A typical curve is shown for z = 0.5B, N = 5 and horizontal spacings between strips, x = 0.5B in Fig. 6. The peak of the curve was obtained at u = 0.5B. For u-values up to 0.5B, there was a slight increase due to the fact that

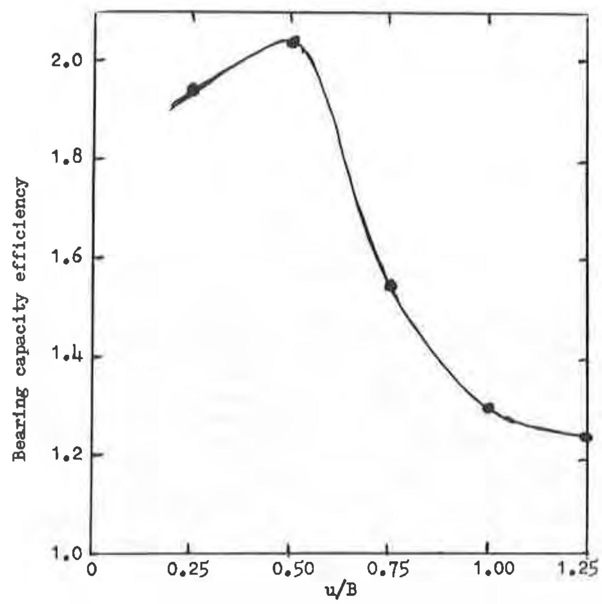

Fig. 6 Efficiency variation with depth of first layer (x = z = 0.5B, N = 5)

the topmost layer was positioned too close to the footing base and its effect would be to make the next layer more effective. Thus, because for u - values less than 0.5B the soil mass above the first layer was too small to generate enough frictional resistance for the footing, there was a probable failure due to the strips being depressed by the elastic wedge of soil below the footing thus reducing conditions to one whereby the depth of top layer = u + z. For greater u - values, the efficiency decreased and approached unity as u was infinitely increased. Note that the strength of the strips was always achieved through their tensile strength whereas for the gravels with no tensile strength, this was achieved through the compressive strength of each gravel and through the intergranular contact stresses.

(b) Vertical spacking between layers:- Typical results of the variation of bearing capacity efficiency with z are shown in Fig. 7 for many values of horizontal spacings x between strips. The behavior is similar to that of crushed rock reinforcements described above (Fig. 4). For z greater than B, very little capacity improvement was achieved for all x-values because of the magnitude of z. In the behaviors for gravel and strip reinforcements, the results with strips that can be compared with those of gravels would be for x = 0 when all strips are placed side by side with no spacing between them.

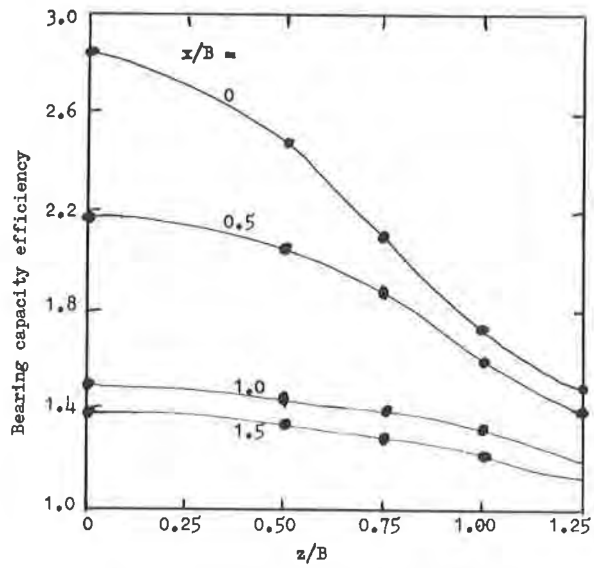

Fig. 7 Efficiency variation with vertical spacing between layers (u = 0.5B, N = 5)

It has been shown by Binquet and Lee (6) that three modes of footing failure can be identified depending on u and z: for high u - values, soil shear would take place completely above the layers of reinforcement. For low u but with few strips or with short ties irrespective of number of layers, failure is more by strip pullout. For low u, many layers and long strips, failure is more likely by fracture of the upper layers of strips. In the experiments, it was observed that for z less than 0.5B, block action of the soil - tie composite was evident. For high z up to z = B, tie break was observed but for z in excess of B, tie pullout resulted.

(c) Number of layers:- Unlike with gravels, the strips had no compressive strength and therefore, with u = 0, the behavio was as an unreinforced soil mass. With reasoning similar to that of results from gravel reinforcement, it is evident in Fig. 8 that the addition of more layers after the third does not contribute much to the bearing capacity improvement.

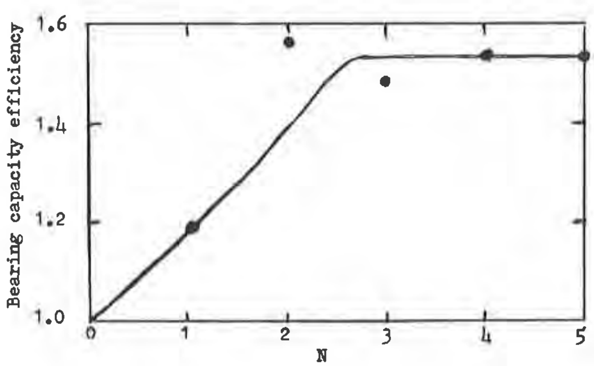

Fig. 8 Efficiency variation with number of layers (u = 0.5B, x = z = 0.5B)

CONCLUSIONS

Results obtained from using the two types of reinforcements indicate that some bearing capacity improvement is obtained with the use of the reinforcement materials, the degree of improvement depending on depth below the footing of the first layer, vertical spacings between layers and number of layers of reinforcement. Fig. 7 also shows that bearing capacity efficiency also depends on the horizontal spacings between strips of fiber reinforcements. It was shown that depending on the arrangement of the reinforcements, improvement in capacity values by a factor of up to three times that of the unreinforced soil can be obtained with the rope fiber material . However, the improvement factor with the crushed rock was not more than two.

A comparison of the results show similar behaviors with crushed rock and rope fiber reinforcements used in the sand. However, it is worth noting that the mechanisms of behavior are different. With strip reinforcements, strength is derived entirely from the tensile strength of the reinforcements material which in turns governs the maximum soil-tie frictional resistance that can exist on the strip surfaces. With crushed rock, strength is derived from the compressive strength of the reinforcement as well as the intergranular contact stresses. Observation at the end of each test with the crushed rock indicates that the strength that could normally be derived from the rock material per se is grossly undermined by the fact that the sand above and below the gravels are forced into the voids in the gravel, thereby causing stress release in the sand, and also that no appreciable soil - gravel friction is achievable (because there is no continuity in the gravel material) especially when compared with the strips. The problem of stress release in the sand does not exist with strip reinforcements.

It appears that the problem pointed out in the earlier work of the authors (3) concerning the biodegradable nature of the rope strips is not helped much by replacing the strips with gravels. To achieve the goal then of using locally obtainable raw materials as reinforcement material, an alternative high tensile strength strip material that is locally available is still required provided it is not biodegradable. Alternatively, the quest for determining what locally available methods and materials can be used to treat the rope fiber reinforcements to render them waterproof and free from insect attack is still much alive.

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Recent Experience with Fabric-Faced Retaining Walls

Expérience récente avec de soutènemente à façade en géotextile

SUMMARY

Three experimental retaining walls were constructed using a composite, fabric facing supported by a Reinforced Earth mass. Because the basic components are strong, light weight and relatively inexpensive, it is expected that such walls could have many different uses on both military and civilian projects in remote areas. This paper briefly describes the intended purpose of the test walls, the composition of the fabric facing, the general construction sequence, and a discussion of potential uses for this type of construction. The use of the composite fabric for erosion protection is also presented.

SOMMAIRE

Trois murs de soutènemente experimentaux ont été construit avec un géotextile composé et soutenu par un massif de Terre Armée. Puisque les constituants élémentaires sont solides, légèrs et assez économiques, il est prévu que ce genre de mur peut avoir plusieurs usages pour projects militaires et civils dans des endroits lointains. Cette note technique décrit brièvement l'objet des murs d'essai, la composition de la peau en géotextile, la méthode de construction ainsi qu'une discussion en ce qui concerne les usages possible de ce genre de construction. L'usage du géotextile composé pour la protection contre l'érosion est aussi présenté.

INTRODUCTION

During the summer of 1980, three experimental retaining walls varying from 2 to 6 meters (6 to 18 feet) high were constructed in Woodbridge, Virginia, U.S.A., by the author with the support of the Reinforced Earth Company. These walls, which averaged 30 meters (100 feet) in length, were unique in that the vertical wall facing was a composite fabric supported by a Reinforced Earth mass. The following describes some of the more significant aspects of design and construction of these walls.

PURPOSE

The concept for fabric faced walls evolved from an earlier concept to stabilize steep slopes (1:1 or steeper) using an open weave fabric and Reinforced Earth strips. It was hoped that this might result in a relatively inexpensive method for improving land utilization in densely populated areas. Since the design modifications to develop a vertical wall were minor, this was the next logical step.

The various components of the fabric facing are significantly less expensive than concrete. However, because they have a shorter design life (estimated to be 10 to 15 years), these walls are considered to be temporary structures. Thus the ultimate market for such walls would be in remote areas, where the need for strong, flexible, inexpensive structures is greater than the need for polished appearance and long design life.

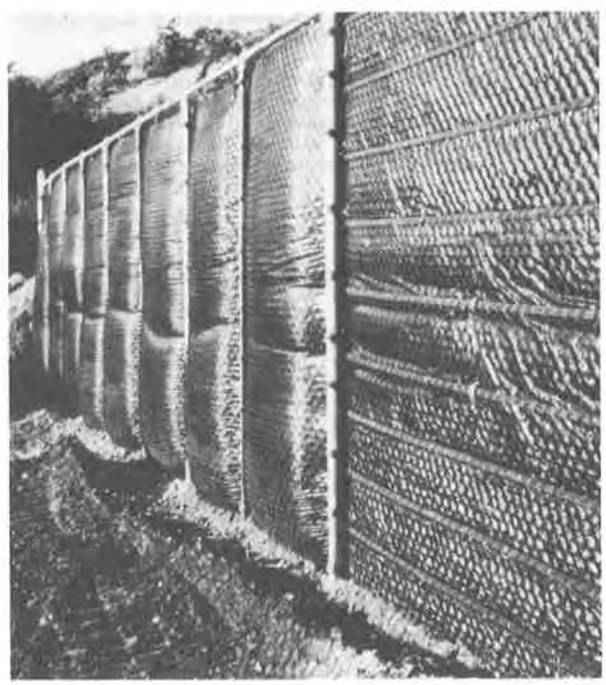

PHOTO 1 FABRIC FACING, TEST WALL NO. 3

PHOTO 2 TEST WALL NO. 3, BEFORE BACKFILLING

PHOTO 3 TEST WALL NO. 3, BACKFILLED

FACING MATERIALS

The wall facing consisted of woven polypropylene fabric (Poly-Filter X) bonded to polyvinyl chloride coated and galvanized, steel wire mesh. The steel mesh used in all three walls was a conventional 9 gauge chain link fencing fabric (Photo 1).

This composite material has several significant advantages; it is strong, light weight, flexible, free draining and relatively inexpensive. It is also possible, by special arrangement with materials suppliers, to use various colors to better blend into the surrounding environment.

During construction a variety of connectors and stiffness were tested to determine the most satisfactory method of attaching the facing fabric to the Reinforced Earth strips embedded in the backfill. Attempts to build pre-fabricated facing panels, which could be lifted and bolted into place, proved unsuccessful. In the end, the best facing erection technique was to hang and stretch the facing fabric on conventional, steel fence posts spaced on 2-meter (6-foot) centers. The Reinforced Earth strips were then attached to the posts. Figure 1 shows a typical cross section through Test Wall No. 3.

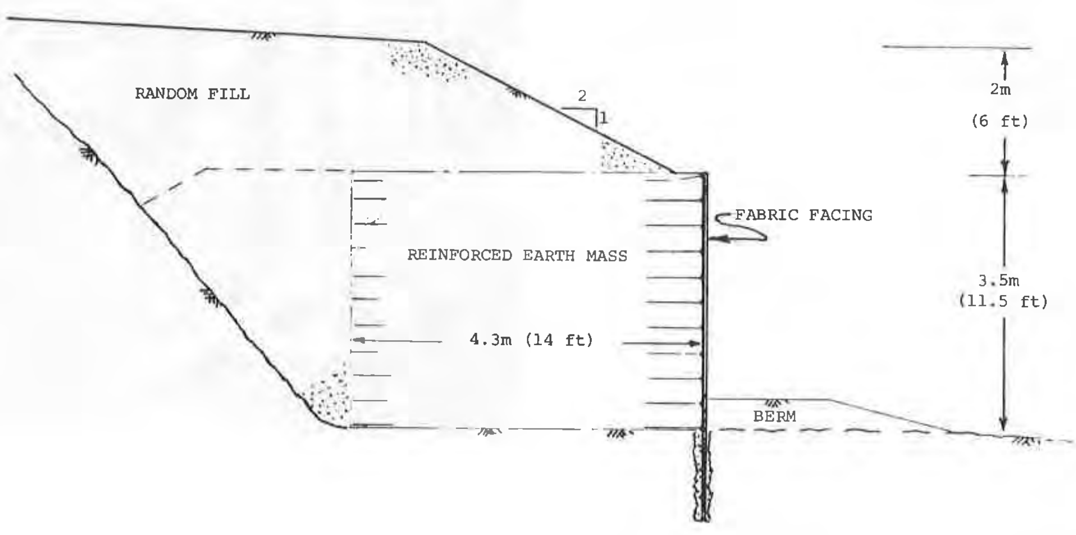

FIGURE 1 TYPICAL CROSS SECTION, TEST WALL NO.3

REINFORCED EARTH MASS

Reinforced Earth has been used world wide to construct hundreds of retaining walls. Generally these walls are faced with either precast concrete panels or galvanized steel panels.

As in other Reinforced Earth structures, the three experimental walls used galvanized reinforcing strips built into the granular backfill to provide tensile resistence to the earth mass, much as reinforcing bars do in reinforced concrete (Photo 4). Thus the fabric facing served largely to contain the granular backfill and provide a finished exterior surface.

PHOTO 4 REINFORCED EARTH STRIPS, TEST WALL NO. 1

CONSTRUCTION PROCEDURE

After several attempts, the most satisfactory sequence of construction was as follows: rough grade site, set fence posts, attach and stretch facing fabric onto posts (Photo 6), attach connectors, bolt on lower level of Reinforced Earth strips (Photo 7), place and compact initial lifts of backfill. Additional strips are attached as the backfilling progresses (Photo 8). This is repeated until the backfill is brought up to the top of wall (Photos 9 and 10). Following this procedure, a 4-meter-high wall, approximately 35 meters long, could be completed in 2 to 3 working days using a relatively small labor crew (4 to 5 men and light weight equipment. Because of design and construction constraints, it is estimated that 8 meters (25 feet) is a realistic limit on the height for this type of wall.

Following construction, wall performance was observed by applying traffic loads from trucks, construction equipment and a dead load from stacked concrete panels (Photo 5). Though the walls were not loaded to failure, it was determined that they performed as designed; no significant distress under heavy load.

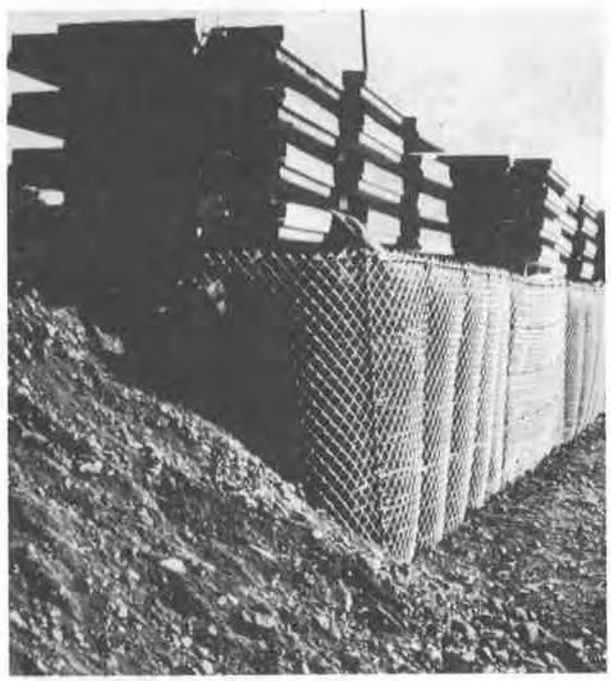

PHOTO 5 LOAD TESTING TEST WALL NO. 1

POTENTIAL USES

Because the structural elements (facing, connectors and reinforcing strips) are relatively light weight, this type of wall construction is well suited for remote locations. The construction of logging roads in mountainous terrain is one potential use.

The materials are also well suited for a variety of military applications, such as roads, helicopter landing pads, railroad embankments and airfields. With a little advanced planning, entire retaining wall systems (except the backfill) could be packaged and air dropped into the construction site.

The facing material also makes an excellent slope cover or channel lining to reduce erosion of sandy or fine grained soils. For this use the facing fabric is "nailed" to the underlying ground using "nails" fabricated from No. 4 reinforcing bars. The "nail" spacing and length varies considerably depending on the soil conditions and the severity of the flow through the area to be protected.

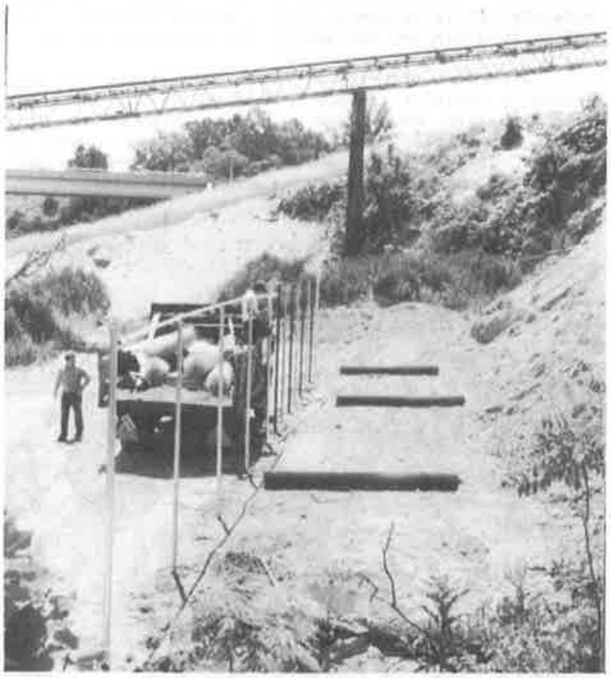

PHOTO 6 HANGING FABRIC FACING, TEST WALL NO. 3

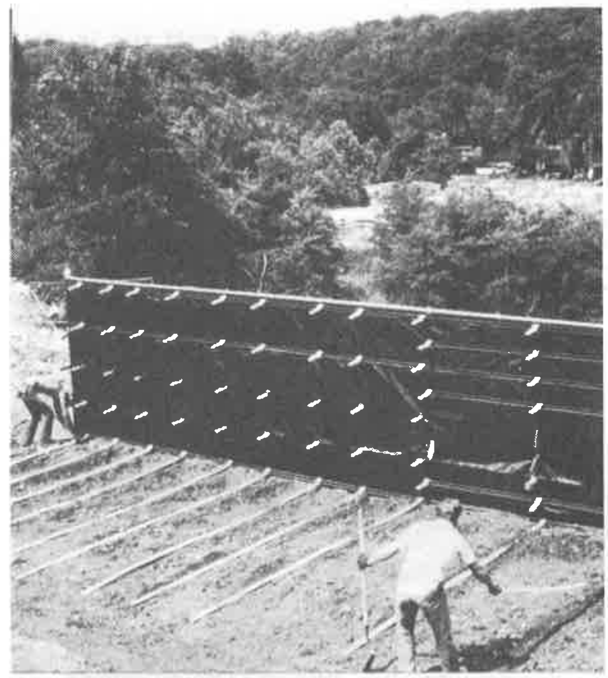

PHOTO 8 ATTACHING REINFORCING STRIPS, TEST WALL NO. 3

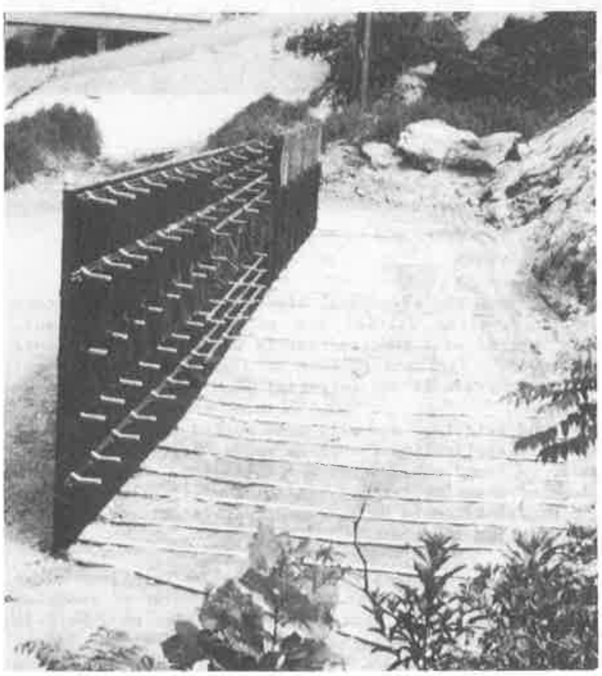

PHOTO 7 READY TO START BACKFILLING, TEST WALL NO. 3

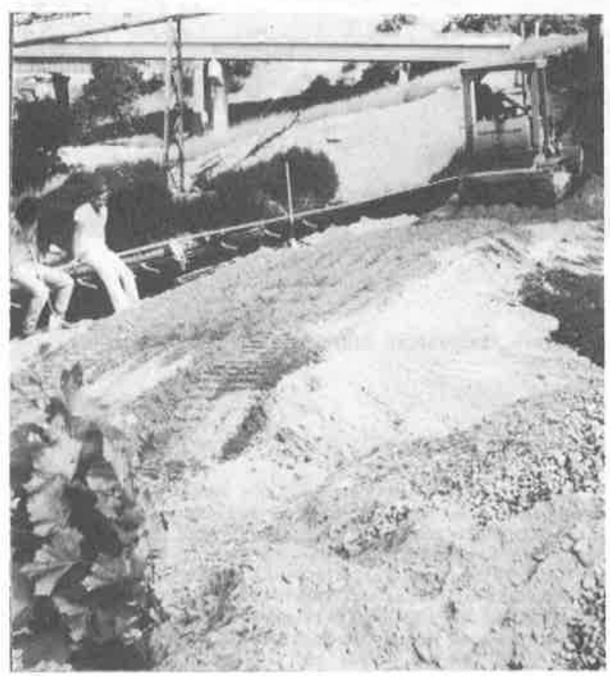

PHOTO 9 SPREADING BACKFILL, TEST WALL NO. 3

PHOTO 10 COMPACTING BACKFILL, TEST WALL NO. 2

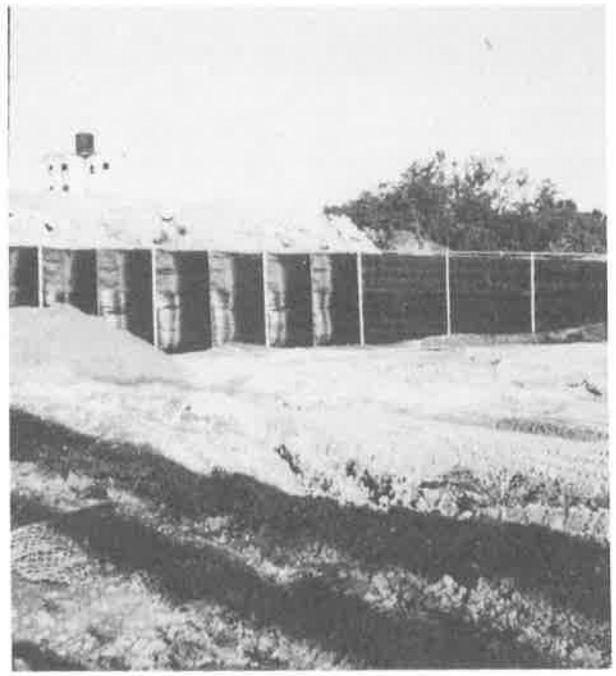

PHOTO 12 TEST WALL NO. 3

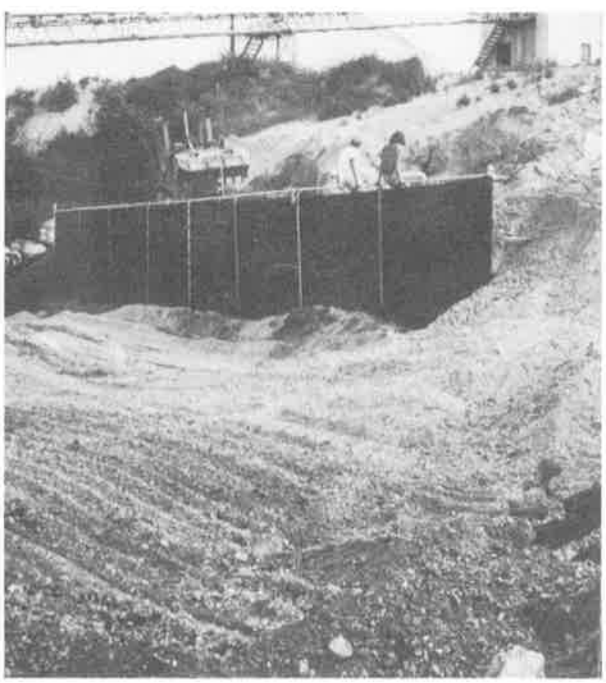

PHOTO 11 BACKFILLING TEST WALL NO. 3

CONCLUSIONS

The three test walls demonstrate that fabric-faced Reinforced Earth walls are strong, flexible, relatively inexpensive and portable. Though test construction has shown the concept to be a practical engineering application, marketing of the system has been deferred. However, some ongoing study is in progress to mitigate long term problems related to ultraviolet deterioration, to determine the effects of fire and lightning, and improve the visual appearance by using colored components to blend with the environment. Improved procedures for bonding the cloth to the wire mesh are also being evaluated.

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Internal/External Fabric Reinforcement of Sand

Renforcement interne et externe de sable avec les textiles

The stress-deformation behavior of internally/externally reinforced sand masses was investigated experimentally. Internal reinforcement was provided by insertion of fabric layers within the sand; external, by simultaneous encapsulation in a woven fabric or geotextile.

The short term, stress-deformation behavior of sand reinforced in this manner was controlled by the respective modulis and other mechanical properties of the geotextiles and their method of placement. Internally/externally reinforced granular masses, e.g., reinforced "earth pillars" and "trench foundations," merit further investigation as load bearing structures in soft, cohesive soil.

On a expérimenté l'action de contrainte-déformation du sable renforcé à l'intérieur et à l'extérieur. Le renforcement ā l'intérieur s'est fait en insérant des couches de tissés dans le sable même. En même temps, ā l'extérieur, on a entouré le sable d'un tissé ou d'un géotextile.
A court terme, on a contrôlé l'action de contrainte-déformation du sable ainsi renforcé a l'aide des modules respectifs, des autres caractéristiques mécaniques des géotextiles, et de la méthode avec laquelle ils ont eté installés.
Ces masses granulaires renforcées ā l'interieur et ā l'extérieur, comme les "piliers de terre" renforcés et les "fondations de tranchée", méritent d'être étudiées davantage car en tant que structures, elles peuvent supporter du poids dans des sols mous et argileux.

INTRODUCTION

This paper describes the results of preliminary research on the stress-deformation behavior of sands which have been internally/externally reinforced with synthetic fabrics or geotextiles. External reinforcement was provided by encapsulating a sand in a woven fabric; internal by inserting layers of either woven or non-woven fabric within the sand. Both types of reinforcement are used conjunctively - a key concept. The ultimate purpose of this research is to determine the feasibility of using internally/externally reinforced granular masses as load bearing structures.

Most fabric reinforcement systems todate consist of interlayered fabric in a granular fill. Fabrics have also been used occassionally for encapsulation and containment (5). In the latter case the fabric acts as both a separation and quasi reinforcement medium. Examples include membrane encapsulated soil for road bases (2, 11) and containment of light weight fill dikes constructed on weak, highly compressible ground (9). The question arises, why not combine both internal reinforcement (intercalated fabric layers) with external reinforcement (fabric encapsulation) in a purposeful and synergystic fashion?

Potential applications of internally/externally reinforced granular masses included above grade structures - embankments and walls; and below grade foundations - "earth pillars" and "trench foundations" (Fig. 1). All these applications entail the use of both encapsulating and intercallated fabrics or geotextiles. Granular fill is shown in these examples; other structural fill materials could eventually be investigated such as lightweight fly ash and saw dust.

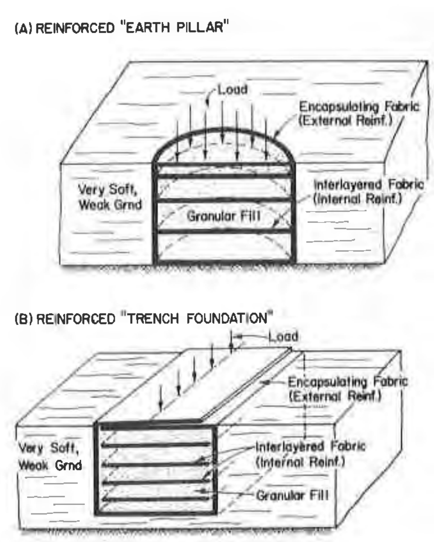

Fig. 1 Internally/externally, fabric reinforced below grade foundations.

The reinforced "earth pillar" concept illustrated in Fig. 1 is an alternative to the stone column - vibro replacement method (3) which has been used to strengthen cohesive soils. In the latter method a combined system of compacted, granular columns in a matrix of native, cohesive soil supports a vertical load which is transmitted through a rigid spread footing. After distribution through a granular blanket, the load is transferred and concentrated initially on the compacted granular cylinders or "stone columns." The cylinders tend to dilate under this increased load and exert a lateral stress on the native, surrounding soil; but this lateral stress (and dilation) are resisted by passive earth pressure. This interaction is repeated until a state of equilibrium is reached. The rigidity and load carrying capacity of the columns depends largely upon the amount of lateral restraint or confining stress that can be mobilized in the surrounding clay.

In the reinforced "earth pillar" concept (Fig. 1) lateral restraint comes not only from the surrounding soil but also from the encapsulating and intercallated fabric. The increase in confining stress induced by the fabric may equal or exceed the restraint provided by earth pressure from the surrounding soft clay.

The reinforced "trench foundation" works on the same principal but the geometry of the system and method of installation are different. This system avoids the need for a vertical seam or joint on the encapsulating fabric. A "trench foundation" is in essence a trench drain, i.e., a trench which is dug in the ground, lined with a perveous fabric, and backfilled with washed stone. A trench drain is used to intercept groundwater; with some modifications it could also be used to support loads.

A key question in using internally, externally reinforced granular masses as load bearing structures is the amount of lateral confining stress that can be induced by the fabrics. Both the maximum amount and the rate at which the confining stress are mobilized will govern the strength and stress deformation behavior of the reinforced, granular material. Another critical consideration is the potential influence of creep or stress relaxation in the fabrics particularly at high working stresses.

1. MECHANICS OF REINFORCEMENT

1.1 Extensible vs. Non-Extensible Reinforcements

Important differences can be demonstrated (6, 7) in the stress-deformation response of sands reinforced with relatively low modulus synthetic fabrics or natural fibers (PLYSOILS) as opposed to high modulus metallic reinforcement (REINFORCED EARTH™). Tensile modulus and permissable elongations in the reinforcement are important considerations. High modulus steel reinforcement with good frictional contact will greatly inhibit internal soil tensile strains, limit boundary deformations, and increase strength provided the reinforcement does not rupture.

In contrast, low modulus fabrics will not likely limit boundary deformations nor increase strength to the same extent. They have the advantage, however, of being able to undergo high elongation and to continue mobilizing tensile resistance at relatively high strains. Because of their lower modulus, higher elongation at break, and generally high frictional properties (12) they are far less likely to either pull out or break; instead they simply yield or stretch with only limited, local slippage occurring where tangential stress exceeds skin friction at the inclusion-sand interface.

In the case of internally/externally reinforced sands the modulus ratio of the two types of reinforcement is also important. The overall stress-deformation behavior of the reinforced composite will depend upon the rupture strength, tensile modulus, and elongation properties of both internal and external reinforcements. The issue is to identify optimal combinations of internal/external reinforcement properties which lead to desirable stress-deformation behavior, e.g., high peak strength, minimal loss of strength at large strain, and avoidance of overstress in either reinforcing system.

1.2 Equivalent Confining Stress Concept

Different concepts have been advanced to explain the mechanics of earth reinforcement (8). Laboratory studies have been conducted in order to define the basic mechanism. Schlosser and Long (10), Yang (13), and McGown et al. (7) each reported the results of triaxial compression tests on cylindrical samples of dry sand containing thin, horizontal layers of tensile reinforcing material.

The results of these triaxial tests on reinforced sand have been interpreted in two different yet related ways. Yang (13) hypothesized that tensile restraint in the reinforcement induced an equivalent confining stress increase $\Delta\sigma_3$. Thus, from the Mohr-Coulomb formulation for the strength of a cohesionless material, it follows that

$$\sigma_{1_f} = (\sigma_3 + \Delta\sigma_3)N_\phi \tag{1}$$

where: σ_{1_f} = major principal stress of failure; σ_3 = applied confining pressure on the test specimen; $N_\phi = \tan^2(45 + \phi/2)$; ϕ = friction angle of unreinforced sand.

Schlosser and Long (10) proposed that the reinforcements induced an anisotropic or pseudo cohesion C_R which was a function of their spacing and tensile strength. The strength of the reinforced composite accordingly is given by

$$\sigma_{1_f} = \sigma_3 N_\phi + 2\,C_R\sqrt{N_\phi} \tag{2}$$

The anisotropic or pseudo cohension (C_R) was computed from a force-equilibrium analysis of a reinforced composite. The following expressions can be derived:

$$\text{Horizontal Reinforcement:}\quad C_R = \frac{\alpha_f}{\Delta H}\,\frac{\sqrt{N_\phi}}{2} \tag{3}$$

$$\text{Inclined Reinforcement:}\quad C_R = \frac{\alpha_f[N_\phi\cos^2\beta - \sin^2\beta]}{\Delta H\;\; 2\sqrt{N_\phi}} \tag{4}$$

where: α_f = force per unit width of reinforcement at failure, kN/m; ΔH = spacing between reinforcements, m; β = angle of inclination of reinforcement counterclockwise from the major principal plane, degrees.

Comparison of Equations (1) and (2) indicates a correspondence between $\Delta\sigma_3$ and C_R, viz.,

$$C_R = \frac{\Delta\sigma_3}{2}\sqrt{N_\phi} \tag{5}$$

Comparison of Equations (3) and (5) in turn shows the following:

$$\Delta\sigma_3 = \frac{\alpha_f}{\Delta H} \tag{6}$$

Thus, the unit tensile resistance in the reinforcement ($\alpha_f/\Delta H$) s directly equal to an equivalent confining stress increase ($\Delta\sigma_3$).

The relationships expressed in Equations 3 and 4 assume that failure occurs by breaking of the reinforcement rather than pullout or stretching. The likelihood of breaking a low modulus fabric reinforcement which behaves as an extensible inclusion is slim. Elongations to break in non-woven geotextiles are quite high - often exceeding 50%. In contrast vertical strains at peak stress in a triaxially loaded, dense sand are an order-of-magnitude less. Horizontal strains are even lower; the exact value depending upon the ratio of horizontal to vertical strain during a test. It is the horizontal (lateral) strain which governs the amount of mobilized tensile resistance in horizontal, layered inclusions. This tensile resistance can be computed to a first approximation as follows:

$$\alpha_{\varepsilon_H} = J_{sec_{\varepsilon_H}} \cdot \varepsilon_H = J_{sec\,\varepsilon_H} \cdot \nu\varepsilon_V \tag{7}$$

where: α_{ε_H} = force per unit width of fabric reinforcement corresponding to lateral strain in the sand, kN/m; ε_H = horizontal (lateral) strain in the sand; ε_V = vertical strain in the sand at peak stress; $J_{sec_{\varepsilon_H}}$ = secant modulus of fabric reinforcement between elongation O and ε_H, kN/m; ν = Poisson's ratio for sand.

This relation assumes that there is no slip at the sand-fabric interface and that Poissons ratio is a good measure of the horizontal to vertical strain in the sand at failure adjacent the reinforcement.

2. TEST RESULTS

2.1 Reinforcements

Commercially available geotextiles with a range of mechanical and rheological properties were selected for testing. Both woven and non-woven fabrics were included. Fine brass screen or strainer cloth (80 mesh size) was included as well in order to examine the influence of a relatively high modulus reinforcement.

The results presented herein are based on two woven fabrics (Geolon 200 and 400) and two non-woven fabrics (Typar 3401 and 3601) which are believed to be fairly representative of reinforcement geotextiles used in practice. Properties of these fabrics are summarized in Table 1. This information was compiled from data supplied by the manufacturer. The non-woven geotextile is a spun bonded, polypropylene sheet manufactured by E. I. Dupont Company; the woven fabric is made from polypropylene fibers and is manufactured by the Nicolon Corporation.

Table 1 - Geotextile Properties

Trade Name	Thickness (mm)	Grab Tensile (kN/m)	Millen Burst (kN/m^2)	Elongation to Break (%)	Secant Modulus @5% Elong (kN/m)
Typar 3401	0.38	23	1172	62	80
Typar 3601	0.48	39	1813	63	210
Geolon 200	0.46	35x35	2482	20x12	
Geolon 400	0.74	66x44	2896	30x18	180

2.2 Triaxial Tests

The triaxial tests were run on reinforced samples of dry, Muskegon dune sand in a dense condition. Properties of the sand are summarized in Table 2.

Table 2 - Sand Properties

Name	D_{50} (mm)	C_u	(degrees)	e_{max}	e_{min}
Muskegon sand	0.23	1.5	39.5	0.73	0.50

Reinforcements were placed in evenly spaced, horizontal layers. Reinforced sand specimens failed by bulging or lateral spreading between reinforcements (Fig. 2).

Fig. 2 Deformed shaped of internally reinforced sand tested in triaxial compression.

Typical results of triaxial tests are plotted in Fig. 3 for sand reinforced with increasing numbers of layers (N=1, 3, & 5) of the synthetic, non-woven fabric Typar 3601. Results of reinforcement tests with the brass strainer cloth (N=3) are shown for comparison as well. Strength increased with increasing concentration of reinforcement. The break in the curves corresponds to a critical confining stress; below this critical stress the reinforcements tended to slip or pullout as opposed to stretching. This behavior corroborates earlier triaxial test results reported by Yang (13) and Schlosser and Long (10). The critical confining stress was typically about 98 kPa (14 psi) for the synthetic fabrics tested and 214 kPa (31 psi) for the brass strainer cloth. Above this confining stress, failure envelopes all tended to parallel the envelope for the unreinforced sand. These results indicate that the friction angle of the sand was unaffected by the reinforcement.

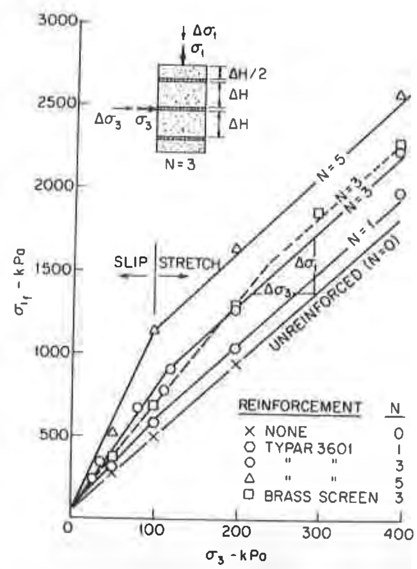

Fig. 3 Results of triaxial tests on Muskegon dune sand reinforced with different numbers of layers of fabric reinforcement.

Although reinforcement with synthetic fabrics increased ultimate strength, they tended to reduce overall stiffness of the sand as shown in Fig. 4. This tendency was more pronounced as the number of reinforcement layers increased. The reinforcements also tended to increase the amount of strain to reach peak stress from 4% for sand alone to 11% for sand reinforced with 5 layers of Typar 3401. Geotextile reinforcement also limited significantly the loss of post peak strength which is typically observed in dense sands tested in triaxial compression.

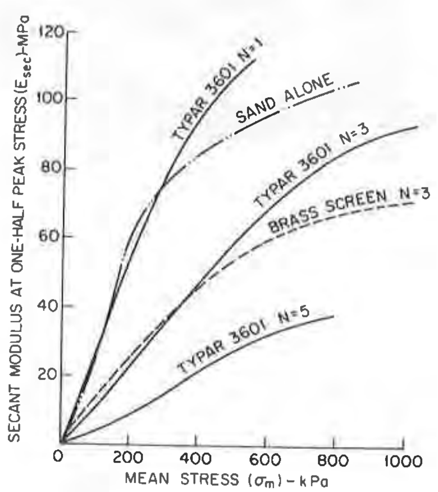

Fig. 4 Secant modulus vs. mean confining stress in triaxial compression for reinforced Muskegon dune sand.

The equivalency between confining stress increase ($\Delta\sigma_3$) and mobilized unit tensile strength in the reinforcement ($\alpha\varepsilon_H/\Delta H$) predicted by Equations 3 to 7 is compared in Fig. 5. The two variables are shown plotted vs. one another for a number of different test runs. The reinforcement induced confining stress was obtained directly from experimental data. The mobilized unit tensile strength in the reinforcements was computed according to Equation 7 using the stress-deformation relationship for the fabric (Table 1), spacing between reinforcements, and an assumed ratio of $\varepsilon_H/\varepsilon_V$ = 0.25. The latter is simply a typical value of Poissons ratio for a dense sand. A rough correspondence exists; a slightly lower strain ratio would improve results particularly for the Typar 3601 reinforcement which has a higher modulus, and which tends to restrict lateral strain more. An interesting consequence of this correspondence is that at failure or peak stress in the sand only a small fraction (<10%) of the tensile strength of the fabric is apparently mobilized. This same result was observed earlier by Gray and Ohashi (4) during direct shear tests on dense sand reinforced by low modulus, natural fibers.

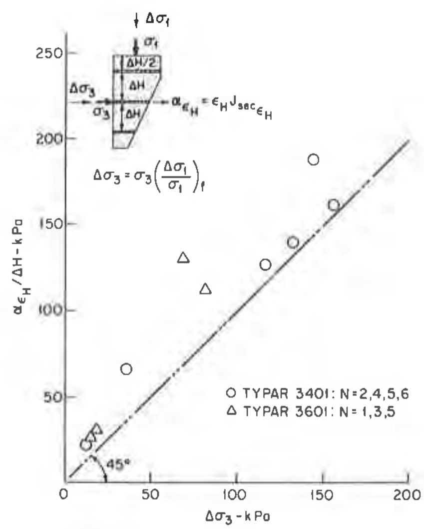

Fig. 5 Comparison between induced confining stress increase and mobilized unit tensile strength in the reinforcements.

Anisotropic reinforcement cohesion (C_R) was computed from experimental data using Equation 5. This cohesion is shown plotted vs. a reinforcement thickness/spacing parameter ($\Sigma T_g/\Delta H$) in Fig. 6. The number of layers of reinforcement are indicated adjacent each data point. This method of plotting makes it possible to examine and compare the influence of type of reinforcement, number of layers, reinforcement thickness and spacing. It also permits comparison with results of internal reinforcement cohesion computed from uniaxial compression tests on internally/externally reinforced sands and with theoretical predictions from finite element model studies.

2.3 Uniaxial Compression Tests

Quasi uniaxial compression tests were run on 127-mm high x 61-mm diameter specimens of dry Muskegon dune sand

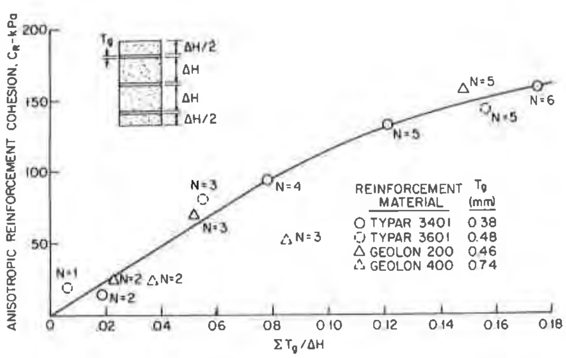

Fig. 6 Anisotropic reinforcement cohesion vs. thickness/spacing parameter for Muskegon dune sand reinforced with woven and non-woven geotextiles.

encapsulated (externally reinforced) in a woven fabric (Geolon 400). Another series of tests were run on samples that were both encapsulated and internally reinforced with horizontal layers of different reinforcing fabrics (Typar 3401, 3601, and brass strainer cloth). These tests simulate loading of a reinforced "earth pillar" (Fig. 1). A typical deformation pattern in an internally/externally reinforced sand column near failure is shown in Fig. 7. Failure usually occurred as a result of localized bursting of the encapsulating fabric. When bursting occurred adjacent the seam this resulted in lower compressive strengths.

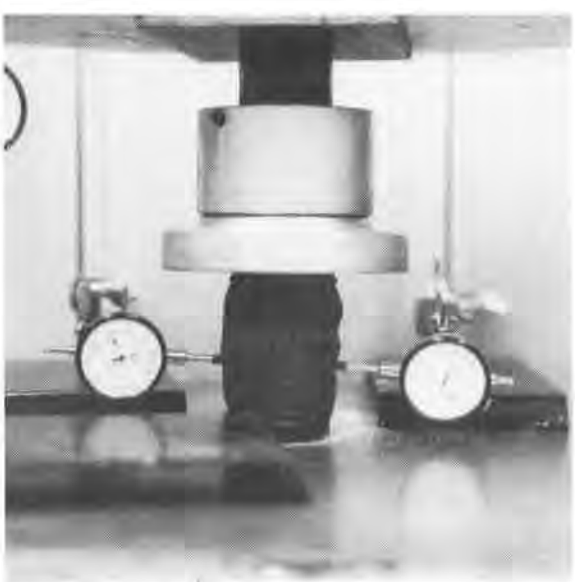

Fig. 7 Deformed shape of internally/externally reinforced sand tested in "uniaxial" compression.

The confining stress increase induced by encapsulation alone ($\Delta\sigma_3$) was calculated from Equations 3 with the external or applied confining stress (σ_3) set equal to zero. The major principal stress at failure (σ_{1_f}) was equal to the uniaxial compressive strength (q) in this case. The induced confining stress (834 kN/m^2) was only equal to one fourth of the burst strength of the fabric (Table 2). The need to glue a vertical seam in the fabric may have contributed to premature failure and incomplete mobilization of fabric burst strength. Conversion to plain strain testing conditions which simulate loading of a reinforced "trench foundation" (Fig. 1) may eliminate this problem. In this case the seam (overlap) will be on top of the reinforced mass directly beneath a distributed load.

Apparent reinforcement cohesion (C_R) from encapsulation was calculated using Equation 5. Additional reinforcement cohesion from internal reinforcement was calculated using Equation 2. These results are summarized in Table 3.

Table 3 - Results of Uniaxial Compression Tests on Internally/Externally Reinforced Sands.

Reinforcement	N	q_1	q_2	$(\varepsilon_v)_f$	Extl. C_R	Intl. C_R
Encapsulation Only (Geolon 400)	0	3827	-	0.26	896	-
+ Typar 3401	2	-	3978	0.25		35
" 3401	4	-	4482	0.27		153
" 3401	5	-	4744	0.22		214
+ Typar 3601	3	-	5392	0.25		365
" 3601	4	-	4454	0.22		146
" 3601	5	-	5033	0.26		282

N = no. of reinforcements; q = compressive strength (kPa); C_R = reinforcement cohesion (kPa); $(\varepsilon_v)_f$ = axial (vertical) strain at failure.

Internal reinforcement cohesion varied with the number of reinforcing layers and ranged from 35 to 365 kPa whereas the external reinforcement cohesion was 896 kPa. The internal reinforcement cohesion was the same order-of-magnitude as reinforcement cohesions claculated from results of triaxial tests (see Fig. 6).

The internal reinforcements did not greatly increase the ultimate compressive strength of the cylindrical sand specimens (see Table 3). Their main influence seemed to be on the stress-deformation response (Fig. 8). Internal reinforcements increased the stiffness of the sand by constraining lateral deformation and limiting overstress in the encapsulating fabric. This influence was particularly pronounced at high strains. Increasing the number of internal reinforcements increased stiffness. A much more linear stress-strain relationship was observed in these internally/externally reinforced composites almost up to failure — in spite of the fact that failure occurred at quite large strains (ε_v > 20%). This behavior is opposite that observed with internal reinforcements alone (Fig. 4). In the latter case an increase in layers of reinforcement (N > 1) resulted in a loss of rigidity. The former response suggests, therefore, a synergistic type of interaction between internal and external reinforcements in a sand.

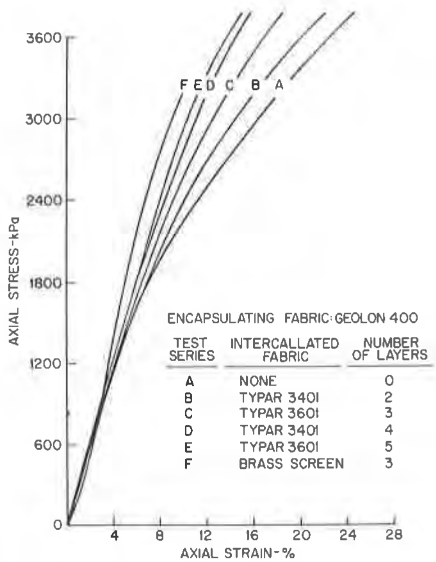

Fig. 8 Stress-deformation behavior of internally/externally, fabric reinforced Musekgon dune sand.

3. CONCLUSIONS

Preliminary test results indicate that conjunctive reinforcement of granular columns with both external (encapsulating) and internal (layered) geotextiles can both strengthen and stiffen the column significantly. Stress-deformation response of such a reinforced composite can be controlled to a large extent by selection and placement of fabrics with appropriate moduli and other properties.

Internally/externally reinforced granular masses show promise of being used as load bearing structural units in soft, cohesive soils. Reinforced "earth pillars" and "trench foundations" are possible alternatives in this regard to the vibro replacement - stone column system in such soils.

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Behavior of Geotextiles in Embankment Reinforcement

Le comportement des géotextiles comme renforcement de remblai

A shallow embankment was constructed across an area of deep black marsh muck. A spunbonded polypropylene geotextile was placed between the muck and granular fill and evaluated for embankment reinforcement.

After construction and traffic use, test pits were excavated over the fabric. Plate bearing tests were run on the fabric and, by cutting the fabric, on the muck beneath.

It was shown that there is a membrane effect from the geotextile. The effective elastic bearing capacity of the soil was increased from π c to (π+2)c.

On a évalué un tissu spunbonded en polypropylène, utilisé pour renforcer un remblai peu profond; ce dernier a été construit sur un terrain se composant de boue noire de marais. On a posé le géotextile entre la boue et le remblai granulaire.

Après une période d'usage normale, on a creusé des trous de sondage au-dessus de tissu; ensuite, on a fait des chargements d'épreuve sur le tissu et, en coupant le tissu, sur la boue en-dessous.

Ces essais montrent que le géotextile exerce une action de membrane. La limite effective de charge élastique du sol a été augmentée de π c à (π+2) c.

INTRODUCTION

Papers presented at the First International Conference on the use of fabrics in geotechnics reported that fabrics influenced the behavior of granular fill only under large deformations [Jarrett, et al. (1)]. It was argued that such large deformations cannot occur in a structure in use. Otherwise, a roadway or embankment would fail. Subsequent work has shown that large deformations do take place during the construction of a roadway or embankment on soft soil [C. B. H. Cragg (2)].

My occupation as a geotextile engineer for a large geotextile producer has taken me to every major continent of the world during the past decade. My position involves field development and supervision of projects concerned with roadways, embankments, and slab foundations for small buildings. These projects are almost universally on soils having a shear value of 55 kPa (8 psi) or less. I have noted that from 30 to 50% less fill is required to attain a given elevation with a geotextile than without one. I have further noted that pavements with base material underlaid by a geotextile exhibit less localized base failure.

TEST SITE AND SOIL PROPERTIES

In the course of a test program, a shallow embankment was constructed across an area of deep black marsh muck. Ground water level was at or slightly below the surface (Figure I).

Figure I
Site and Embankment

Extensive soil explorations were conducted on the site prior to construction using a Farnell Model 245 soil assessment cone penetrometer. The cone has a base area of 129 mm^2 (0.2 in.^{2}) and a base diameter of 13 mm (0.5 in.). This instrument reads directly in California Bearing Ratio Units. The average CBR was found to be 1.1 and by conversion, the vane shear 31.8 kPa (4.6 psi). As it was not possible to move plate bearing capacity equipment onto the site prior to construction of the shallow embankment, plate bearing capacity on the in-situ soil was supplied by the Delaware State Department of Transportation.

The embankment constructed was 83 meters (600 ft) long by 3.6 m wide (12 ft) at top and 0.45 m (18") deep. The spunbonded polypropylene geotextile was placed in the base of the embankment. (See Table 1 for physical properties.) The embankment was constructed on the geotextile using a fine to medium sand containing a trace of silt, having an optimum moisture content of 11% and a maximum dry density of 2024 kg/m^3 (126.5 pcf) by ASTM:D698 - Standard Proctor Test. The sand was placed by back-dumping from a truck and spreading with a D-7 Caterpillar bulldozer. Compaction was to a minimum of 95% Standard Proctor using a steel wheeled vibratory compactor. The embankment was traversed 450 times by a fully loaded sand truck having a rear axle load of 50,000 kg (23 Kip) and a front axle load of 20,000 kg (9 Kip). Plate tests were run after this traffic loading.

GEOTEXTILE	GRAB TENSILE STRENGTH	ELONGATION
TYPAR® SPUNBONDED POLYPROPYLENE, STYLE 3401 136 g/m^2 (4 oz/yd^2)*	600 N (135 LBS)	62%

• TYPAR® IS A REGISTERED TRADEMARK OF E. I. DU PONT
• DATA BY DU PONT

Table 1
Geotextile Physical Properties

TESTING APPARATUS AND PROCEDURE

Plate bearing capacity tests were conducted directly on the geotextile as per ASTM-D1196-57 "Nonrepetitive Static Load Tests of Soils". Six test pits, 90 cm in diameter, were excavated in the embankment over the geotextile. Three of the pits were constructed for "plate on geotextile" tests and three for tests where the geotextile was cut around the circumference of the 30 cm diameter plate. A reactive force was provided by the rear hitch of a D-4 Caterpillar bulldozer. The load was applied to the 30 cm plate by a calibrated hydraulic cylinder. Major adjustments for depth of pit were made with a hydraulic truck jack in series. The deflection beam was a 0.15 m x 5 m aluminum I beam supported by concrete blocks.

Figure 2 shows the plate test apparatus.

Figure 2
Plate Test Apparatus

A number of trials were run in an attempt to measure deflection with dial indicators. The rate of deflection was so great that an accurate reading could not be made. A meter stick was substituted for the dial indicators and found to be satisfactory. Rather than pause at given loads or deflections, readings were taken on the fly. Deflection took place at approximately 5 cm/min.

DISCUSSION

Rodin (3) and Whitman and Hoeg (4) state that the onset of plastic deformation occurs in frictionless soils (θ=0) under loading when

$$Q_e = \pi c$$

Where Q_e = elastic bearing capacity of the soil and

c = undrained shear strength of the frictionless soils.

Complete bearing failure occurs when

$$Q_{ult} = (\pi+2)c$$

Rodin modified this expression to state Q_{ult} = (6.2)c for a circular footing.

Table 2 tabulates results of the plate tests.

TEST	ULTIMATE BEARING CAPACITY Q_{ULT}	INCREASE IN Q_{ULT} PERCENT	MODULUS OF SUBGRADE REACTION K_S	INCREASE IN K_S PERCENT
In Situ Soil	**197** kPa 28.6 PSI	———	**1613** KN/m^3 10.3 $KIPS/FT^3$	———
Soil Under Geotextile	**275** kPa 39.9 PSI	**39%**	**2703** KN/m^3 17.2 $KIPS/FT^3$	**67%**
Plate on Geotextile	**335** kPa 48.6 PSI	**70%**	**4839** KN/m^3 30.8 $KIPS/FT^3$	**200%**

Table 2
Results of Plate Tests

Failure is indicated in Figure 3 by arrows. Failure was characterized in all cases by a sudden punching of the plate through the geotextile or soil.

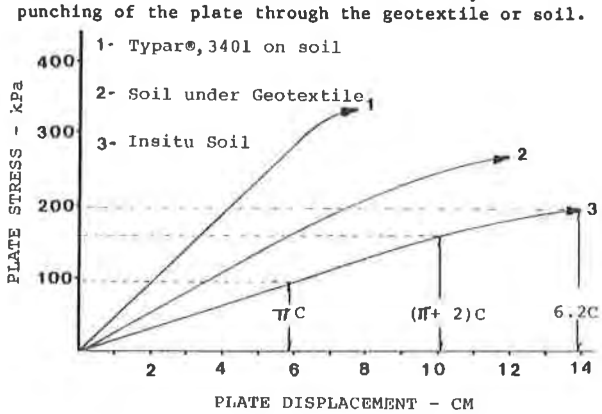

Figure 3
Plate Displacement vs. Stress

The soil under the geotextile increased in effective bearing capacity at failure (Q_{ult}) by 39%. Steward and Mahoney (5) noted this phenomenon in the United States Forest Service Quinault Test Road. A further 31% increase in Q_{ult} was realized with the geotextile. This increase is theorized to be due to the modulus or membrane effect of the geotextile.

As one concerned with field projects, I am interested in the practical application of such information. One section was built without a geotextile. The contractor noted that he used twice as many loads of sand per station to attain the same elevation without a geotextile as he did with a geotextile. No plate tests were run on this section as there was a great deal of admixture of sand with soil. It is theorized that the excess sand went to areas of local shear failure and that the reason for excessive fill consumption was due to the lower modulus of subgrade reaction K_s and an irregular cross section of the embankment without a geotextile.

Anyone who has conducted soil explorations knows that no matter how careful one is there will be areas of low bearing capacity that will be missed. It is not economical to design for such omissions. Potholes in roadways and uneven embankment profiles are evidence that this occurs in everyday life. The stress on the subgrade has exceeded πc. A study of the plate curves will show that even when conventional design procedures for embankments are used, the designer can be more comfortable with his factor of safety when a spunbonded polypropylene geotextile is used at the base of the embankment because stress at the subgrade can safely be between πc and (π+2)c. Practically, a "floating" embankment can be built with confidence. The author has been involved with two embankment projects 8 kilometers long and a building on a slab foundation that successfully utilized these principles.

In the test described in this paper, the spunbonded polypropylene geotextile has modified the response of the soil under load. The elastic response of the soil has been extended from πc up to (π+2)c. The modulus of subgrade reaction (Ks) has been increased by as much as 200%.

CONCLUSIONS

1. There is a definite membrane effect from the use of geotextiles in the base of an embankment.

2. When a spunbonded polypropylene is used at the base of an embankment, the effective elastic bearing capacity of the soil Q_e can be raised from πc to (π+2)c.

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Study of Stability of Filling-Up Slopes Reinforced by Layers of Geotextile
Etude de la stabilité de talus en remblais renforcés par des géotextiles

In order to estimate the contribution of a geotextile used as a reinforcement to the stability of an embankment slope, a computation program in finite difference adaptable to a minicomputer has been elaborated. This program takes into account the deformation necessary to the putting in tension of the geotextile as well as a possible sliding of a geotextile in relation with the soil.

Following up a study of interaction between soil and reinforcement and the transmission of strains between those two elements, the hypothesis and the computation process of stability will be developped (a concrete case will be dealt with in appendix).

This study has led us to a reflection on the notion of safety factor, concerning geotextile reinforced slopes.

Afin d'évaluer la contribution d'un géotextile utilisé comme armature à la stabilité d'un remblai en pente, un programme de calcul en différences finies, adaptable sur minicalculateur, a été élaboré. Ce programme tient compte des déformations nécessaires à la mise en tension du géotextile ainsi que du glissement éventuel du géotextile par rapport au sol.

Après une étude de l'intéraction sol-armature et de la transmission des efforts entre ces deux éléments, les hypothèses et le processus de calcul de stabilité seront développés (un cas d'espèce sera traité en annexe).

Cette étude nous a conduit a une réflexion sur la notion de coefficient de sécurité en ce qui concerne les talus armés par des géotextiles.

INTRODUCTION

Cette étude a pour but d'élaborer un programme de calcul simple (adaptable sur minicalculateur), qui permette de rendre compte de l'effet stabilisant, sur un talus indéfini, d'un géotextile utilisé comme armature.

Le sol et le géotextile seront considérés comme des matériaux distincts ce qui suppose qu'une étude préalable de la transmission des efforts devra être menée. Ensuite, après avoir déterminé les allongements puis les tensions dans les nappes et montré un exemple de calcul nous tenterons de définir ce que peut être la sécurité d'un talus armé vis à vis d'un glissement éventuel.

I - MECANISMES DE TRANSMISSION DES EFFORTS

La transmission des efforts d'un matériau à l'autre se fait par l'intermédiaire du frottement (Fig. 1).

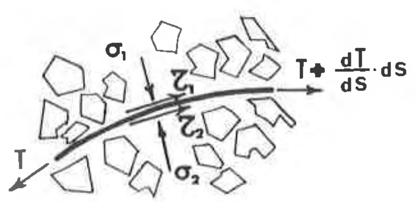

Fig. 1 : Contraintes subies par un élément de nappe dans le sol.

Si l'accroîssement de tension dépasse une valeur tolérable par le sol, alors on verra apparaître un glissement du géotextile par rapport au sol.

$$\left|\frac{dT}{dS}\right| \leqslant \left|\tau_1 + \tau_2\right| \qquad (1)$$

Or τ_1 et τ_2 dépendent du coefficient de frottement sol-armature par la relation :

$$|\tau_i| \leqslant f \cdot \sigma_i \qquad (2)$$

En outre, pour que ces valeurs soient atteintes, il faut que le sol au voisinage de la nappe soit apte à résister à ce cisaillement soit :

$$|\tau_i| \leqslant \sigma_i \operatorname{tg} \Phi + c \qquad (3)$$

où Φ et c sont les caractéristiques du sol, sinon la rupture du sol au voisinage de la nappe aura lieu avant que le frottement limite sol-armature ne soit atteint.

De toute façon, pour que le sol résiste au cisaillement imposé par la mise en tension de la nappe, il est nécessaire qu'il y ait une modification des contraintes au voisinage de la nappe.

Dans ce qui suit, nous supposerons que le sol est en équilibre limite au voisinage de la nappe. Nous supposerons en outre que le géotextile ne transmet d'efforts que dans son plan. On aura alors :

$$\sigma_1 = \sigma_2 = \sigma \qquad (4)$$

$$\tau_1 = -\tau_2 = \tau \qquad (5)$$

Sur l'une des faces inférieure ou supérieure de la nappe, la contrainte de cisaillement dans le sol est opposée à celle nécessitée par le gradient de tension dans la nappe, ce qui conduit à une modification des contraintes. Elles doivent vérifier les relations :

$$|\tau| = f \cdot \sigma \qquad (6)$$

$$\tau \cdot \frac{dT}{dS} \leqslant 0 \qquad (7)$$

Etudions les contraintes au voisinage de la nappe sur la face supérieure du géotextile (Fig. 2 et 3).

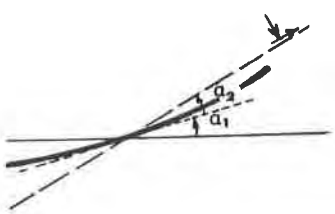

Fig. 2 : Position du géotextile (gros trait continu) et de la facette de la rupture (longs tirets) par rapport à l'horizontale (trait fin continu).

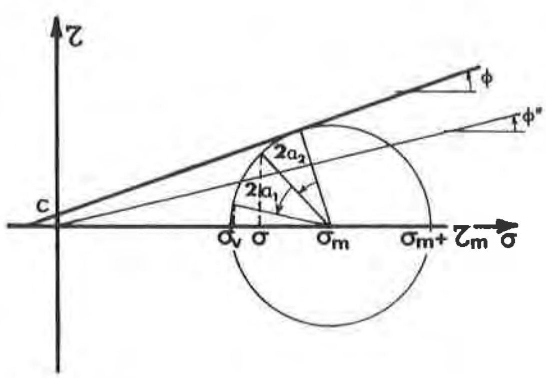

Fig. 3 : Etat de contrainte dans le sol avant la modification de α_2.

$$\tau_m = \sigma_m \sin\Phi + c\cos\Phi \qquad (8)$$

$$\sigma_v = \sigma_m - \tau_m \cos\left(\frac{\pi}{2} - \Phi - 2\alpha_1 - 2\alpha_2\right) \qquad (9)$$

de (8) et (9) on tire :

$$\sigma_m = \frac{\sigma_v + c\cos\Phi\sin(2\alpha_1 + 2\alpha_2 + \Phi)}{1 - \sin\Phi\sin(\Phi + 2\alpha_2 + 2\alpha_1)} \qquad (10)$$

$$\tau_m = \frac{\sigma_v\sin\Phi + c\cos\Phi}{1 - \sin\Phi\sin(\Phi + 2\alpha_2 + 2\alpha_1)} \qquad (11)$$

D'autre part :

$$\tau = \tau_m\cos(\Phi + 2\alpha_2) \qquad (12)$$

$$\sigma = \sigma_m - \tau_m\sin(\Phi + 2\alpha_1) \qquad (13)$$

D'où :

$$\tau = \frac{\sigma_v\sin\Phi + c\cos\Phi}{1 - \sin\Phi\ .\ \sin\beta_1} \cdot \cos\beta \qquad (14)$$

$$\sigma = \frac{\sigma_v(1 - \sin\Phi . \sin\beta) + c\cos\Phi(\sin\beta_1 - \beta)}{1 - \sin\Phi\ .\ \sin\beta_1} \qquad (15)$$

avec : $\beta = \Phi + 2\alpha_2$ et $\beta_1 = \Phi + 2\alpha_1 + 2\alpha_2$

Il suffit de faire varier α_2 de manière à ce que les relations (6) et (7) soient vérifiées. On s'arrangera en particulier pour que l'évolution de τ soit monotone au cours de la variation de α_2 (τ ne passera ni par un minimum, ni par un maximum au cours de la rotation des contraintes principales).

Comme on ne connaît pas le sens de $\frac{dT}{dS}$ a priori, on va calculer $\underline{\tau}$ et $\overline{\tau}$ en faisant varier α_2 dans les deux sens avec :

$$\underline{\tau} = -f \cdot \underline{\sigma} \qquad (16)$$

$$\overline{\tau} = f \cdot \overline{\sigma} \qquad (17)$$

On aura alors la relation :

$$2\,\underline{\tau} \leqslant \frac{dT}{dS} \leqslant 2\,\overline{\tau} \qquad (18)$$

Il faut noter que la traversée de la nappe provoque une modification de σ_v que l'on peut calculer pour les valeurs de α_2 correspondant à $\underline{\tau}$ et à $\overline{\tau}$ (Fig. 4). Sur la face supérieure du géotextile la relation (9) sera vérifiée. Sur la face inférieure on aura :

$$\sigma'_v = \sigma_m - \tau_m \cdot \cos\beta_2 \qquad (19)$$

soit :

$$\sigma'_v = \sigma_v + \frac{\sigma_v\sin\Phi + c\cos\Phi}{1 - \sin\Phi\ .\ \sin\beta_1} \cdot (\cos\beta_1 - \cos\beta_2) \qquad (20)$$

avec : $\beta_2 = \beta_1 - 4\alpha_2$

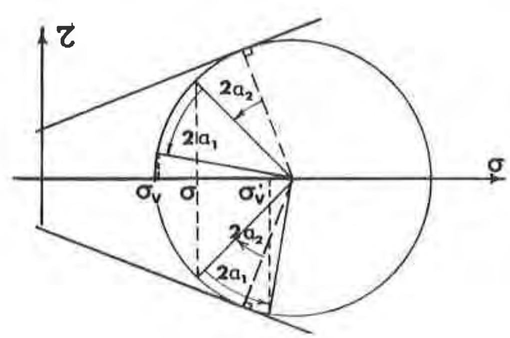

Fig. 4 : Contraintes verticales supérieure et inférieure dans le plan de MOHR.

Entre deux nappes l'accroîssement de σ_v est égal à la contrainte due au poids du sol placé entre ces deux nappes.

II - CALCUL DE STABILITE

Ayant défini les conditions de transmission des efforts, on peut maintenant s'intéresser au problème de stabilité proprement dit.

Compte tenu du module des géotextiles, il semble difficile de négliger la déformation nécessaire à l'établissement d'une tension suffisante à l'équilibre du talus. Afin de conserver un modèle simple, nous supposerons par la suite que l'on peut représenter les déplacements du sol par des rotations autour d'un point unique. En outre nous supposerons que le centre de rotation correspond au centre du cercle le plus défavorable au sens des méthodes globales de calcul de stabilité. Cela suppose que le long des cercles de glissement le sol est en équilibre limite ce qui est contradictoire avec la nécessité de reprendre les efforts dans les nappes. Cependant on peut lever cette difficulté en supposant que la perturbation des lignes de glissement n'affecte que le voisinage immédiat des nappes et qu'il n'entraîne qu'une faible modification du champ global de déplacements.

Il suffit alors d'affecter à chaque rayon une rotation pour définir entièrement la déformée du talus, de la nappe et donc la tension au sein de cette dernière.

Ces rotations doivent toutefois vérifier des conditions :

$$R_1 > R_2 \Longrightarrow 0 \leq \theta(R_1) \leq \theta(R_2) \tag{21}$$

où $\theta(R_i)$ est la rotation associée au rayon R_i .

On montre que, si α_1 est l'angle que fait au point M la tangente à la nappe et l'horizontale et α l'angle du rayon passant par M avec l'horizontale, et que d'autre part M' , α' et α_1' sont les transformés de M , α et α_1 après rotation, l'allongement de la nappe en dehors de tout glissement vaut :

$$\varepsilon^* = \frac{\cos(\alpha + \alpha_1)}{\cos(\alpha' + \alpha_1')} - 1 \tag{22}$$

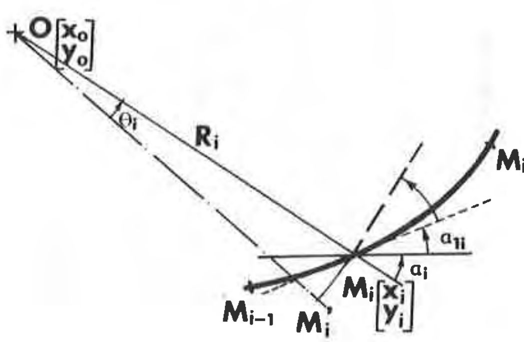

Fig. 5 : Position des divers éléments par rapport au centre de rotation.

Si l'on choisit une résolution par différences finies (Fig. 5) :

$$\operatorname{tg}(\alpha_i) = \frac{y_o - y_i}{x_i - x_o} \tag{23}$$

$$\alpha_i' = \alpha_i + \theta_i \tag{24}$$

$$\operatorname{tg}(\alpha_{1i}) = \frac{y_{i+1} - y_{i-1}}{x_{i+1} - x_{i-1}} \tag{25}$$

$$\operatorname{tg}(\alpha_{1i}') = \frac{y_{i+1}' - y_{i-1}'}{x_{i+1}' - x_{i-1}'} \tag{26}$$

On peut remarquer également que, avant rotation des contraintes :

$$\alpha_{2i} = (\pi - \Phi - \alpha_i - \alpha_{1i}) + k\pi \tag{27}$$

A partir de ces valeurs et de la hauteur de sol au-dessus des nappes, on peut déterminer les valeurs de non glissement $\underline{\tau}_i$ et $\overline{\tau}_i$. Il suffit alors de vérifier la relation (18). Pour cela la connaissance d'une relation liant la tension et l'allongement du géotextile est nécessaire. Nous prendrons par la suite une relation linéaire entre la tension et l'allongement mais toute autre relation pourrait convenir. Dans notre cas :

$$2\,\underline{\tau}_i \leq K\,\frac{\varepsilon_{i+1}^* - \varepsilon_{i-1}^*}{d_{i+1,i-1}} \leq 2\,\overline{\tau}_i \tag{28}$$

Si pour un point de la nappe la relation (28) n'est pas vérifiée cela signifie que la nappe a glissé par rapport au sol. Il faut alors évaluer le glissement en chaque point :

$$\varepsilon_i = \varepsilon_i^* + \varepsilon_{gi} \tag{29}$$

Les ε_i doivent vérifier les relation suivantes :

$$\varepsilon_{i+1} - \varepsilon_{i-1} = 2\,\overline{\tau}_i \cdot d_{i+1,i-1}$$

ou

$$\varepsilon_{i+1} - \varepsilon_{i-1} = 2\,\underline{\tau}_i \cdot d_{i+1,i-1} \tag{30}$$

suivant le cas, pour tout les noeuds où un glissement a lieu.

En fait, pour N noeud où ε_{gi} doit être cherché on dispose de N - 2 équations et de deux inéquations. On doit donc faire appel à deux relations complémentaires pour lever l'incertitude.

Si le glissement n'atteint pas une extrémité de la nappe, la longueur totale de celle-ci n'est pas modifiée par le glissement soit :

$$\sum_N \varepsilon_{gi}\,\frac{d_{i+1,i-1}}{2} = 0 \tag{31}$$

Si le glissement atteint l'une des extrémités :

$$\varepsilon_o = 0 \quad \text{ou} \quad \varepsilon_{N_T} = 0 \tag{32}$$

La deuxième relation s'intéresse au déplacement aux noeuds voisins du point correspondant à la valeur maximale de ε . On suppose que le glissement sera d'autant plus important que le gradient de tension sera plus fort ; soit si i et i+1 sont les noeuds concernés :

$$\frac{\varepsilon_{gi+1}}{\varepsilon_{gi}} = \frac{\varepsilon_{i+2}^* - \varepsilon_i^*}{d_{i,\,i+2}} \cdot \frac{d_{i-1,i+1}}{\varepsilon_{i+1}^* - \varepsilon_i^*} \tag{33}$$

De plus on ne peut pas prévoir a priori le nombre de noeuds qui subissent un glissement, donc il faut étendre, noeud par noeud, le domaine concerné par la redistribution des allongements jusqu'à ce que la relation (28) soit vérifiée en chaque point.

Lorsque la tension a été calculée dans toutes les nappes, on peut alors évaluer le coefficient de sécurité associé à chaque cercle défini par son rayon. L'équilibre est atteint lorsque les conditions suivantes sont réalisées en tout point :

$$\frac{d\theta'}{dR} \neq 0 \Longrightarrow F = 1$$

ou (34)

$$\frac{d\theta'}{dR} = 0 \Longrightarrow F \geq 1$$

Pour trouver l'équilibre on peut partir de la distribution des rotations nécessaire à l'équilibre du talus en l'absence de nappe puis procéder par approximations successives jusqu'à la solution.

Finalement, il faut vérifier que la rupture n'est pas atteinte dans les nappes.

Un exemple de calcul est mené en Annexe.

La rupture du talus a lieu soit par défaut d'ancrage : on assiste a un glissement pour les noeuds extrêmes de la nappe, soit par rupture des armatures.

III - CONCLUSION

En complément de cette étude, nous envisageons de mener une série d'expériences pour vérifier les déformations calculées voire pour modifier certaines hypothèses afin de mieux décrire le phénomène.

Cependant, on peut déjà réfléchir au problème que pose la sécurité d'un talus armé lorsque l'on a déterminé les nappes par ce mode de calcul.

On ne peut pas comparer des talus avec des déformations différentes. Si F^* est le coefficient de sécurité du talus non armé ayant la même déformée que le talus précédent, l'accroissement de la sécurité peut être évalué par :

$$a = \frac{1}{F^*} \qquad (35)$$

Mais cette approche n'est pas suffisante. En effet les déformations nécessaires à la mise en tension des nappes peuvent être incompatibles avec l'utilisation future du remblai. Il faut donc se fixer un coefficient complémentaire qui correspondrait à la sécurité vis-à-vis du déplacement. Par exemple, si h est la déflection maximale calculée et si h_o est la déflection tolérée :

$$b = \frac{h_o}{h} \qquad (36)$$

le coefficient de sécurité du talus armé pourrait alors être :

$$F = a \cdot b \cdot F_o \qquad (37)$$

si F_c est le coefficient de sécurité du talus non armé correspondant.

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ANNEXES

- Caractéristiques du sol :

  γ = 13,5 kN/m^3
  tg ϕ = 0,138
  C = 10 kN/m^2

- Caractéristiques du géotextile :

  allongement à la rupture : 50 %
  tension à la rupture : 2,5 kN/ml
  coefficient de frottement sol-armature : 0,124

- Caractéristiques du talus :

  pente : 30°
  hauteur : 10 m
  sécurité non armé : F_o = 0,878

- Tableau 1 : Rotations et déplacement à l'équilibre

Rayons (m)	Rotations (10^{-2} rad.)	Déplacement au sommet du talus		
		Total (m)	Vertical (m)	Horizontal (m)
15,98	5,42	0,87	0,78	0,38
16,83	5,42	0,91	0,83	0,38
17,69	5,42	0,96	0,88	0,38
17,87	5,33	0,95	0,90	0,37
18,54	3,24	0,60	0,55	0,23
19,39	1,45	0,28	0,26	0,10
20,25	0,1	0,02	0,02	0,01
21,10	0	0	0	0

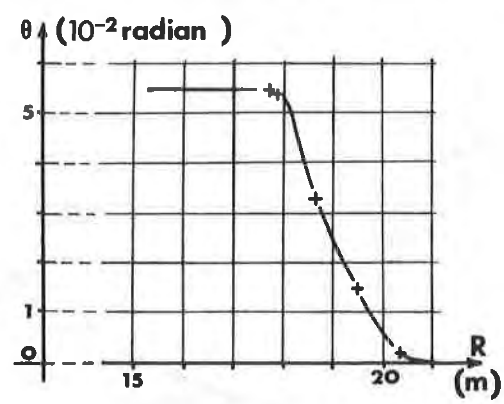

Fig. 6 : Courbe des rotations en fonction des rayons.

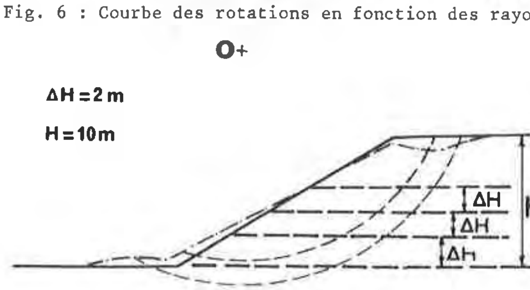

Fig. 7 : Talus et nappes géotextiles avant rotation.
Talus après rotation (—.—.—).

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Use of Non-Woven Geotextiles to Construct a Deep Highway Embankment Over Swamp Soil

Emploie du géotextile non-tissé dans la construction d'un terreplein routier profond sur des sols marécageux

This paper describes the project and construction of a bridge approach-four lane highway embankment located in an area of fast development in Southeast México. Traffic of 20,000 vehicles per day is anticipated. Part of the highway runs through swampy and high compressibility soils. A non-woven heat-bonded geotextile was used for the construction of the highway. To evaluate the influence of the geotextile on the behavior of the road, instrumentation and measurements in four embankment sections were made, two where high compressibility soils existed and the other two in the swampy zone. In each case a non-fabric control embankment was built to compare results. This paper describes the results and conclusions of the swamp test sections. The conclusions are that the use of the geotextile under the embankment was beneficial in reducing the irregular and excessive penetration of the fill material into the swamp and that it also helped to form a working pad.

Le document décrit le projet et la construction d'un terreplein routier a quatre voies de circulation qui sert d'acces a un nouveau pont situé dans une zone a dévelopement rapide au sud-est du Mexique.
Une portion de la route longe une zone marécageuse et des sols de haute compressibilité. Pour la construction du terreplein on a utilisé un geotextile nontissé. A fin d'évaluer l'influence du geotextile dans le comportement de la route on utilisé et on effectue des mesures en quatre points: deux dans la zone de haute compressibilité et les deux autres, dans la zone marécageuse. On a construit le terreplein avec et saus membrane pour comparer les résultats. Daus ce document on mentionne les résultats et les conclusions dérivées des mésures effectueés dans les points de la zone marécageuse. Pour cette experience on a conclu que la mise en place au préalable du geotextile dans la construction du terreplein s'est traduite par la diminution des incrustations irregulieres et excesives du matériel de construction dans les sous-bassements.

INTRODUCTION

Because of the increasing industrial growth of the Shore Plain Oil Basin in the Southeastern area of México, the design and construction of an improved road infrastructure for that region has become necessary. The existing highway does not provide convenient transportation facilities especially between the city of Coatzacoalcos and the industrial areas of "Pajaritos", "La Cangrejera" and "Morelos". This is due to the inefficiency of the bridge that crosses the Coatzacoalcos river. The Mexican Secretariat of Human Settling and Public Works has solved that problem by constructing a new bridge called Coatzacoalcos II, which is located 20 km upstream from the old bridge. This new bridge required the construction of 2 approach highways from both sides of the river with 30 km in length. The highway along the left riverbank crosses a soft soils zone which is located at the Southwestern part of the city of Minatitlán. The new highway will be 22.50 m in width and will allow for 4 traffic lanes. An average of 20,000 vehicles per day is expected when the road is in service, most of them being trucks. A heat-bonded non-woven polypropylene geotextile was used for the construction of the highway embankment. In order to determine the influence of the fabric in the behavior of the road, instrumentation and measurements were carried out in four sites, two of them where high compressibility clayey and silty strata existed and two other in a swamp zone. This paper describes the studies carried out in the swamp zone. Results of such studies and conclusions are also included.

Fig. 1 Shows the initial and the final sections of the projected highway embankment:

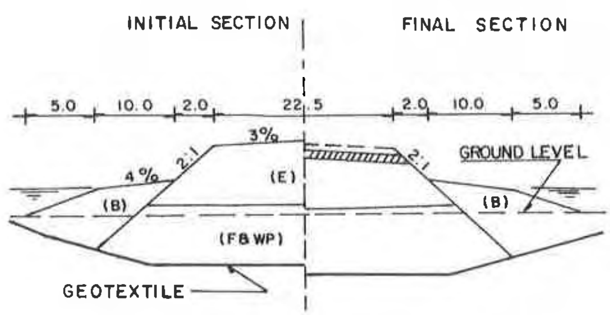

FIG.1 PROJECT SECTION OF THE HIGHWAY EMBANKMENT

DIMENSION IN METERS

E - EMBANKMENT
B - BERMS
F & WP - FILL AND WORKING PAD

1 REGIONAL CHARACTERISTICS

The project is located on the shore plain of the Gulf of México. The most ancient geological formations in this area belong to the Tertiary period (Middle Miocene). They have a sedimentary origin and are formed by stiff clays, shales and loose sandstones that outcrop in the hills of the region (ref. 1). The flood flatland of the Coatzacoalcos river through which part of the highway runs, is formed by recent fluvio lacustrine deposits and by highly organic deposits that rest on the strong Tertiary's formations. The annual rainfall is about 2,500 mm, with rains almost all year long and with an annual average temperature of 26° C. This region is one of México's most active seismic areas, with a maximum acceleration of the ground of 110 cm/s2, for a return period of 50 years (ref. 2). The most recent greatest intensity earthquake took place in August 26, 1959 with an intensity of VII on the Modified Mercalli Scale and it caused severe damages such as the failure of one section of the Minatitlán-Coatzacoalcos highway that goes through a big swamp land near the new road that this paper describes.

2 STRATIGRAPHY AND PROPERTIES

The line of the left riverbank highway, as mentioned before, is located in a swamp zone which has an almost permanent water head of approximately 0.50 m above the ground level. In a length of nearly 12 km we find big peat deposits from the surface, very compressible, with a highly fibrous structure, low shear strength and thickness that goes from 1.5 to 3.5 m. Its water content (ω) reaches values as high as 750%, higher than the liquid limit (ω_L) that reaches values of 700%. The maximum void ratio (e) is 14, and the unit weight (γ) is 9.81 kn/m^3. The shear strength (τ_f) in unconfined compression tests varies between 2.94 and 7.85 kPa. Under the peat layer we find a high plasticity silt stratum, with an average thickness of 1.5 m, ω=70%, ω_L=80% and ω_P=40%. Under this silt there are highly compressible organic clays, with an average ω of 100% and τ_f between 9.81 and 14.72 kPa and lenses of very loose quartz fine sands. Below a depth of 15 m, sandy formations are predominant, whose compactness rapidly increases with depth. Therefore we may consider that the total depth of compressible soils, both organic and inorganic, is of about 13 to 17m. From the consolidation tests it has been observed that the group of deposits is normally consolidated and that the secondary consolidation is significative for peat.

3 DESIGN CONSIDERATIONS

Based on the analysis of stability of embankments built over soft soils and on the magnitude and distribution of the stresses, it was concluded that the low shear strength of the upper formations made it necessary to use berms in the embankment sections to obtain satisfactory behavior of the embankment. The highway has four traffic lanes, one central ridge and shoulders with a crown 22.5 m wide. Surface drainage of 3% beginning from the center line is provided. The projected berms at either side of the road are 10 m wide, giving a total of 56,5 m in width for the working section, as shown in Fig. 1. The structure is as follows:

a) Non-woven, heat-bonded polypropylene geotextile named Typar, Style 3401 manufactured by E.I. Du Pont de Nemours and Co. placed on the subgrade and sewn on-site. This geotextile is used to reduce localized shear failure that would generate increased consumption of fill material that would penetrate differentially into the soft underlying strata and also to prevent contamination of the select material while allowing drainage and to help construct a working pad.

b) Fill and working pad formed with clayey sand placed over the geotextile. Use of this fill material is made since there is no other of better quality available in the neighboring areas.

c) Embankment formed by clayey sand compacted to 95% of the modified Proctor Compaction test. The upper 0.30 m on which the sub-base layer will rest should be compacted to 100% of the above mentioned test.

d) Pavement formed by a 0.15 m thick sub-base, base course improved with Portland cement 0.20 m thick and 0.07 m asphalt concrete running surface.

e) Lateral berms made up of the same material as the embankment.

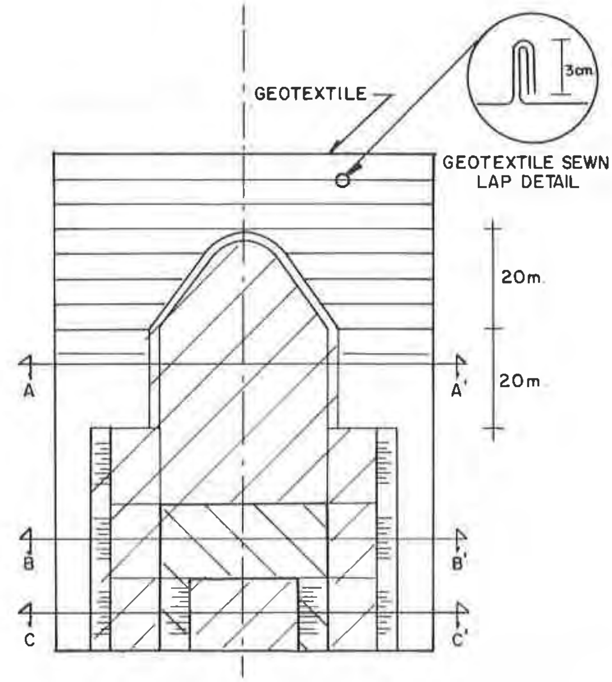

FIG. 2 ARROW ATTACK LOADING AND CONSTRUCTION STAGES (PLAN VIEW)

CROSS SECTION A-A' (SEE FIG. 2)

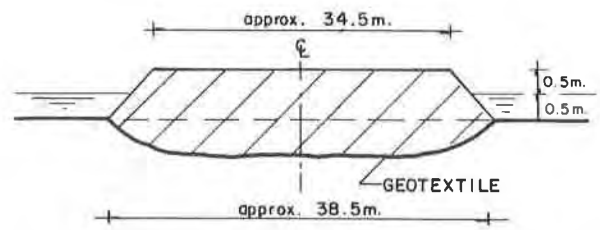

FIG. 3 CONSTRUCTION OF WORKING PAD

CROSS SECTION B-B' (SEE FIG. 2)

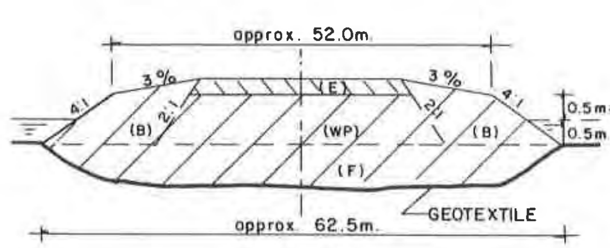

FIG. 4 CONSTRUCTION OF BERMS AND EMBANKMENT

E - EMBANKMENT
WP - WORKING PAD
B - BERMS
F - FILL

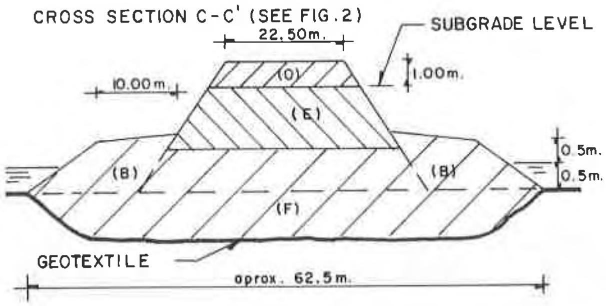

FIG.5 FINAL SECTION OF EARTHWORKS

O- OVERLOAD
E- EMBANKMENT
B- BERMS
F- FILL

4 CONSTRUCTION PROCEDURE

Construction of the embankment was designed according to the following stages:

a) Preparing the ground by removing all the vegetation with a diameter of 0.03 m or greater. During construction it was observed that letting the vegetation remain was practical and it was not removed.

b) Installing the polypropylene geotextile in strips 76 m wide placed transversally to the road. Edges are sewn to cover the project area using thread made of the same material, as shown in the detail of Fig. 2. The length of each strip is 19 m (width of the strips is 22% in excess of working base to provide for settlement). Fabric is spread out directly over the swamp zone, wheather there is water head on the land or not.

c) Constructing the working pad. The starting edge of the geotextile is fixed on firm ground by covering it with fill. In this stage an arrow attack procedure is used (see Fig. 2), backdumping the material and spreading it from the center to the sides utilizing a D-6 type bulldozer which passes 8 times over the fill to provide some practical compaction. The working pad is constructed at full project width including the berms. During this process, settlements and deformations are produced; for this reason the fill material has to be adjusted, redistributed and re-leveled constantly until the working pad reaches the level established in the project (approximately 0.50 m above the water head). (See Fig. 3).

d) Simultaneous working up of the embankment and berms up to the final level of the berms. (See Fig. 4). The berms, 10 m wide with a 4:1 slope are symetrically formed on both sides of the working pad utilizing same materials and using the same construction procedure as in c). The body of the embankment is formed by 0.3 m deep layers of material, compacted until they reach 95% of their maximum dry unit weight, according to the modified Proctor Compaction test. To complete the berms same procedure is used.

e) Completion of the body of the embankment and a 1.0 m deep surcharge measured from top of the body of the embankment following the same construction procedure as used in d). (See Fig. 5).

Once 80% of the predicted settlement is reached, the surcharge material is removed and dumped over the berms. Following this, 0.3 m of material are scarified and recompacted to 100% of its maximum dry unit weight according to the modified Proctor Compaction test. This is followed by the final construction of the pavement.

Photo 1 Aspect of Geotextile sewn on-site with electric portable bag closing machines and 1050 denier polypropylene thread.

5 INSTRUMENTATION AND MEASUREMENTS

In order to evaluate the influence of the use of the fabric in the behavior of the road in areas of soft soils, a research program was developed to include the construction of two test sections along the embankment where Typar fabric was not used and fill was placed directly on the subgrade. The first test section was located 100 m from the fabric section on the swamp zone and the other 100 m from the fabric section on the high compressibility area. This provided 4 control stations: one on swamp with geotextile, another one on swamp without it and also one on highly compressible soils with geotextile and another one on same soil without geotextile. This way it would be possible to carry on a full comparative study. The instrumentation of each station consisted on the use of the following devices whose positions can be seen in Figs. 6 and 7.

a) Nine bench marks, five over ground level and four over the surface of the embankment. Two bench marks with a plate base installed at a depth of 2 m from grade line and a deep bench mark as a fixed reference for all levelings.

b) Five hydraulic cells of 0.20 X 0.20 X 0.18 m installed 2 m under grade line, positioned in different embankment spots.

c) Three Casagrande-type open piezometers and two pneumatic piezometers installed at various depths in both pervious and impervious soils placed under the center line of the section.

d) Three inclinometers of aluminium tubing with a digi-

Photo 2 Fill material for construction of the working pad is back-dumped.

tal electric pendulum with an average depth of 20.5 m in the section without membrane and 26.7 m in the section with membrane.

The installation of these devices permitted the evaluation of settlements, horizontal and vertical displacements and the evolution of pore pressures. Readings were taken periodically over a period of 7 months. It is important to mention that in the section without geotextile the instruments failed prior to the dumping of fill due to the strong ground displacements; for this reason the instruments were replaced after the fill was in place. In addition to the boreholes made before the embankment was constructed, three more were made after construction in the section without fabric and six in the section with fabric, located in the center line and on both sides of the road.

6 RESULTS

The results shown in this paper are the ones obtained in control section #1 which correspond to the non-geotextile embankment portion constructed on swamp and to control section #2 which did use geotextile and was constructed on same type of subgrade soil as control section #1. Table 1 shows the magnitude of vertical displacements measured in the bench marks of both sections during an observation period of 7 months. The location for each bench mark can be seen in Figs. 6 and 7. It can be seen also that the bench marks installed in the surface experienced less settlement in section 2 than in section 1. Settlements for the plate-base benches were practically the same. Two months after construction, a substantial reduction in the rate of settlement was observed, which can be assumed as an indication of the end of the primary consolidation.

Photo 3 Material is spread out from the center to the sides following an arrow attack type of loading. Displacement of the peat can be seen.

Table 1. Settlement values of bench marks in sections 1 and 2

Section 1 without geotextile		Section 2 with geotextile	
Bench No.	Displacement (m)	Bench No.	Displacement (m)
B-1	0.45	B-10	0.11
B-2	0.44	B-11	0.05
B-3	0.24	B-12	0.00
B-4	0.29	B-13	0.02
B-5	0.16	B-14	0.01
B-6	0.33	B-15	0.32
B-7	0.42	B-16	0.21
B-8	0.37	B-17	0.16
B-9	0.25	B-18	0.12
PB-1	0.28	PB-3	0.25
PB-2	0.25	PB-4	0.30

The readings obtained from the hydraulic cells indicate that the maximum settlement occured on the center and on the side of the embankment where the depth of fill and soft soils is greater. It is also observed that in section 2 (with geotextile), the settlement curve shows a more convex pattern than in section 1, which is advantageous since the stress distribution is more homogeneous. The piezometric measurements show that the pore pressure tends to diminish in both sections with time.

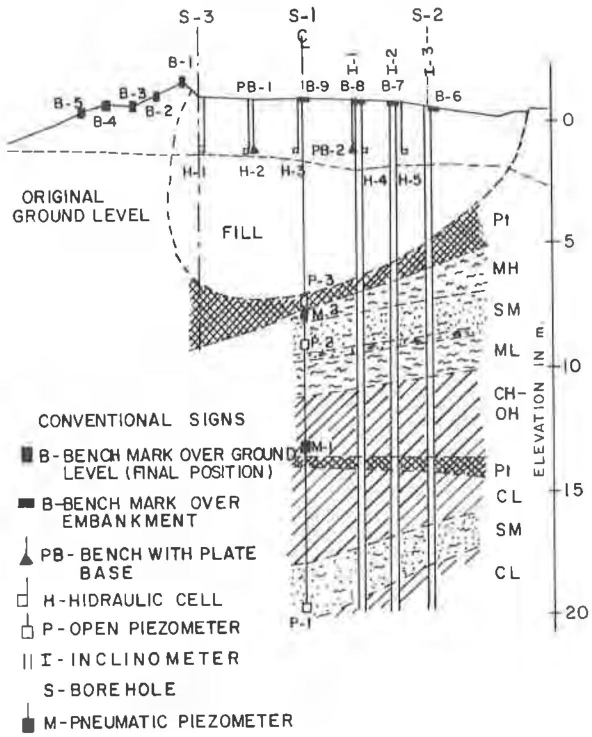

FIG.6 INSTRUMENTATION OF SECTION 1 EMBANKMENT WITHOUT GEOTEXTILE

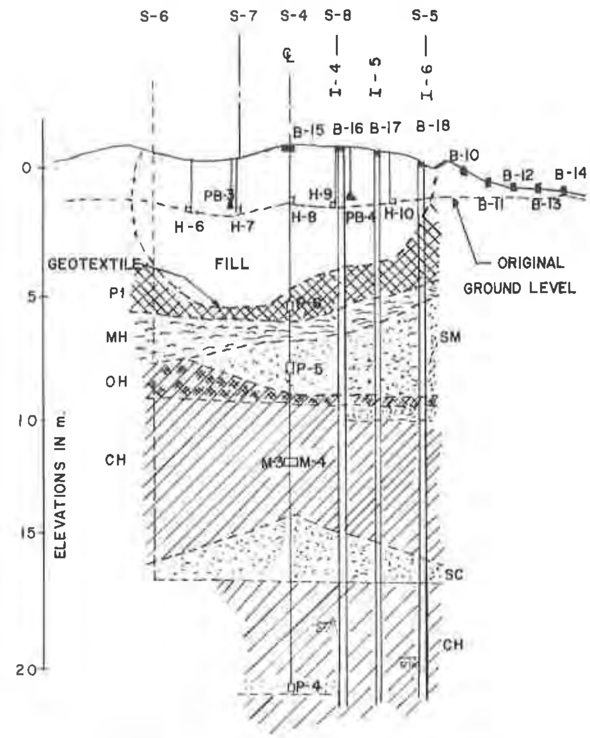

FIG.7 INSTRUMENTATION OF SECTION 2 EMBANKMENT WITH GEOTEXTILE

The excess pore pressure was in average 9.81 kPa, 7 months after the embankment was completed. The horizontal displacements measured by the inclinometers, show that the maximum movements took place in the fill and in the soft soils. Six months after installation of the inclinometers, the maximum horizontal deformations of the fill were 0.035 m in section 1 (without geotextile) and 0.04 m in section 2 (with geotextile). In the soft soils they were 0.04 m in section 1 and 0.017 m in section 2. During the construction operation, strong horizontal displacement of the peat occured over a large distance. Posts situated 50 m apart from the edge of the road were caused to lean. Figures 6 and 7 show the intrusion of the fill into the subgrade determined by boreholes. The fill was found in a loose state, with a value of N=5, except in the first meter, as a consequence of too deep layers of fill placed during construction. A first essential difference between the control sections was found in the initial fill intrusion that took place during and immediately after the placement of the fill. Table 2 shows this difference. It is observed that in general, the depth and thus the volume of the intruded fill is in average two times greater in section 1 than it is in section 2. In both sections an asymmetrical volume of fill is displaced to the left of figures 6 and 7. The causes for this being the inclination and the thickness of the sandy silt layer that lies slightly under the fill. In Figs. 8 and 9 a comparison is made between the stratigraphy before and after the construction of the embankment in the center line of the road for both sections. In these figures it can be observed that the magnitude of the fill intrusion is greater in section 1 than in section 2 as shown in table 2 as well.

Table 2. Intrusion of the fill into the subgrade in sections 1 and 2

Section 1	without geotextile	Section 2	with geotextile
Borehole No	Fill Intrusion (m)	Borehole No	Fill Intrusion (m)
S-1	5.2	S-4	3.4
S-2	3.0	S-5	1.0
S-3	5.1	S-6	3.0

Also, it can be seen that the thickness of the peat layer is considerably reduced in the final stage of the work due to the great displacement of the material induced by the progress of construction and to a lesser degree due to the immediate settlement; this last effect caused a strong thickness reduction on the other soft soils. It should be noted that from a determined depth, the underlying strata were not affected, at least at the end of construction. Figs. 8 and 9 show that in section 1 where the geotextile was not used, deeper strata deformations took place.

7 CONCLUSIONS

As a result of the instrumentation and the observed behavior of sections 1 and 2, the following conclusions were found:

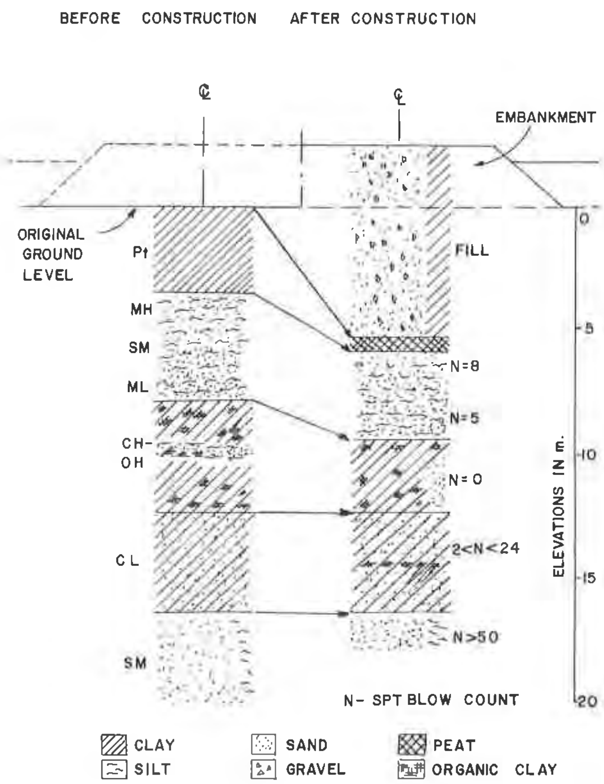

FIG. 8 STRATIGRAPHIC COMPARISON BEFORE AND AFTER CONSTRUCTION OF THE HIGHWAY EMBANKMENT SECTION WITHOUT GEOTEXTILE

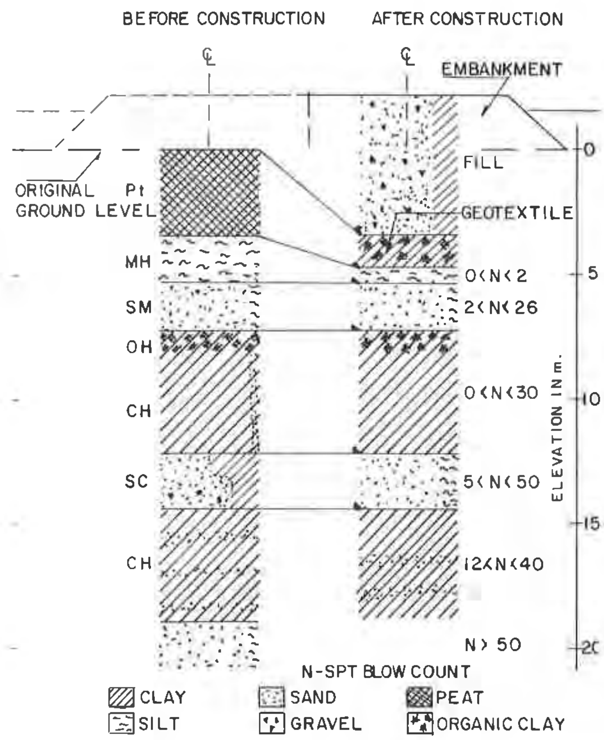

FIG. 9 STRATIGRAPHIC COMPARISON BEFORE AND AFTER CONSTRUCTION OF THE HIGHWAY EMBANKMENT SECTION WITH GEOTEXTILE

a) The maximum intrusion of the fill into the original ground level during and immediately after construction was greater in section 1 without geotextile than in section 2 with geotextile, therefore the use of such material prevented excessive fill consumption for the construction of the embankment. Savings were in the range of 50%.

b) The evolution of settlements was slightly greater without geotextile than with geotextile, but from a practical point of view, it can be said that the settlements occurred in the same way and magnitude in both cases.

c) The horizontal displacements measured in the fill were practically equal in both sections nevertheless the vertical ground displacements were greater in section 1 than in section 2. From this, it can be stated that the restriction imposed by the geotextile reduced the magnitude of the stresses in the contact area between the embankment and the subgrade.

d) The settlement curves reported from the pressure cells measurements showed a more convex pattern in section 2 than in section 1 which implies a more even profile of the embankment base and a more homogeneous stress distribution.

e) From a constructive point of view, the use of the geotextile greately helped the beginning of construction of the embankment by providing a more stable working pad.

f) It is expected that in the future, the more even intrusion of the fill into the subgrade soil, will reduce the need for maintainance since the generation of differential settlement will be less.

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The Behavior of Reinforced Embankment

Le comportement des remblais armés

For the interest about the deformation character of reinforced embankments at Civil Eng.Research Inst.(VUIS) last years a seria of model measurements were managed. Parallel to this work, with the aid of Finite Element Method a mathematical investigation of the deformation of reinforced embankments was managed.
The conclusion of the physical and mathematical modelling is that with reinforcing the embankment a) the horizontal deformation of the embankment and subsoil is substantially influenced,b)it is possible to influence basically the deformation character of the embankment, c)the bearing capacity and also the stability of the embankment increase considerably, d) the stregth mobilization of materials in embankment and in the subsoil is higher when the reinforcement is stiff.

Afin d´ étudier la deformabilité des remblais armés, une série d´essais sur modéles a été effectuée á VUIS ces derniéres années. Paralellement aux mesures, une étude théorique du comportement de ces ouvrages a été menée en application de la méthode des éléments finis.
Les principaux résultats obtenus sur modéles et par analyse théorique sont suivants :
a) I armature influe sensiblement sur la déformation horizontale des remblais et du sol sousjacent, b) elle permet d´agir sur la déformabilité des remblais, c) la portance et la stabilité des remblais augment considérablement, d) en cas des armatures rigides, I activation des résistances mécaniques des matériaux de remblais ainsi que du sol de fondation est beaucoup plus prononcée.

INTRODUCTION

The building of important civil engineering constructions on high embankments based on soft underground is usually a rather complicated task. The inaccurate and unreliable geotechnical parameters of the soils which occur at such a construction, influences the design and it usually results in change of soft subbase layers, in overdesigning the sizes of the fill or in using piles.

During the last 10 years discovered a new way of improving thedeformation characteristics and of increasing the bearing capacity of such fills : through reinforcing the embankments. The technology of reinforcing the fills is simple and therefore it is very atractive for the construction industry. It consists of putting suitable reinforcement into certain layers of fill, otherwise built by traditional technology. At present to the spread of building reinforced embankments defends chiefly the shortage of knowledge about the behaviour of such constructions and the lack of design rules.

In order to obtain informations about the deformations of reinforced embankments last years a seria of model measurements were realized. Paralelly with the mentioned, work has been started on mathematical modelling of the behaviour of reinforced fills with the aid of Finite Element Method.

MODEL TESTS

For model research of reinforced embankment a stand with dimensions 200 x 100 x 50 cm was used. The shape of studied fill is in Fig.1 . The height of the embankment was 23 cm, its wide at the top 15 cm and the inclination of the slopes were 1 : 1,75 . The embankment was built of sand with grain size 0,05 - 2,00 mm with $\phi = 35^{\circ}$. The weak sublayer was built of ballotina of grain size 1,2 - 1,8 mm with $\phi = 20^{\circ}$. The homogenization of the model materials tamped in layers was realized with the help of a contact vibrator. During the tests extraordinary attention was paid to the technology of compacting the mentioned materials for this appeared to be the basis for comparing the results obtained in various type of models.

As reinforcement of the embankment polypropylen woven fabric of strength 15 kN/m with 17 % elongation and a brass sheet 0,15mm thick with strength 210 MPa were applied. In order to increase the skin friction of the reinforcement and so improve the transfer of the shear forces, the surface of the brass sheet was roughened. The embankment was reinforced in one and two layers.

For the establishment of the deformation field of the modeled constructions a device functioning on the basis of inductive coupling of two coils was used (Bison Instr.). Two

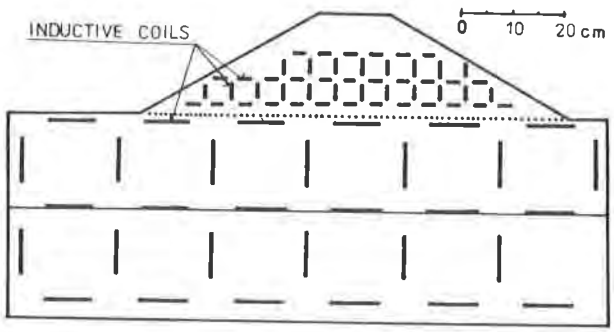

Fig.1 The position of measuring coils in the embankment

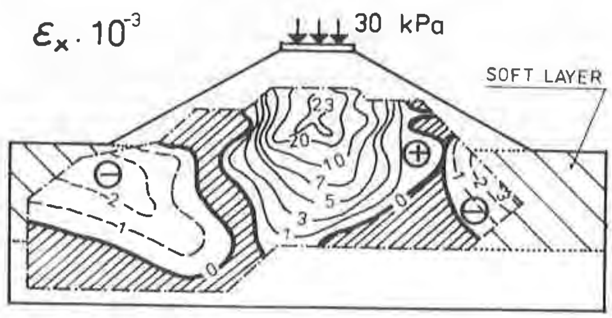

Fig.2 Izotens of horizontal strain
The embankment is not reinforced

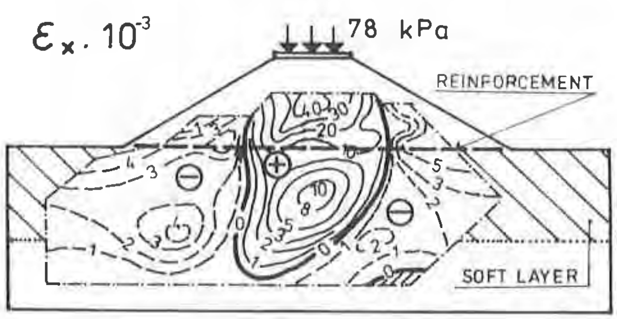

Fig.3 Izotens of horizontal strain
Stiff reinforcement

types of coils were used : with diametres 2,5 and 10 cm. The distribution of the coils in the models is indicated in Fig.1 . During the model measurements the distances were measured by the aid of the little coils with an accuracy of 0,015 mm and with the larger ones with accuracy 0,08 mm . Together 81 coils had been built into the models and with their help it was able to obtain the necessary information about the relative deformation of the environment in horizontal direction in 45 sub-zones of the fill and in 26 sub zones of the "subsoil". In vertical direction the information came from 25 sub-zones of the fill and from 17 sub-zones of the subsoil. Through interpolation of the date obtained in this way it was possible to constract the isolines of the relative horizontal, vertical and shear deformation of the fill and the subsoil at various loads applied on the top of the embankment in the form of uniforme pressure and with various reinforcement of the embankment.

Some of the results obtained in this way are indicated in Fig.2 and 3 . Fig.2 contains informations about the izotens of relative horizontal deformation at 30 kPa uniforme pressure on the top of the embankment. Fig.3 illustrate izotens of relative horizontal deformation at 78 kPa uniforme préssure on the top of the reinforced embankment. From Fig.2 and 3 follows that by reinforcing the embankment in one layer with a relatively stiff reinforcement the horizontal deformation of the subsoil decreses by more than 50 %. Proporcionally to this value growes the bearing capacity of the fill. It is evident, that in this case by the aid of a single layer of stiff reinforcement, the bearing capacity of the embankment rose 160 %.

FEM CALCULATIONS

During the preparatory work of this calculations the best finite elements from the mathematical and geometrical point of view were sought. E.g. among others models with 5000 elements were examined. Attention was also paid on the selection of the constitutive relations of the materials. The ellected relations take into account : a) the initial state of materials given by the compacting of the fills and by geostatic stress state of the soils, b) the hardening of the fills with increasing normal octahedrical stress level, c) the loosening of the fills with with increasing octahedrical shear strength mobilization, d) the failure of the materials were simulated by limit values of the deformation modulus and the Poissons ratio. At the incremental calculations the tangent stiffness method was used and so the non-linear behaviour of materials were obtained through linearized steps.

The tangent values of the deformation modulus E and of the Poissons ratio were computed for every loading step according the relations (1).

$$E_i = E_o + k_E \cdot \sigma^p_{oct} \, (1 - i^n)$$

$$\mu_i = \mu_o + k_\mu \cdot \sigma_{oct} + \qquad (1)$$

$$+ \, / \mu_{max} - (\mu_o + k_\mu \cdot \sigma_{oct}) / \cdot i^m$$

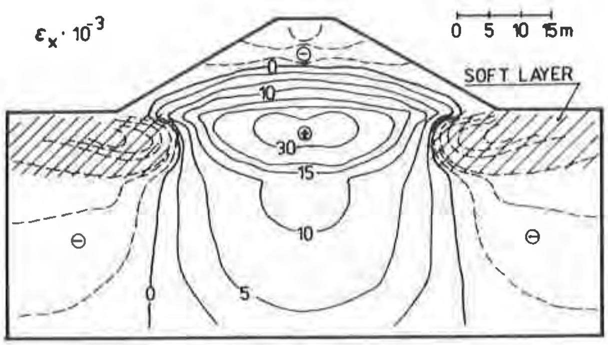

Fig.4 Izotens of horizontal strain

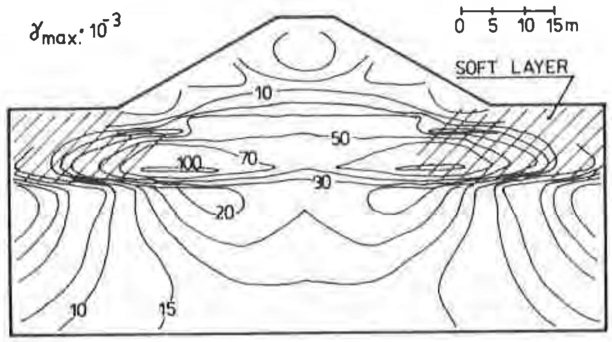

Fig.7 Izotens of shear strain

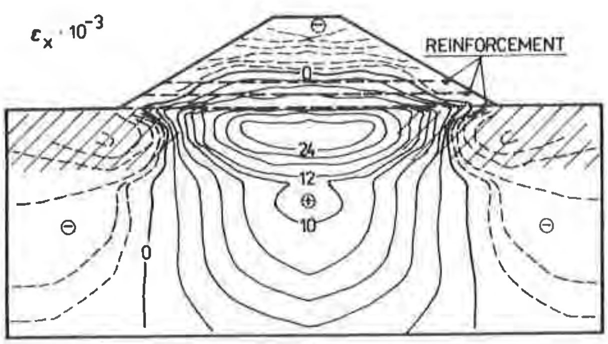

Fig.5 Izotens of horizontal strain
Woven textile reinforcement

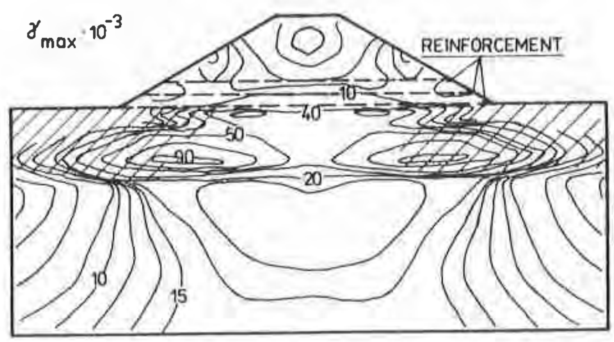

Fig.8 Izotens of shear strain
Woven textile reinforcement

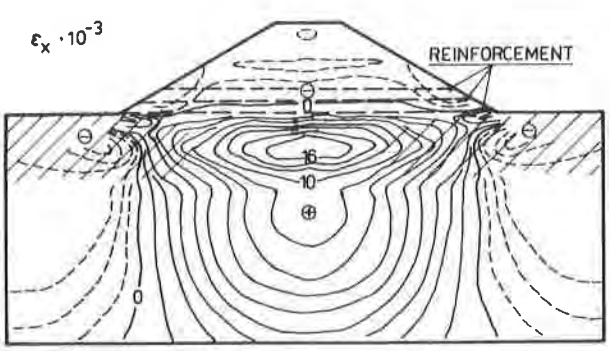

Fig.6 Izotens of horizontal strain
Metal reinforcement

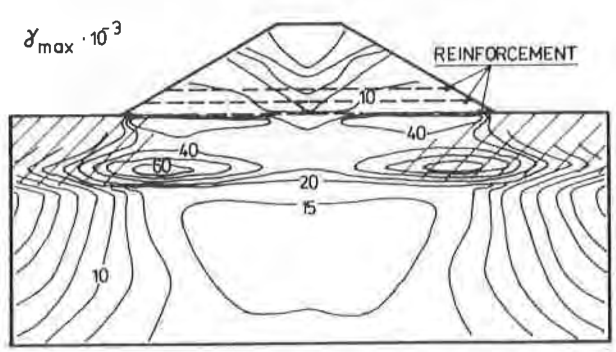

Fig.9 Izotens of shear strain
Metal reinforcement

In (1) is :

E_0 , μ_0 - the initial values of the deformation parameters

k_E , k_μ, p - hardening parameters expressing the influence normal octahedral stress

$$i = \frac{\tau_{oct}}{\tau_{oct}^{limit}}$$ - the level of shear strength mobilization

The embankment was modelled as GW soil with $\phi = 35^\circ$, the weak layer of the subsoil as CH soil with $\phi = 5^\circ$, c = 20 kPa. The sizes of the analyzed embankment are evident from Fig.4. The fill was reinforced in one bis three layers. The reinforcement had various stiffnesses. The embankment was loaded incrementally by his selfweight and alternatively also with various uniform pressure which acted on the top of the embankment - like at the physical model measurements.

Some results from these parametrical studies are in Fig.4 - 9 . Fig.4,5,6 give us information about the izotens of horizontal strain in and around the embankment reinforced and not reinforced. Fig.7,8,9 give us information about the izotens of shear strain in and the embankment reinforced and not reinforced. Fig.5,8 give information about the influence of woven textile reinforcement. Fig.6,9 give information about the influence of metal reinforcement.

From these figures it is evident, that the woven textile reinforcement - when situated in three layers - in the embankment, is capable to diminish the horizontal normal strain and also the shear strain in the weak subsoil roughly by 20 % . At the same situation the metal reinforcement is capable to diminish both type of strains in the subsoil roughly by 50 % .

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CONCLUSION

From the physical and mathematical model measurements and calculations in the case of reinforced embankments it can be drawn the following general conclusions :

- The vertical deformation of the subsoil is little influenced by the presence or absence of reinforcement in the embankment, while the horizontal deformation is substantially influenced with it.
- By reinforcing the fills based on weak subsoils both, the bearing capacity and also the stability of the embankment increase considerably.
- With reinforcing the fills, it is possible to influence basically the deformation character of the embankment.
- With stiff reinforcements the loaded fills reach much higher strength mobilization.
- In the case of stiff reinforcement in the vicinity of reinforcement the deviator stress is lower what has a considerable impact on the way of loading the subsoil layers.

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A Rapid Banking Method Using the Resinous Mesh on a Soft Reclaimed Land

Méthode de remblayage rapide utilisant le filet résineux sur le terrain mou remblayé

A case history of the rapid embankment for the highway on the extremely soft reclaimed land was successfully performed by spreading the resinous mesh over the ground. The mesh has been hitherto employed as a sort of the bamboo method. The broad embankment for the reclaimed land should have been slowly filled up without any remolding of the ground. Therefore, the mesh has played a role as the restrained layer exerted between the embankment and the ground to reinforce the earth. The main scope of the present case history is, in contrast with the conventional use, to carry out the rapid displacement of the extremely soft soil into the transported soil. Different from the forced displacement without any kind of geotextiles, in this case, the ultimate settling and spreading of embankment do not develope unlimitedly because of restrainment of the resinous mesh. Resultantly, the two layered ground forms the natural pattern of the foundation. Consequently, the current banking method is characterized by the possibility of the rapid and economical embankment using the ordinary construction machines.

Un cas de remblayage rapide pour la route sur le terrain extremement mou a été effectué, en posant le filet résineux sur le terrain. Jusqu'à présent, le filet résineux était utilisé comme une sorte de "méthod de bamboo". Dans la méthode habituelle, le remblai vaste pour le terrain de remblayage devait être entassé lentement sans aucun rebuttage de terrain. Par conséquent, le filet jouait un rôle comme couche restreinte entre remblai et terrain pour renfocer le sol. En contraste avec l'usage conventionnel, le présent cas de remblayage a pour but pricipal deffectuer le déplacement force rapide de sol extrêmement mou par le sol transporté, en plus de l'acquisition de la facilité de circulation des machines de construction. A la difference du déplacement forcé sans géotextile, dans le présent cas, l'affaissement illimité de remblai ne s'est pas developpe a cause de la restriction de la couche de borne par le filet résineux. Il en résulte que le terrain à deux couches a formé le type naturel de la foundation. En conséquence, la présente méthode de remblayage se caractérise par la possibilité d'un remblayage rapide et économique utilisant les machines ordinaires de construction.

1 INTRODUCTION

Recently in Japan, the reclaimed land has been very rapidly constructed near the shore because of the economical demand. The reclaimed lands for every use have been increased up to about 540 million m^2 since 1960. Most of them consist of the extremely soft clay soil which is transported from the off-shore by the dredger and is sedimented.

For requirement of the site of near-shore structures such as the factory, the storage facility and the electric power plant, there have been built the stable structures whose surface layer of the soft ground is improved into the rigid plate by means of cement or lime stabilization. Besides, the geotextiles such as the permeable cloth and the resinous mesh treated in the paper have been prevailed to fulfill its function as the restrained layer in the ground, while the design procedure for stabilzing the surface layer of soft grounds and the mechanism of the interaction between soil and geotextile have not completely been known.

The aim of this paper is to present a case history where the displacement method using the resinous mesh on the extremely soft reclaimed land is employed for the road embankment. Also, the reasonable and economical design procedure is proposed on the basis of the results of model footing tests and field earthworks.

2 CONCEPT OF EARTHWORK METHOD USING RESINOUS MESH

2.1 Conventional Use of Resinous Mesh

The effectiveness of a resinous mesh for earthworks on soft grounds has been continuously explained at laboratory and at field by Yamanouchi (1967, 1970, 1975, 1980). The principle of the earthwork method using the resinous mesh is almost the same as the fascine work which had been practiced from old times. Yamanouchi has proved that the resinous mesh is available for preventing from the penetration of subbase materials into the subgrades, the pumping up and the deflection of the pavement. At the present time, in addition to these uses, the mesh is used jointly with the other stabilizing method, for instance, sand drain, for the purpose of the improvement of not only the surface but also the depth in soft grounds. The conceptual description of the difference in these methods is illustrated in Fig. 1.

It can be suggested from past experiences that the method using the resinous mesh for improving the vast reclaimed land is classified as follows :

(1) Belt-like embankment for highway ($B/D < 2 \sim 3$)
(2) Over-spreading earth for construction of the extensive site ($B/D > 2 \sim 3$)

where B : the width of embankment, D : the depth in stabilized layer. Most of the case histories where the mesh has been utilized until the present time belong to the category of the case (2). Since even in the case (1) the placement of the resinous mesh is done on the basis of the same idea, the post-construction settlement due to consolidation of soft clay beneath the stabilized layer continues over the long term period. Hence, for embankment on soft grounds where the field condition fits in with the case (1), the forced displacement method using the resinous mesh which is described in the present

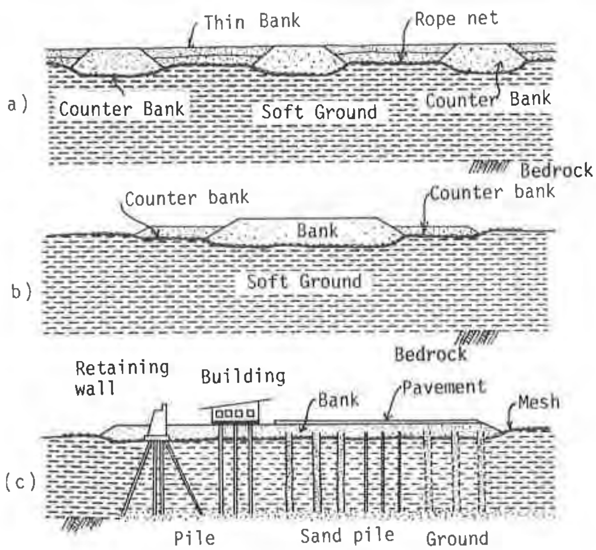


Fig. 1 Examples of earth reinforcement using resinous mesh

paper will be recommended in order to decrease the post-construction settlement of soft ground as little as possible.

2.2 New Banking Method using Resinous Mesh

There is shown in Fig. 2 the typical ground section where the resinous mesh is spreaded over the soft ground for the surface stabilization. In laying the mesh over the minimum area of the surface of ground, the subsoil is replaced by as much the banking soil as possible,by taking advantage of the temporary decrease of the bearing capacity of the ground due to remolding. Thus, the efficient displacement will be attained and the reasonable layered ground is formed as shown in Fig. 3 since the tensile force of the mesh itself and the restrained effect among the mesh, the banking soil and the soft soil is exerted sufficiently.

The above-mentioned method is characterized by the advantageous features as follows :

1) Since clay or silt layer is decreased by the displacement, we can minimize the post-construction settlement.

2) Stress developed by traffic loads is smoothly distributed because of the rigidity and the homogeneity of the fill material.

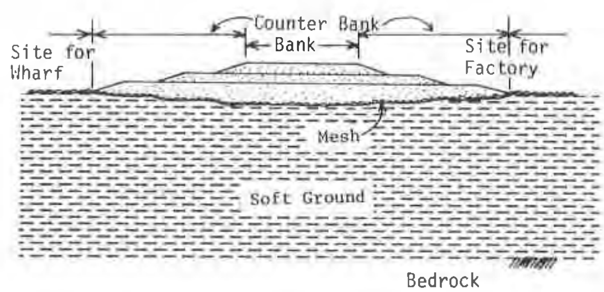


Fig. 2 Conventional earthwork method using resinous mesh

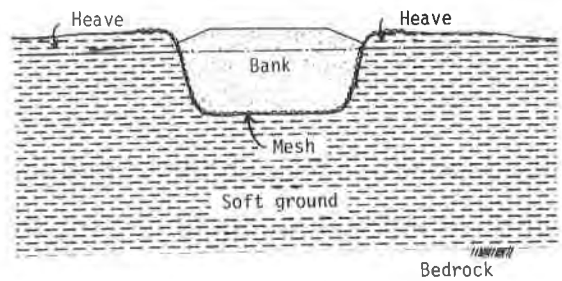


Fig. 3 Displacement method using resinous mesh

3) By displacement of the soft ground into the permeable fill material, it also works as a kind of the sub-drainage.

3 MODEL FOOTING TEST

3.1 Purpose of Model Test

As a simulation of the displacement method using the resinous mesh, the several series of model footing tests are carried out at laboratory to investigate deformation characteristics of the ground involving the mesh during earthworks. Also, test results are expected to provide information available for the design procedure and the methods how to connect the mesh each other and how to spread it over the soft ground.

3.2 Outline of Test

As shown in Fig. 4, the two-dimensional plane strain type footing model (4m in width, 1.0 m in height and 0.5 m in length) is made of the lucite. The mixture of clay which was taken from the reclaimed land at the port of Kanda is poured into the apparatus. The average index characteristics are as follows : Specific gravity, G_s = 2.63, liquid limit, w_L = 113%, plasticity index, I_p = 76, initial water content, w_I = 130%, compression index, C_c = 0.58 and swelling index, C_s = 0.09. The sawdusts made of the lucite as the mark are attached at the corner of

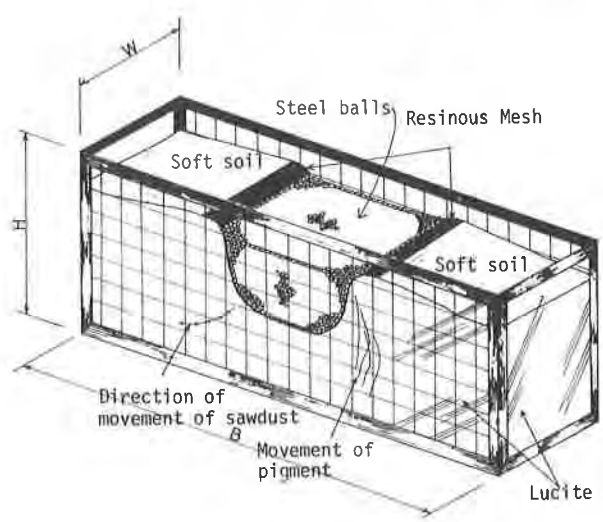


Fig. 4 Outline of apparatus of model footing test

the square comparted by 10 cm x 10 cm to observe the deformation behavior of soft ground. The so-called "mosquito net" is used instead of the resinous mesh with consideration of the similitude between field and laboratory. As the load, the steel balls with 38 kN/m^3 as the unit weight (2.5 cm in diameter) are spreaded over the ground surface to keep unity as B/D. The artificial clay ground has been cured for two days after pouring. Loading is done 5 times step by step. Each loading duration is about 2~3 hours. The total stress is accumulated up to 7.64 kPa on the average.

3.2 Kinds of Model Tests

Four kinds of model footing tests are run to simulate the following situations :

1) Fill with the mesh not laid

2) Fill with the mesh laid to observe the lateral flow

3) Fill with the mesh laid to observe the movement of soil particle and the lateral flow

4) Fill with the mesh laid over the ground interbedded by the sand seam

Change in index propoerties of the soft soil was examined before and after loading. The distinct difference of them was not recognized. This implies that the loading by means of the steel balls is done very rapidly as if in a manner of the undrained-unconsolidated test.

Representative observation is illustrated in Figs. 5 and 6. Behavior of the soil especially seen at the series (3) of the model test is pursued by the movement of the white pigments painted in the surface of the front lucite. The number in Fig. 6 shows the sequence of observation at the end of each loading. According to Fig.6, the following tendencies are appeared :

Fig. 5 Model footing test for the banking method using the resinous mesh (test series (4))

The ground at the vicinity of the center of the loading plate moves diagonally downwards, the ground beneath the fill shifts almost horizontally and the ground at the toe of fill moves diagonally upwards, then the heaving of the ground occurs. It is observed at the line outside the embankment that the soft soil is thrusted out at the same time of loading, then this movement reaches gradually at the depth from the surface. The lateral flow of the depth progresses in parallel with the longitudinal section. On the contrary, the surface movement is extremely complex. As a whole, the movement of the painting is in agreement with the movement of soil particles.

Thus, from the results of model tests, we can approximately grasp a behavior of the layered ground including the mesh. Besides, test results suggest that we should pay attention to the following items in filling embankment at field for displacement using the resinous mesh.

1) When the subsoil is replaced by the transported soil, the tensile strength of the mesh is required. Otherwise, the mesh is torn by the shearing force acting between soil and mesh with accompany by the embankment. Hence, the mesh must be reinforced by the rope or the cable.

2) Observation of the ground movement is essential in execution of the earthwork at field from the beginning of embankment especially for settlement prediction.

4 EMBANKMENT ON THE RECLAIMED LAND USING RESINOUS MESH

4.1 General View of the Port of Kanda

The port of Kanda (Fig. 7) which is located in the northern part of Kyushu in Japan had played an impotant role to export the coal produced at the Chikuho area which was the background of the heavy industry in Kitakyushu. Recently, it has been assigned as the base of the timber import. The construction of the timber storage and its relative facilities was started in 1972 for the expence of about thirty hundred million yens and then they had been completed in 1977. When the wharf facility for the timber and the factories of the timber relative industries are constructed, the port of Kanda involving the Matsuyama area where the fill was completed by using the resinous mesh must be an international base port for importing and manufacturing the timber. In addition, since the big factory of the Japanese famous automobile company was constructed on the reclaimed land of the port of Kanda near the Matsuyama area, the amount of the invisible trade in Kanda has been rapidly accumulated. For these reasons, the necessity of the road construction has come up, which is expected to connect this wharf with the national highway Route 10 as one of the arteries of Kyushu. The outline of the layout of the

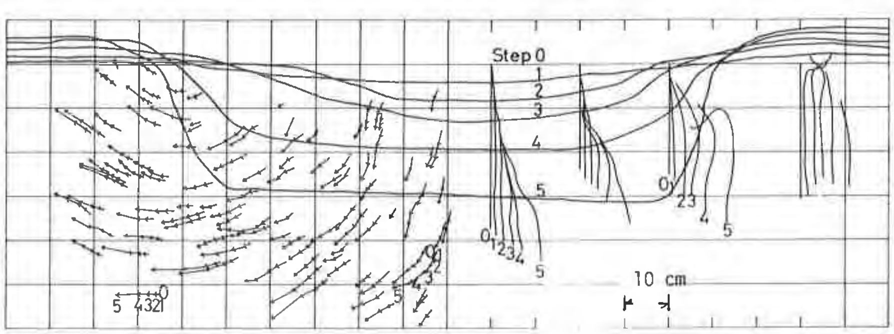
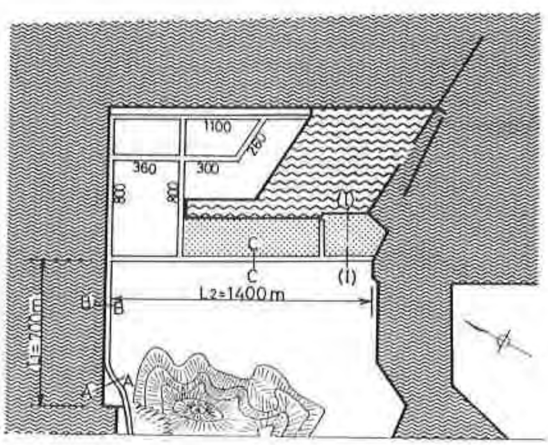


Fig. 6 Movements of soil particle and soil mass due to step loading (test series (3))

Fig. 7 General view of the port of Kanda

facilities and the planned line of the road in the Matsuyama area is illustrated in Fig. 8. The road passing through the wharf facilities for such as the storage of timbers and the factory site are constructed over the extremely soft reclaimed land at Kanda of northern Kyushu. The cross section along the I - I line in Fig. 8 is reproduced in Fig. 9. The partitioning bank between the reclaimed land and the sea is built to surround the desired area of the sea (step 1). The soft soil sometimes including the sand seam is poured through dredging into the shallow sea compartment divided by the surrounding

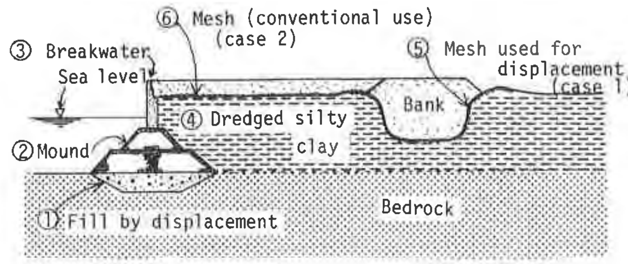


(Dimension in distance : m)

Legend

Sea
Revetment bank (step 1)
Reclaimed land (step 2)
Road by the case 1 (step 3)
Wharf site by the case 2 (step 3)

Fig. 8 Outline of the reclaimed land where road fill is constructed using resinous mesh (Matsuyama area)



(Number in the figure shows the order of earthwork)

Fig. 9 Cross-sectional view at the I-I portion in Fig. 8

bank (step 2).

Index properties of the soft soil taken from the site for the road embankment using the mesh are tabulated in Fig. 10.

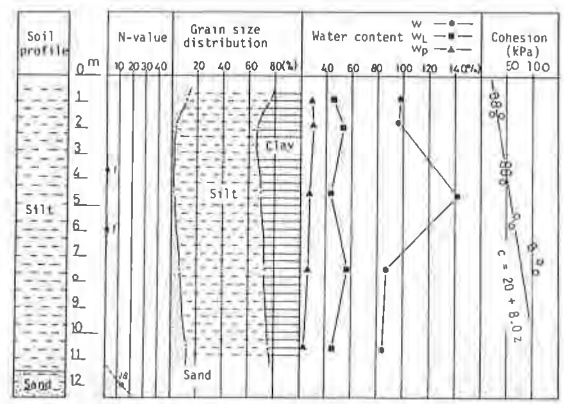


Fig. 10 Index properties of the soft soil before embankment

In the case history described in the present paper, fill for the road is carried out by following the procedures :

1) The resinous mesh is laid over the surface of the soft ground to acquire the trafficability of the heavy construction machines as shown in Fig. 11 (a). At the beginning of laying over the ground, the resisting force of the mesh is exerted. Movement and settlement of the ground are controlled by restrainment developing between the soil and the mesh.

2) To reinforce the mesh, as was already pointed out in the model test, the woven fiberous rope is tied to the mesh at intervals of about 1 m as shown in Fig. 11 (b).

3) The fill material transported from the hillside near the Matsuyama area is laid over the mesh by the bulldozer. Then, after laid soil by 30-40 cm depth, the soil is uniformely compacted under repetition of travelling. This procedure is repeated several times. The restrained layer between the mesh and the soils is formed to resist the settlement due to consolidation and plastic flow.

4) According to the rapid embankment, the lateral plastic flow is occurred. Then, the softsubsoil is replaced by the fill material. Thus, the fill and the ground remains stabilized. Typical cross section was schematically shown in Fig. 3.

5) Due to the lateral flow, heaving is extended over the

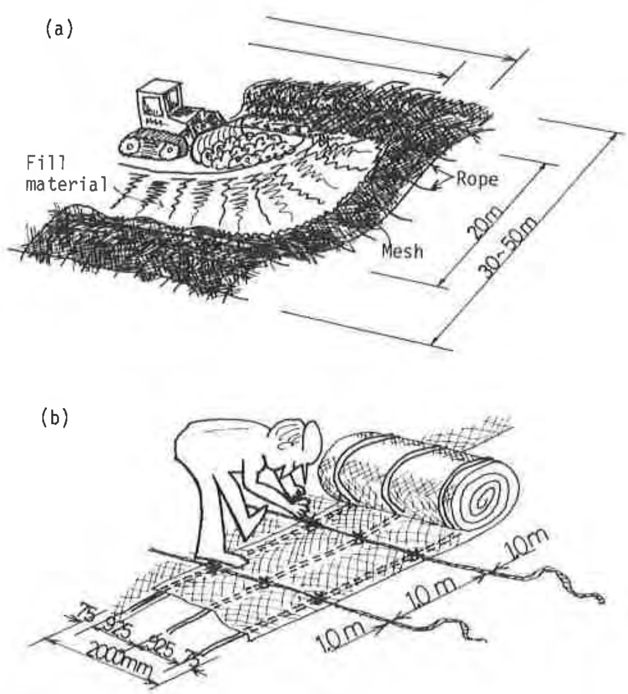


Fig. 11 General view of performance using resinous mesh

surface with 60 m on the both sides of the fill. The heaving surface functions as the counter weight.

6) The alternative repetition of laying the mesh, filling the soil and compacting the layer continues along the traverse direction of the road.

Movement and behavior of the ground due to embankment are observed by the pore-water pressure indicator, the settlement indicator, lateral inclinometer and the displacement pile which are installed under the ground in depth and the surface. Among them, it is worth paying attention to the movement of the piles at the surface. Based on the the observations of the movement of the pile, it is concluded that the depth and extent of the heaving and displacement along the longitudinal section after final fill, as visualized in Fig. 12, varies depending on the index property, the sand seam and the speed of filling.

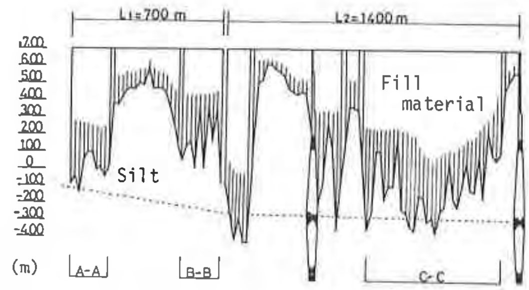


Fig. 12 Profile of ground with the base established by displacement

Every symbol in Fig. 12 corresponds with that in Fig. 8. The typical scene where the mesh is laid over the soft soil is demonstrated in Fig. 13.

5 TENTATIVE DESIGN PROCEDURE

The displacement method using the resinous mesh is proved to be valid even for embankment over the extremely soft ground. However, the design procedure at field has not been established. It is also too difficult to estimate the settlement due to the displacement because settlement contains consolidation and a large amount of plastic flow arising from the rapid embankment. The key to the design of this method seems to exist on the comprehension of the interactions between the mesh and the

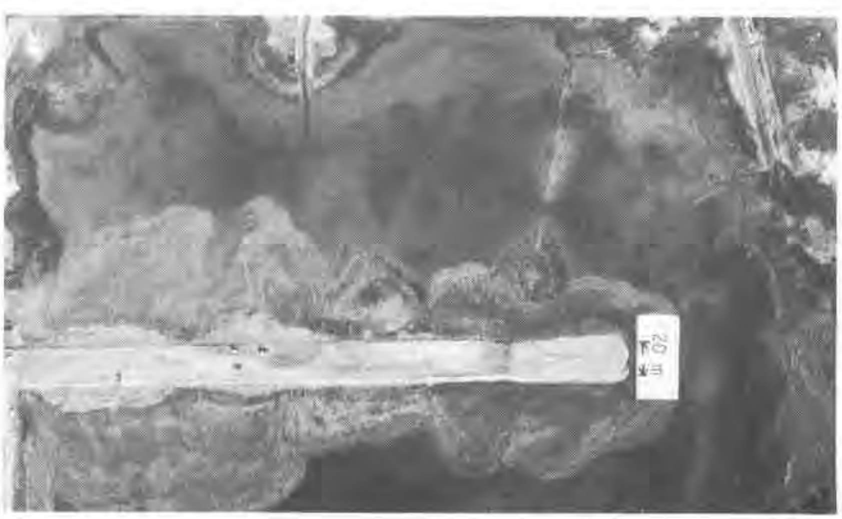


Fig. 13 Embankment over the reclaimed land using resinous mesh

soil accompanied by loading. Yamanouchi et al. (1970) run the experiments at laboratory for the understanding of this mechanism.

Owing to the review of methods for calculating the allowable bearing capacity of clay strata, the following formula is adopted to propose the method for estimating the amount of the transported fill material needed for embankment.

$$q_a = \gamma H + 5.3\,c \qquad (1)$$

where q_a ; allowable bearing capacity, H ; depth of the fill, c ; cohesion of silt. Fig. 14 schematically shows the profile of the subsoil after embankment with its base established by displacement.

Since the extended width of the heaving was three

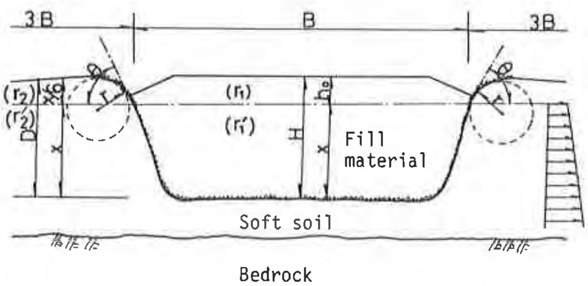


Fig. 14 Assumed cross-section for estimating the quantity of the transported soil

times of the width of the bank (3B = 60 m) at the field observation, the average height of the heaving is reduced to $(1/2)\cdot B\cdot x \div 3B = x/6$. An attempt to take the effect of the tensile strength of the mesh into consideration in settlement computation is based on the method proposed by Yamanouchi et al. (1979). They obtained the approximate equation for calculating the allowable bearing capacity with consideration of the tensile strength of the mesh as follows :

$$q_a = \frac{1}{F}\left[5.3\,c + T\left(\frac{\sin\theta}{B} + \frac{1}{r}\right) + D_f F_s\right] \qquad (2)$$

where symbols may be referred to Fig. 14. From the balance of the forces acting on the plane between the soil and the mesh shown in Fig. 14, we have

$$\gamma_1 h_o + \gamma_1' x = \gamma_2' x + \gamma_2 \cdot \frac{x}{6} + 5.3\,c + T\left(\frac{\sin\theta}{B} + \frac{1}{r}\right) \qquad (3)$$

Substitution of every value of unit weights per volume (γ_1 = 18 kN/m^3 γ_1' = 18 kN/m^3, γ_2 = 14 kN/m^3, γ_2'=15 kN/m^3) and cohesion (c = 20 + 0.8 z) into Eq. (3) gives

$$H = 1.16\,x + 1.22 \qquad (4\text{-a})$$

or

$$h_o = 0.14\,H + 1.06 \qquad (4\text{-b})$$

Thus, we can obtain the relationship among the height of the fill, h_o, the depth of the displacement, x, and the total height of the fill, H, computed by Eq. (4-a) as shown in Fig. 15. Therefore, by utilizing Fig. 15, we can estimate the amount of the transported soil mass and then the reasonable design and the economical estimation

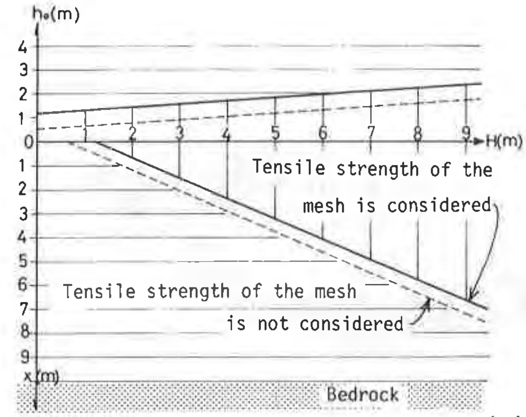


Fig. 15 Design chart for displacement method using resinous mesh

can be carried out before earthwork using the resinous mesh.

6 CONCLUSION

From the model tests and the field earthworks, the new embankment method using the resinous mesh was suggested to become one of the most economical and reasonable methods for improving both the surface and the subsoil in depth of the extremely soft reclaimed land. This method was resultantly used for displacement of the soft soil. Hence, this method should be separately considered from the improvement of the broad area of the soft ground which has been conventionally ever adopted. By strengthening through the connection of the fiberous rope with the mesh each other, the replacement is reasonably advanced by the mesh and the ground in a body.

The road in the Matsuyama area improved by this displacement method is now under opening to traffic after paving. A portion of the embankment filled up by the preloading was removed because the road body has remained completely rigid and there have been occured no troubles so far.

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Utilization of Geotextiles in Waste Management

Utilisation des géotextiles dans les décharges controlées

Landfill and industrial waste impoundment systems require special design methods due to the required functions of the system and the unique characteristics of waste materials. Geotextiles have proven to offer cost effective design alternatives that solve some of the problems associated with impoundment design in a wide variety of waste management applications. To increase awareness of geotextile utilization in landfill and industrial impoundment design projects, six applications are presented. The applications include haul road construction over unstable waste deposits, dike construction, leachate bed drainage design, intermediate drainage layers, observation wells, and final containment cover design. Design considerations and techniques, fabric requirements, and construction considerations are discussed. Emphasis is given to special fabric requirements unique to these applications and available design techniques are referenced. A case history of geotextile use in each application is reviewed.

Les systèmes de décharge pour remblais et déchets industriels nécessitent des méthodes de dimensionnement spéciales, étant donné les fonctions que le système doit remplir et les caractéristiques unique des décets. Il est bien connu que les géotextiles offrent des alternatives avantageuses au point de vue coût et que leur utilisation aide à résoudre quelques-uns des problèmes de dimensionnement dans une grande veriété d'applications de décharges contrôlées. Pour mieux comprendre l'utilisation des géotextiles dans les projets de décharge de remblais et de déchets industriels, six applications sont présentées. Ces applications sont les suivantes: les voies provisoires non-revêtues construites sur décharges instables, la construction des digues, le dimensionnement des lits de drainage pour eaux vannes, les couches de drainage intermédiaires, les puits témoins et le dimensionnement du revêtement supérieur. Les considérations et techniques relatives à la conception, les besoins en textiles et les considérations concernant la construction sont traités. Un antécédent concernant l'utilisation des géotextiles est examiné pour chaque application.

INTRODUCTION

Successful design and operation of industrial waste impoundments is, in the experience of the authors, often complicated by the unique character of the waste. Industrial wastes often are produced in the form of sludge type materials characterized by high moisture content and low inherent stability. Furthermore, waste materials frequently have a larger organic fraction which, through long-term decomposition, produce gases which further decrease the stability of the mass.

One approach with the disposal has been to impound the waste without extensive pretreatment. Incorporated into the landfill design must be a mechanism to induce consolidation of the sludge and associated release of pore water (leachate), channels to route the leachate to collection and/or treatment facilities, and a media for collection and venting of gas. The primary objectives of the design are to: 1) improve stability such that future use of the area may be feasible, 2) decrease the volume of the waste through consolidation with a minimum of cost for raw materials, and 3) minimize risk to the environment and include an environmental monitoring plan.

In recent years, geotextiles have become increasingly utilized as problem solvers in design and operation of waste impoundments, to accomplish the objectives noted previously. The function of geotextiles in impoundments can be broadly grouped into three areas: 1) drainage, filtration; in which the fabric serves as a medium for water movement or allows movement of the water and leachate while retaining the waste materials, 2) separation; in which layers of different particle sizes are separated by the geotextile, and 3) reinforcement; in which the geotextile is used as a reinforcing element in the landfill through direct earth reinforcement or stress redistribution.

The authors' experience has involved geotextile usage in the following landfill applications, each of which involves one or more of the three main functions:

1. Construction of haul roads over waste deposits and soft ground.
2. Direct soil reinforcement in construction of dikes.
3. Construction of leachate drainage beds.
4. Support of intermediate drainage layers.
5. Construction of observation wells.
6. Placement and support of final cover materials.

This paper contains a discussion of design considerations, fabric requirements, and construction considerations related to these applications, as well as a review of case histories involving the use of geotextiles in these modes.

HAUL ROAD CONSTRUCTION

Two distinct types of haul roads are often utilized in operation of landfills. The first of these consists of haul roads leading to the site. Access roads must be constructed and maintained throughout the service life of the facility. Since landfills are frequently constructed in areas with marginal soil conditions, soft ground roadway construction techniques are generally required. Haul road construction over soft ground using geotextiles is widely discussed in the literature (1) (8). Design procedures presently available are both useful and adequate.

Haul road construction over waste deposits, however, are typically more difficult. Haul road construction over the waste materials may be required for site access, waste handling procedures, or placement of cover materials. The strength of waste deposits are often so low that pedestrian access is not possible with any significant degree of safety. In these instances, the available design procedures have proven inadequate as the waste exhibits little or no shear strength. Displacement design methods such as those used by Haliburton, 1980 (5) may not be practical. Through a trial and error approach, it has been found that the haul roads must essentially be floated over weak waste deposits. In this regard, geotextiles have been used in conjunction with a light weight fill material such as bark to produce a "buoyant" road.

In our experience, bark has been successfully utilized for haul road construction because the unit weight is typically less than 2/3 that of the waste. Other light weight materials, such as sawdust, logs, cinders, and "popcorn" slag may also be considered. However, all of these materials, including bark, are particulate and when used separately, tend to mix with sludge over time. This reduces the service life of the road. Geotextiles solve the mixing problem by separating the bark from the waste and preventing extensive waste intrusion. Thus, the prime design criteria in selection of the fabric is low porosity and, as such, most available geotextiles can be utilized. When used with "string" bark, strength properties of the fabric are secondary considerations of lesser importance due to the high intrinsic strength of the bark. It has been observed that light weight woven or non-woven fabrics are an economical selection and perform satisfactorily. However, tensile strength of the fabric may be of primary importance when used with other light weight, more granular aggregates.

A paper mill waste water treatment residue landfill constructed in Central Wisconsin illustrates this use. The general layout of this site is depicted in Figure 1. The landfill was triangular in shape and 11 hectares in size, with the fill beginning at one apex and proceeding radially to the opposite side. The depth of waste increased linearly away from the apex by ramping upwards. Tandem axle dump trucks were utilized to transport the new materials over previously deposited waste to the active fill areas. After several trials of alternate haul road designs, the workable solution consisted of first laying a light weight non-woven heat welded spun bonded geotextile over the waste to the desired width of the haul road. The fabric was then covered with approximately 0.6 m of bark and 0.3 m of gravel. This procedure has been used successfully to progressively advance the haul road as the fill base proceeded.

FABRIC REINFORCED DIKE CONSTRUCTION

Construction of impoundment areas usually involves construction of one or more earthen dikes. Dikes are generally required around the edges of the impoundment to form the impoundment area. These dikes are similar to earth dam designs. External slopes are generally designed at 3 horizontal to 1 vertical with internal slopes designed at 2 horizontal to 1 vertical. Internal slopes may be steepened to take advantage of the additional lateral support provided by the waste material after filling. Even in depressed areas, a dike may be required adjacent to the existing embankment to provide an impermeable liner and meet other environmental constraints. These dikes generally consist of clayey soils compacted against the embankment over a specified horizontal distance, then designed with a particular slope into the impoundment area for overall stability. A third type of embankment may be constructed within the impoundment to form cells which are used to separate disposed materials. These dikes allow for stabilization of one area while filling operations continue in another. Internal dikes generally have steep side slopes on the order of 1.5 horizontal to 1 vertical. As the impoundment area is filled, dike construction may be required out over disposed waste materials, especially with internal cell dikes. Figure 2 illustrates the different types of earthen dike design.

Problems associated with conventional dike construction involve import of offsite materials, decrease in usable impoundment area, construction in marginal soil areas, erosion of embankment slopes, and differential bearing over existing dike and waste materials. The latter problem results in sagging edges and limits the height of construction.

Utilization of geotextiles to construct retained earth dikes offers a unique alternative design that can be used to maximize disposal volume and minimize cost by decreasing the amount of soil and time required for dike construction. Utilization in each type of dike construction is shown in Figure 2. In retaining structures, geotextiles are placed in the backfill to give the backfill potential resistance, and thereby, reduce earth pressure against the wall. In addition, geotextiles can be used to control erosion problems at the

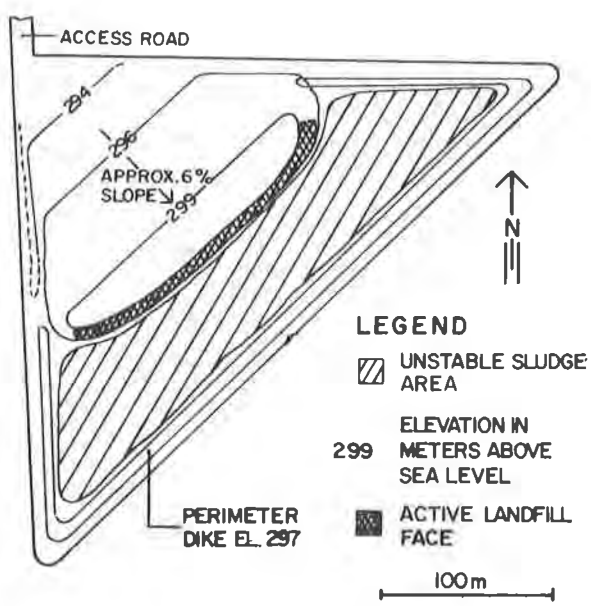

Fig. 1 Sludge landfill

face of slopes and increase stability for construction over marginal ground. Internal cell dike construction can take advantage of near vertical walls such that construction over in-place waste materials can be avoided. Where construction is required over waste materials, the utilization of geotextiles as a reinforcing element within the embankment can be used to reduce differential movement.

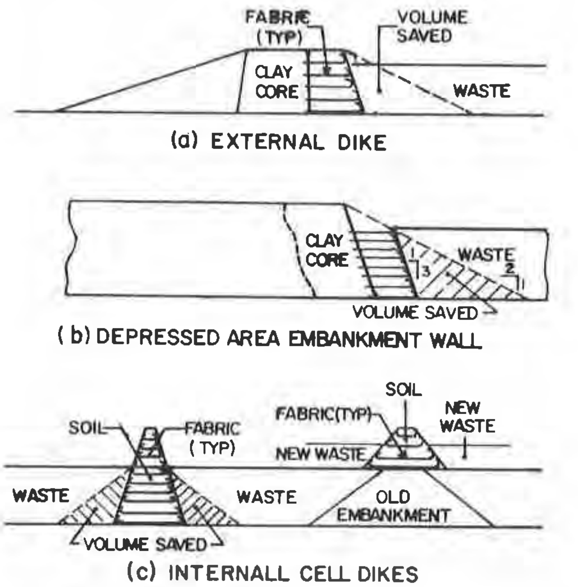

Fig. 2 Fabric reinforced dike systems for impoundment design

conventional construction of the interior face. This design would have decreased the operating life of the impoundment by several years due to the volume occupied by the embankment and possibly have precluded the use of this area for the impoundment. Property boundaries prohibited other alternative solutions. A fabric reinforced dike system was designed to replace the conventional embankment design. The approved design fabric reinforced wall is shown in Figure 3.

For design of the wall, advantage was taken of the lateral stability of the sludge by constructing the embankment in three 5 to 6 meter stages. Each stage was to be constructed after sludge had been placed within 1 m below the top of the embankment from the previous stage. In this manner, up to a 20 meter high wall could be constructed at a 1 horizontal to 3 vertical slope.

Several methods from the previous referenced fabric reinforced embankment design procedures were utilized for evaluating the stability of the wall. The geotextile to be used in the design was selected on the basis of a minimum strength and modulus requirement assumed for the stability analysis and evaluated using wide-width tensile test methods. The chemical constituents of the paint sludge were evaluated as to their potential aggressiveness on the fabric. Several alternative fabrics were selected and, as of this writing, were undergoing chemical and biological resistance testing. In addition, direct shear and pull-out tests will be performed on the soil fabric system once the borrow material has been selected to evaluate the coefficient of friction between the fabric and the soil. A conservative value of 2/3 the estimated friction angle for the soil was used for the design.

Numerous design methods are available in the literature for design of fabric reinforced embankments (1) (3) (6) (8). These procedures generally present classical approaches to slope stability problems and are not unique. Analysis of stress value for the reinforced soil mass generally consists of either a rigid block analysis, equivalent homogeneous soil model, or tie-back action. In addition to the reinforced soil mass, the overall stability of the system must be evaluated, especially when constructed over a marginal soil area. Since the interior walls of the embankment may be completely covered by the waste materials, a less conservative design approach from those indicated in the previous references may be considered. Increased stability may be realized as the impoundment area is filled, therefore, long-term design may not be as critical as in conventional systems. Use of cohesive soils which have not previously been considered in design of retained walls due to creep potential may be feasible due to the stabilization effect.

The geotextile properties required for fabric reinforcement and retaining structures are well covered by Bell and Hicks, 1980 (1). Additional considerations include chemical and biological aggressiveness of the waste materials on the fabric. Ultraviolet exposure, which usually deteriorate the geotextiles, may only occur over a short period of time and as such, coating of the fabric for ultraviolet protection may be reduced.

A dike similar to systems A and B in Figure 2 was recommended to increase the design slope of an industrial paint sludge disposal area in southern Michigan. The impoundment area was an abandoned gravel pit with the new impoundment area to be located adjacent to an existing system. A 300 by 100 meter dike was to be constructed around the perimeter of the containment area. Because of a clay core construction requirement and required height of embankment (15 to 20 meters), a 3 horizontal to 1 vertical slope was required for

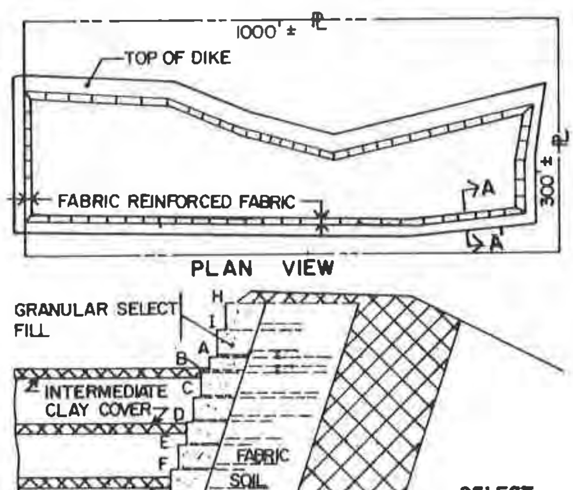

Fig. 3 Fabric reinforced dike design

Construction of the first stage of the embankment is to begin in March of 1982. An embankment monitoring system consisting of inclinometers and a fabric strain monitoring device will be utilized to monitor the wall during and after construction. Due to the stage method of construction, the entire wall will be completed over a 10 year period.

LEACHATE DRAINAGE BEDS

One component critical to landfill management is a media for collection and transport of leachate to treatment facilities. This generally takes the form of a trench(s) or bed at the base of the landfill typically composed of some combination of drainage aggregate and perforated pipes. In a related matter, when an aquiclude or aquitard is present at shallow depths below a landfill, it may be possible to cut off lateral ground water flow with a slurry trench or clay cutoff wall. In this case, it may be possible to use a ground water collection facility in lieu of or in combination with a leachate drainage bed.

Recent and developing legislation shows signs of increasing responsibility of leachate management to the owner for an extended period of time after landfill closure. It is increasingly necessary to enhance long-term performance of these facilities. One concern, particularly with wet, unstable waste, is intrusion of the waste into the drainage aggregate. This can impair, or in several cases, obstruct leachate or ground water collection.

Geotextiles can be utilized to enhance drainage performance by reducing the potential for waste intrusion. This is accomplished by either partial or complete encapsulation of the aggregate with the fabric. The fabric must be designed and selected to be compatible with the materials retained and those restrained. Significant considerations include the particle size of the restrained material (waste or soil), the permeability of the fabric compared to that of the waste or the soil and aggregate, and clogging potential of the fabric. These can be evaluated by standard testing procedures and geotextile-sludge system filtration and clogging model studies such as the gradient ratio method (4). Also of concern are characteristics of the fluid such as its aggressiveness towards the fabric, and the susceptibility of the fabric to biological attack when in direct contact with the waste. This latter criteria can only be evaluated presently with specialized testing specific for each project. Additional research is recommended for these applications. A summary of geotextile drainage design methods and considerations have been presented by Rankilor, 1981, and Bell, 1980 (7) (1).

In the authors' experience, it is recommended that drainage beds and trenches always be constructed by partial or full encapsulation of the aggregate. Not only does this procedure separate the restrained materials from the drainage material, but may also allow for backflushing of the system if clogging should occur. Alternate procedures which have consisted of fabric encapsulated pipe surrounded by aggregates seem less desirable. The potential for waste or soil intrusion into the aggregate remains and the possibility of successful backflushing of a clogged system is reduced.

A ground water collection trench using a geotextile encapsulated system was incorporated in the design of a landfill in Central Wisconsin. Prior landfilling at the site had resulted in ground water degradation below the site and the leachate fluid was detected moving laterally towards the bordering river. In conjunction with the landfill expansion, a bentonite slurry trench was excavated around the site perimeter to cut off lateral migration of ground water flow. At this site, a leachate drainage media was placed at the base of the landfill. However, since the prior landfill was not lined, and since some leachate was expected to pass the primary collection media, the ground water collection system was installed. The site stratigraphy consisted of approximately 7 m of washed sand alluvium over weathered bedrock of a clayey consistency. The system was installed at the sand/weathered rock interface where the highest contaminant concentration was observed, apparently due to a leachate density gradient.

The system consisted of a 200 mm nominal diameter perforated PVC pipe surrounded by 230 mm to 300 mm of 10 mm to 20 mm aggregate. This was further encapsulated in the non-woven heat bonded fabric. In this application, the non-woven fabric was suitable due to the low clogging potential associated with the washed sands and adequate permeability properties. The system has been functioning satisfactorily for approximately 2 years.

INTERMEDIATE DRAINAGE LAYERS

As noted previously, one landfilling method currently employed involves impounding waste in a non-stable condition and then surcharging the mass to increase stability. As with any consolidation problem, one key to improving stability has been to incorporate internal drainage. Our experience with early landfills indicated that sludge lifts in excess of 5 to 7 m resulted in near uncontrollable conditions. It is now recognized that lifts must be limited to 5 m, with overlying drainage layers, to successfully layer wet sludge. Geotextiles have been utilized to aid in placement and support of drainage layers. In many ways, this is identical to the haul road application. However, because the fabric can be secured at the perimeter dikes in this application, the strength properties of the fabric can be more fully utilized.

The authors' experience is presently limited to using geotextiles to support and maintain a drainage layer consisting of .3 to .6 m of sand due to the ready availability and relative economy of sand near all of the landfill sites. In less fortunate areas, geotextiles may have even more significant roles. For instance, two layers of fabric could be utilized to sandwich a relatively thin (10 to 20 mm) layer of sand to form a drainge layer. Alternately, a thick fabric could be utilized independently to provide drainage in the plane of the fabric. Combinations of fabrics bonded to compression-resistant open mesh polymer matting could also be considered. These latter two approaches appear useful for vertical as well as primarily horizontal drainage applications.

Primary fabric selection criteria includes porosity and permeability of the fabric relative to gas and fluid infiltration, the equivalent opening size relative to particle size distribution of adjacent materials, percent open area, and fabric strength. Procedures are available in the literature for determining factors of strength for embankment construction in soft ground and are useful for estimating allowable loadings on the fabrics in landfill locations (2) (5) (9).

The landfill site discussed in conjunction with the haul road considerations (see Figure 1) is also useful in illustrating the use of drainage layers. Initial filling began in 1976 starting at the northwest apex. The landfill was to be filled using a progressive ramp technique, sloping upwards from the apex of the 6% slope with sideslopes dressed at 3 horizontal to 1 vertical. Haul roads were constructed over the sludge to allow the sludge trucks access to the active filling phase. The landfill was operated successfully in this manner when the waste stream consisted only of primarily treatment residue.

After approximately one year, secondary treatment residue was introduced in the waste stream. The combined sludge had a consistency of approximately 20% solids. Introduction of the combined sludge resulted in an instability of the active landfill face and sloughing of the sludge pile towards the perimeter dike, particularly to the southeast. To permit continued landfilling, the perimeter dike was raised and additional sludge was added to the active face. Ultimately, this resulted in an uncontrolled sludge section, approximately 60 meters by 370 meters in plan dimension, and 6 meters in depth. The sludge was in an extremely unstable condition, and unable to support even personnel access. The wet sludge areas are indicated in Figure 1.

To provide area for additional waste deposition, an expansion was constructed to the east of the landfill. The filling plan, however, required that additional waste material ultimately be placed over the wet area. Thus, stabilization of this area was necessary.

Stabilization was accomplished using a geotextile overlay which was in turn covered by 0.6 m of sand. The sand was required to increase the effective stress on the sludge, thus promoting pore water release and consolidation, to act as a media for removal of pore water expelled to the surface, and as an underdrainage layer for subsequent lifts of sludge. The geotextile served to confine the sludge under the weight of the sand and separate these materials.

A woven monofilament polypropylene geotextile was selected. The particular fabric was selected on the basis of its apparent opening size, filtration characteristics, compatibility with the sludge and selected sand, and most importantly, its strength and elongation characteristics. Due to the anticipated variability of sludge consistency, large differential settlements were anticipated. The selected fabric was therefore required to have relatively high tensile strength and modulus which would aid in distributing the load and reduce differential movements. The strength properties required for the fabric were calculated on the basis of anticipated loading during construction and the maximum anticipated differential movement.

The geotextile was manufactured in 1.8 m wide strips and to lengths as required. Four adjacent panels were sewn together at the place of manufacture to form panels of 7.2 m widths which were used on the project. Panels were unrolled over the sludge beginning at the southwest corner. The panels were unrolled in a direction perpendicular to the southeastern dike. The geotextile was anchored at the southeastern dike in soil and was extended to the opposing dike in stable sludge areas where it was again anchored. Anchorage requirements were calculated on the basis of the load imposed on the anchor during construction, estimated frictional characteristics of the soil-fabric interface, and required length of fabric and overburden to resist pull-out (8).

All adjacent panels were field stitched using a hand-held electric sewing machine. The panels were double lapped and double stitched using a monofilament nylon thread. Workmen were generally able to walk on the geotextile to complete the stitching. However, in certain areas, planking was required on top of the fabric to provide access.

Sand placement began at the southeastern dike and proceeded toward the northwestern or active face. In this manner, the mud waves ahead of the sand placement were pushed toward the active face where it ultimately blended in, forming a gradual slope toward the perimeter. Sand placement was accomplished using a "clam" bucket from the dike to place the sand in the desired position. Soil spreading and shaping was done with a light weight dozer. During construction, a separation problem was noted with a few of the factory seams which was found to be due to inadequate stitching. This resulted in localized failure of the sand layer. A satisfactory remedial measure consisted of placing an oversized piece of fabric over the problem zone and again covering that area with sand. This failure indicated the importance of quality control in the textile plant and field observation during placement of the fabric.

The design procedure has resulted in a stable cover. The surface has settled approximately 0.6 to 1.0 m since cover was placed. No additional waste will be deposited in the area for approximately 1 to 2 years. Sludge consolidation during this period should result in a stronger mass capable of supporting the new load.

OBSERVATION WELLS

Landfill designs typically call for installation of observation wells to monitor the impact of the landfill on ground water systems, both in terms of water quality and water gradients. These wells may be required to perform for a significant number of years following abandonment of the impoundment area. A well design that has been found to be dependable consists of PVC pipe with a perforated or slotted end section wrapped with a properly selected geotextile. Selection criteria for the fabric are similar to any drainage application (1) (7). The opening characteristics, both apparent opening size and percent open area, of the fabric must be selected to be compatible with the particle size of the pertinent soil strata. Permeability properties of the fabric should be compatible with the intended use of the well. One other consideration is the method of fastening the fabric to the slotted pipe. Failures have been observed where fasteners came off during installation, or have deteriorated with time. Plastic fastening straps have been found to be very effective. One important aspect of observation well design is that the fabric selection criteria will on occasion be contradictory, particularly for multiple use wells.

If the particular well is to be used both for water quality observations and possibly bail down tests, the designer may be faced with the choice of selecting a fabric with small opening characteristics to limit the quantity of soil fines entering the well, or selecting a fabric with large openings and high porosity to preclude erroneous bail down test results. In early stages of landfill siting, an additional complication is that the selection may have to be made without extensive knowledge of the soil stratigraphy.

It is the authors' experience that fabric selection for observation wells is often casually made. Although there may be no singular (correct) selection, the user should be aware of the potential problems and tailor the geotextile selection as best suits the primary intended use of the well.

FINAL CONTAINMENT COVER

The role of geotextiles in final cover placement on unstable landfills is similar to placement of the intermediate drainage layers. However, higher emphasis must be placed on fabric strength. With intermediate covers, irregular surface settlements may temporarily decrease the effectiveness of the drainage layer, but in time will be covered and regain usefulness as additional materials are placed above. With the final cover, however, excessive differential settlements likely cannot be accommodated, as these will result in permanent surface depressions where surface waters can collect, stagnate, and possibly ultimately increase leachate volumes. Therefore, the fabric selection should be based on the strength and modulus required to distribute loads if differential settlements occur.

One additional concern in design of the final cover is that the geotextile will likely be in the non-saturated zone. It is the authors' opinion that the possibility of a misguided formation (e.g. mud) on the fabric or biological growth could impede release of decomposition gases. It is recommended that the potential for fabric blockage by these methods be evaluated in a testing program prior to final fabric selection. In the extreme, if this phenomena were to occur, the fabric may function similar to a geomembrane.

A project which demonstrates the importance of the strength and gas release functions has been observed in the final abandonment of a paper mill water treatment residue landfill. An abandonment attempt had previously involved placing an unreinforced 20 mil PVC sheet (a geomembrane), directly on top of the waste, with an additional 0.6 m of clay soil to be placed over the membrane. The membrane was to provide support during placement of the clay. Filling operations proceeded from the edge of the fill area. After extending the clay cover only a few meters out over the fill, it was reportedly found that the membrane would not support any equipment available for placement of the clay. A method of placing the clay was finally established where sheets of plywood and straw matting were placed over previously placed materials, and wheelbarrows were used to transport the clay cover. The entire operation was completed by hand. Shortly after placing the cover, bubbling of the geomembrane through the clay cover in numerous areas was observed. Bursting of these bubbles, which were up to 2 m in diameter, indicated gas entrapment was occurring below the liner.

The abandonment currently proposed involves placement of a woven monofilament polypropylene fabric, overlain by 0.3 m of sand and 0.6 m of clay. The fabric was selected on the basis of strength, modulus, and opening characteristics. Potential for gas entrapment due to meniscus formation in the pores was evaluated and found to be negligible. Thus, the fabric should allow passage of gases through the sand layer which will serve as a media for gas collection and venting.

CONCLUSIONS

The examples in this paper demonstrate practical uses for a relatively new and valuable engineering tool in waste management. The utilization of geotextiles was shown to complement the primary objectives of the design of impoundment systems including improving stability, enhancing and facilitating release and collection of pore water and gases, and maximizing disposal volume. In many instances, existing design techniques available in the literature can be utilized for the design. However, unique characteristics of the waste materials must be considered and designs modified accordingly. These characteristics include low inherent stability, high moisture content, large organic fraction which results in large deformations, and production of gases.

Of utmost importance is the possibility of chemical and biological effects on the properties of the geotextile required for the specific design function. Designed and selected correctly, geotextiles may be used to solve problems, reduce cost, and increase utilization of materials.

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Instrumented Case Histories of Fabric Reinforced Embankments over Peat Deposits

Expérience pratique de la construction de talus à renfort géotextile sur des dépôts de tourbe

Two Sections of road embankments, built on soft organic deposits in Ontario, Canada were constructed by using various geotextiles for separation and reinforcement.

At the Manchester site four different fabrics were placed longitudinally at the interface of the organic peat and the base of the fill for separation. At the Bloomington site fabrics were laid in the transverse direction, partially encapsulating a layer of the embankment.

The effectiveness of the fabrics at the Manchester site was evaluated by visual examination of the geotextiles in test pits, excavated one year after installation.

At the Bloomington road the principal function of the geotextiles was reinforcement by membrane support. The stage construction of the embankment was successfully carried out by monitoring of the vertical and lateral deformations and the rate of dissipation of excess pore pressure.

Divers géotextiles ont été utilisés aux fins de séparation et de renforcement dans la construction de deux sections de talus de remblai sur des dépôts de sol organique mou, en Ontario, Canada. On a posé quatre textiles différents dans le sens longitudinal pour séparer la tourbe organique et la base du remblai au chantier de Manchester. A celui de Bloomington les textiles ont été posés dans le sens transversal, renfermant partiellement une couche du talus.

A Manchester, l'efficacité des géotextiles a été évaluée au moyen d'examen visuel dans des puits d'essai, creusés une année après l'installation.

A Bloomington les géotextiles avaient la fonction principale de renforcement par support de membrane. La construction étagée du talus a été effectuée en surveillant les déformations verticales et latérales et le taux de dissipation de la pression interstitiale excédentaire.

INTRODUCTION

In constructing road embankments on highly compressible and very soft organic deposits road engineers must consider the possibility of embankment failure by the induced shear forces and the occurrence of excessive settlements. The intrusion of organic material and mixing of the soft soil with the fill will result in further aggravation of embankment stability. This paper describes two case histories where geotextiles were employed for construction of road embankments on deep organic deposits. The first, Manchester Site, describes the construction of a temporary detour using geotextiles as separators and the second, Bloomington Site, outlines the use of geotextiles as reinforcing membranes. Both sites are located near Toronto, Ontario, Canada.

MANCHESTER SITE

Highway 12 is a major two lane artery connecting Toronto and the resort area of Lake Simcoe. A detour, required to permit a bridge construction along the highway near Manchester, provided an opportunity to assess the role of geotextiles in low embankment construction over swampy lowlands. Four geotextiles, including wovens and nonwovens, were placed below the north 200 m long bridge approach fill. The south approach, over shallow peat, was constructed without fabrics. Embankment settlements and geotextile elongations were monitored over the 6 month active life of the detour.

Site Description

The detour location was relatively level but hummocky due to the cover of long reeds and 1 to 2 m tall woody shrubs. Water was at or above ground surface throughout this area. Amorphous granular peat blankets the site, underlain by sands and soft to stiff clay. South of the river, where the detour was built without geotextiles, peat depths range from 0.3-0.5 m. North of the river, the four geotextile trial sections were constructed over peat, ranging in depth from 1.5 m to 4 m (Figure 1). Physical properties of the peat are summarized in Table 1.

TABLE 1 - Properties of Peat at Manchester Site

PROPERTIES	RANGE OF VALUES
Unit weight, γ (kN/m^3)	9.7 - 11.7
Water Content, w (%)	170 - 616
Organic Content, (%)	30.4 - 82.8
Compression Index, C_c	2.4 - 8.7
Shear Strength, τ_f (kPa)	7.1 - 16.6

Design/Construction

The detour consisted of a sandy silt embankment overlain by gravelly sand and crushed stone subbase and

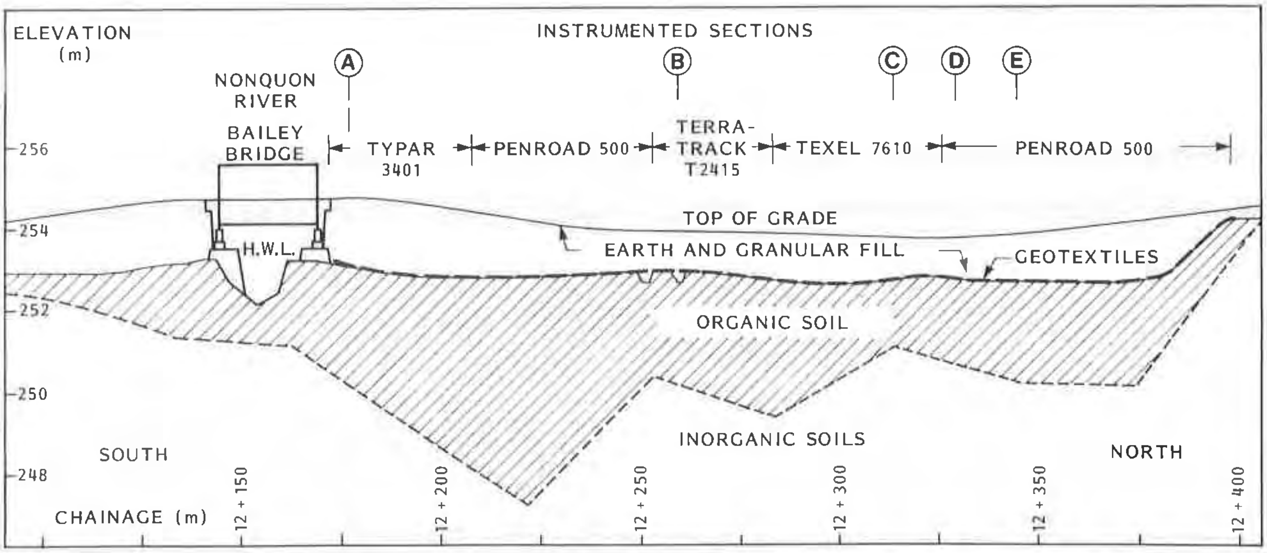


FIGURE 1 - Centre Line Profile Manchester Detour

base courses (Figure 2). Four geotextiles having similar tensile strengths were selected for trial use as separators between the embankment and the underlying peat. Properties of these fabrics are given in Table 2.

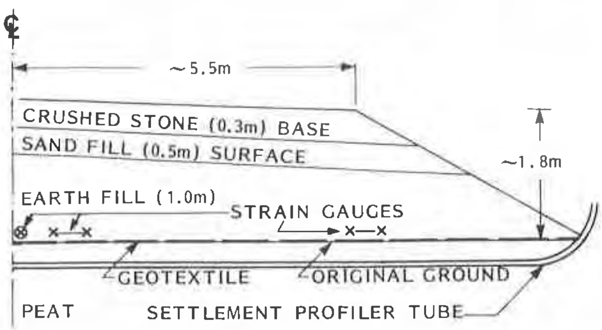


FIGURE 2 - Manchester Detour Cross Sections showing construction and instrumentation details

TABLE 2 - Geotextile Properties (2) Unaged Fabrics

PROPERTIES	PENROAD 500 Woven	TERRATRACK T-2415 Woven	TEXEL 7609 Nonwoven	TYPAR 3401 Nonwoven
Mass, μ (g/m^2)	150	120	280	136
Thickness, T_g (mm)	0.5	0.45	3.0	0.38
Breaking Force, F_G (N) Grab Test (warp)	800	485	480	600
Elongation at Rupture, E(%)	22	16	90	62
Puncture Strength, *F_p (N)	1000	890	1160	750
Equiv. Opening Size, µm (US Standard Sieve)	75 (#200)	300 (#50)	150 (#100)	180 (#80)

*CGSB STANDARD 4-GP-2 11-2 (55)

The location in which each product was used is shown on Figure 1. Stability of the 1.8 m high unpaved detour embankment was not considered to be a problem due to the limited thickness of the peat and the low fill height. Settlement calculations indicated that 0.2 to 0.7 m of vertical deformation should be expected. This agreed quite closely with empirical relationships recently published, between fill height, peat thickness and settlement (1). As a rule of thumb, additional fill, amounting to half the height of the embankment (0.9 m), was provided by the construction forces to maintain final design grades after settlements. After close cutting the woody shrubs to leave stumps of less than 0.15 m height, conventional earth moving equipment was used to construct the detour. Two men in tall rubber boots hand placed the geotextiles over the peat with 1.0 m overlap of the adjacent panels, then earth fill was placed directly on the geotextiles.

Instrumentation

Five sections of the detour were instrumented to monitor embankment settlement and geotextile elongation as shown on Figure 2. Settlements were measured by pulling an electronic pressure transducer through a water filled profiling tube, placed under the geotextile at each instrumented section. Geotextile elongations were measured by gauges resembling miniature hydraulic pistons. The piston body was clamped to the geotextile at a point 200 mm distant from the actuating point for the piston arm. Elongation in the geotextile caused withdrawal of the piston and resulted in lowering of the water level in a sight tube remote from the embankment but connected to the piston body. Three of these strain gauges were installed at each of the instrumented sections. (A,B,C,D and E in Figure 1).

Installation of Geotextile and Observation of Instrumentations

The high grass and reed cover and the roughly 0.3-0.4 m deep water, covering parts of the site made the geotextile installation rather difficult. The Texel fabric, a needle punched polyester, having relative densities higher than water and quick water absorption, became very heavy and unwieldly in open water installation. (3).

The heavy construction equipment readily rutted the earth fill over the fabrics, prestressing the geotextiles.

Settlement monitoring indicated that most of the vertical deformation occurred very soon after fill placement, as expected. Settlements some 140 days after construction are shown on Figure 3. Maximum settlements at each section ranged from 0.2 to 1.0 m.

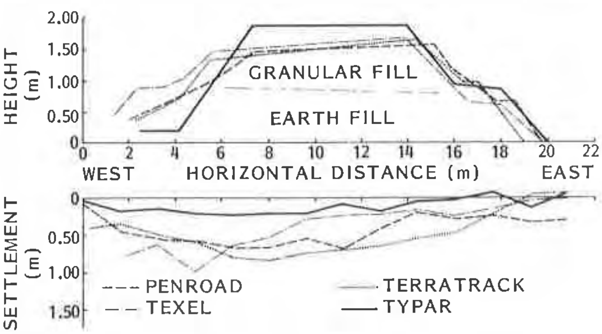


FIGURE 3 - Manchester Embankment Cross Sections, showing fill height and settlements

The elongation of the fabrics during and after construction was monitored by strain gauges as mentioned earlier. In Figure 4 typical values of elongation versus time are plotted. The maximum observed elongation was somewhat less than 10% in both the longitudinal and the transverse directions. The strain gauges registered a gradual decrease in elongation during the months following construction.

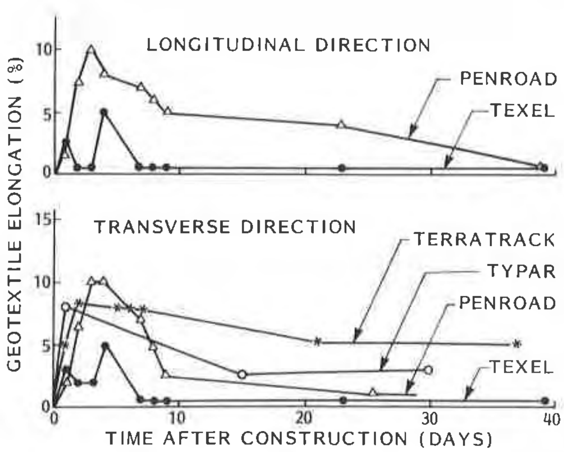


FIGURE 4 - Typical Geotextile Elongations at the Manchester Site

Discussion

Geotextiles used under the detour embankment provided superior separation between the fill material and the underlying organic soil. The fabrics were also very useful in bridging local depressions and soft spots.

Trenches excavated through the fill and down to the organic deposit one year after construction clearly showed that the geotextiles produced a sharp, smooth line of separation, with absolutely no subgrade intrusion and no mixing of the two soils (Figure 5). Similar trenches through the fill with no fabrics revealed an irregular fill-peat interface with layers of intermixed soils of some 0.15-0.2 m thickness (Figure 6).

Fabric survivability and field workability must be assessed prior to any installation as suggested by Haliburton (4). On the basis of subgrade strength and vegetation cover the field workability criterion was not considered to be critical. Considering the construction equipment, the prerutting of the fill over the fabrics and soil conditions, moderate to high survivability requirements were specified. Strength parameters of all the installed geotextiles met these requirements.

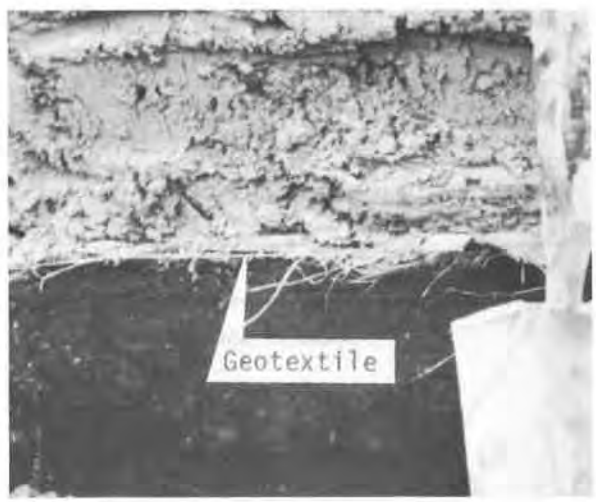


FIGURE 5 - Fill, Peat interface with Geotextile

FIGURE 6 - Fill, Peat interface without Geotextiles

BLOOMINGTON SIDE ROAD

Bloomington Side road, a two lane highway corridor is located some 16 km north of the city of Toronto. The construction started in early 1981, and at the time of writing this report, part of the 2 km road is still under construction. In the vicinity of the road there are numerous undrained organic deposits. The road traverses one of these deposits, having a maximum thickness of 7.6 m and a length of 300 m. Construction of the approximately 1.5 m high road embankment across the organic deposit utilized geotextiles for separation and reinforcement.

Soil Conditions

A thorough field investigation, consisting of 11 sampled boreholes, was carried out at the site of the organic deposit, prior to design. The soil stratigraphy revealed by the borings is shown on the soil profile in Figure 7. The organic deposit was identified to be a highly organic peat (Pt), extending to maximum depths of 6 m and 7.6 m in the west and east of the swamp with considerably shallower peat in the middle. The consistency of the peat was very soft Properties of the peat are recorded in Table 3. The groundwater level was either 0.2-0.3 m above or at the general ground surface. Underlying the peat an inorganic layer of silt and sand (SP & ML) was found, having compact density.

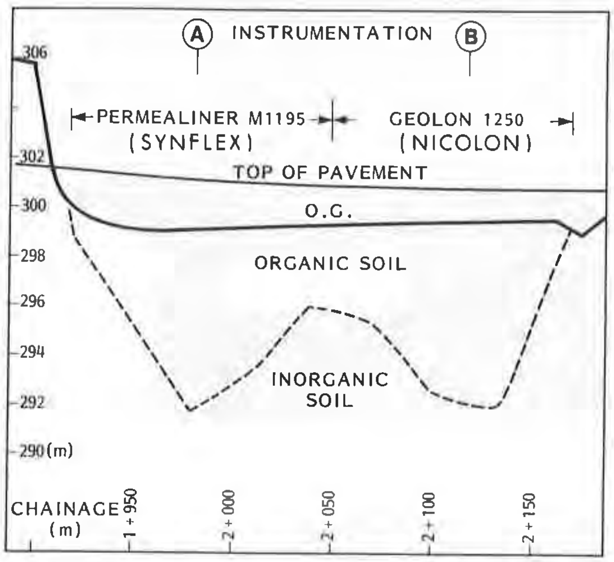


FIGURE 7 - Centre Line Profile Bloomington Side Road

TABLE 3 - Properties of Peat at Bloomington Site

PROPERTIES	RANGE	AVERAGE
Unit Weight, γ (kN/m^3)	9.2 - 11.5	10.0
Water Content, w (%)	291 - 1041	595
Organic Content, (%)	17 - 80	53
Liquid Limit, w_L (%)	216 - 324	275
Plastic Limit, w_p (%)	138 - 255	690
Plasticity Index, I_p (%)	20 - 78	46
Intial Void Ratio, e	7.0 - 18.6	12.6
Compression Index, C_c	3.9 - 10.1	5.6
Shear Strength, τ_f (kPa)	6 - 38	18
Sensitivity, S_t	1 - 3	

Design Considerations

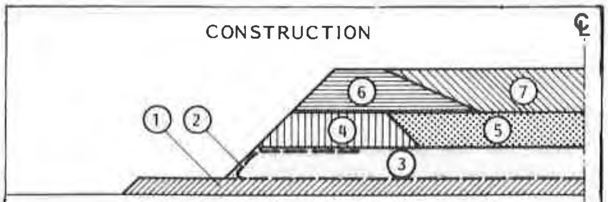
In designing the 1.5 m road embankment over the organic terrain, total or partial excavation of the peat and replacement with inorganic soils was considered. On account of the depth of the deposit and the sloping swamp bottom these methods were found uneconomical. Since a full year was available for the completion of the road it was decided to build the fill in stages. The first stage construction called for "floating" the embankment on the peat surface. It was estimated that the dissipation of excess pore pressures under the fill would take about three months. After this period the embankment could be reconstructed, with an additional surcharge load. Construction of the base, subbase and asphalt pavement courses would follow after another six months of consolidation. Total stress stability analyses of the fill were carried out using remoulded shear strengths of the peat, as suggested by some authors (5). The analyses indicated that shear failures along circular arcs may occur under the designed 1.5 m high fill. The use of berms to minimize the possibility of such failures was not possible at this site due to the width of right of way available. It was therefore decided to place fabrics under the embankment in order to prevent slip failures. Accordingly it was proposed to install woven geotextiles having high tensile strengths and high moduli. A twisted polypropylene, anisotropic, slit film Nicolon woven product was chosen for half the length, for the other half a monofilament polypropylene woven fabric manufactured by Synflex was used. Properties of the geotextiles are listed in Table 4.

TABLE 4 - Geotextile Properties (2) Unaged Fabrics

PROPERTIES	GEOLON 1250	PERMEALINER 1195
Mass, μ (g/m^2)	730	225
Thickness, T_g (mm)	2.26	0.4
Breaking Force, F_G (kN) Grab Test (warp)	5.56	1.78
Elongation at Rupture, E (%)	18	30
Mullen Bursting Pres., (MPa)	11.0	3.5
Equivalent Opening Size,(μm) (US Standard Sieve)	425 (#40)	212 (#70)

Sequence of Construction and Instrumentation

The first stage of embankment construction across the swamp took place in the spring of 1981. After clearing the site a working platform, consisting of a 0.3 m layer of silty sand, was placed on the ground to eliminate placing the geotextiles under water. The fabrics were laid transverse to the road alignment, overlapping the adjacent strips by 0.7 m. A toe to centreline sequence of fill construction was carried out, modified somewhat from the method described by Haliburton (4). Secure achorage was accomplished by folding back the 3 m extra lengths of the fabrics over the first lift of fill, and placing a second lift over the fabric near the toes. These toe fills were compacted prior to filling the middle portion of the embankment. The sequence of construction is depicted in Figure 8.



STAGE I

1. Place working platform
2. Place Geotextile transverse to alignment
3. Place 0.3 m granular and fold back Geotextile
4. Place and compact earth to anchor Geotextile
5. Place and compact embankment core

STAGE II

6. Place and compact earth to profile grade at edges
7. Place and compact earth to profile grade at core

STAGE III

Remove preload and place pavement structure

FIGURE 8 - Construction Sequence

In order to control stage construction and to observe fabric performance some field instruments were installed at both of the deep peat locations. A sketch of the instrumentation is presented on the cross-section of Figure 9. The excess pore water pressure and the rate of dissipation was observed by Geonor type brass piezometres installed beneath the centreline and beneath the shoulder of the fill. Vertical movements were measured by settlement profilers, as previously described. Elongation in the geotextiles was monitored both in the longitudinal and transverse direction at the centreline and near the slopes. Two independent sets of gauges were installed, one operated hydraulically the other by electric current. The former was already described, the latter

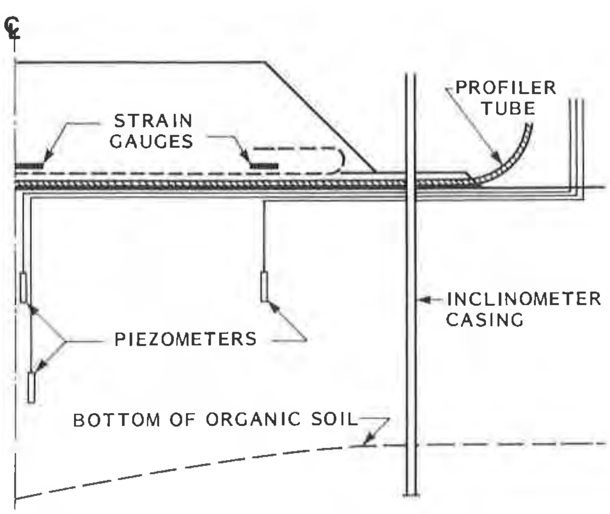


FIGURE 9 - Sketch of Instrumentation Locations

consists of a set of wire coils encased in plastic, affixed to the fabric some 200 mm apart. The distance between the coils is determined by running an electric current through one and measuring the induced current in the other. Lateral displacements were determined by inclinometers, installed vertically at both sides of the fill adjacent to the toes.

Discussion and Results

The first stage of construction was carried out successfully without any sign of prospective rotational failure. It is postulated that this type of failure was prevented by the membrane effect of the geotextiles.

In the middle 80 m portion of the peat the depth of the deposit was less than 3 m. Settlements predicted along this section were in the order of 0.3 m. Actual settlements observed during and immediately after the first stage construction were about 0.4 m. Floating of the embankment as planned, within this section, was thus achieved.

Under section A and B (See Figure 7), where the depth of the organic deposit was considerably deeper than in the middle, much larger settlements were observed than predicted. The magnitude of the vertical deformation was 94% of the total height of the fill. The corresponding excess pore pressure beneath the centre of the embankment was measured to be up to 90% of the applied effective stresses. The excess displacement was due to lateral shear deformation of the peat under undrained conditions. This fact was confirmed by movements of inclinometers indicating an outward lateral force located below mid-depth of the deposit. Field vane tests carried out 60 days after construction resulted in no appreciable increase in the undrained shear strength of the peat, substantiating the above hypothesis.

During the first stage of construction the geotextiles were subjected to elongations of 2% to 5% as measured by the strain gauges. Elongations of similar magnitudes were observed in both the transverse and longitudinal directions, as at the Manchester Site. Strains induced in the longitudinal direction imply that plane strain conditions may not fully apply to low embankments constructed on highly compressible organic materials on account of surficial variations (soft areas, tree stumps, etc.). All the strain gauges registered a decrease in strain with time, as also observed at the Manchester Site. While it is not suggested that a decrease in observed strain corresponds to a decrease of fabric elongation, the observed phenomenon might have been caused by stress re-orientation due to continuing deformation.

During the planned three month waiting period following the first construction stage excess pore pressures dissipated and the rate of settlement considerably slowed down so that construction of the second stage was deemed feasible. The geometry of Stage II construction with the corresponding settlement profile is shown in Figure 10.

Most of the additional settlements occured during construction of the second stage. In the four months following construction, settlements of 0.1 m only were measured.

The piezometers registered excess pore pressures equal to 40% of the newly applied stresses. Elongations of the geotextiles in the transverse direction approached the failure elongation at some locations. The maximum longitudinal elongation was observed to be 8% beneath the centre of the fill.

Some mud waves with tension cracks developed near the toe of the embankment, on account of the additional load confirming lateral shear deformation.

No tension cracking or horizontal deformation was observed within the body of the embankment demonstrating the ability of the geotextile to contain and restrain the embankment thus preventing lateral spreading.

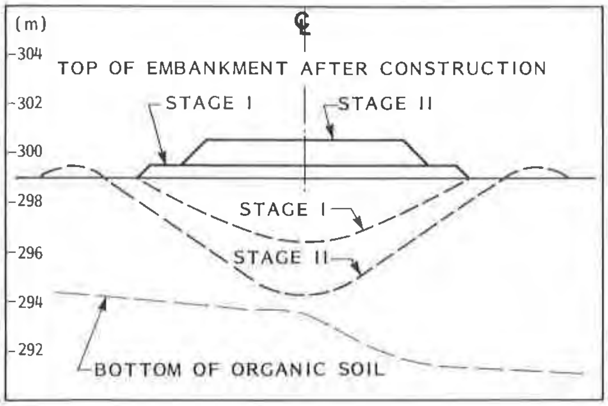


FIGURE 10 - Sketch showing deformation of Original Ground immediately after Stage I and Stage II construction

CONCLUSIONS

From the field studies reported in this paper several useful and practical lessons can be learned and conclusions drawn. These are summarized as follows.

(a) Regardless of the differences in the base polymers of the fibers and in fabric construction, all the geotextiles used at the Manchester fill performed the separation function well. The actual construction experience appears to confirm Haliburton's finding that if a fabric survives the abuse during placement and construction it will provide adequate separation, provided that the opening size and permeability of the textile is within acceptable limits. If drainage along the plane of the fabric is not required then thin woven or bonded nonwoven fabrics will be more suitable than thick needle punched ones, based on economics and the ease of placement. Fabrics manufactured of polypropylene are easier to handle in open water installation than polyester and nylon products, since the relative density (specific gravity) of the former is less than one, with hardly any water absorption.

Except one woven polypropylene fabric, all the geotextiles recovered one year after construction lost some 25% to 36% of their original tensile strengths. The loss of strength did not seem to have any adverse effect on their performance.

(b) While it is not an engineering necessity, building of a thin non-cohesive soil working platform on very soft, rough and uneven swamp surface will facilitate placement and prestressing of the fabric. The additional construction expenditure will be offset by the much speedier placement, and the more even distribution of loads on the fabric.

(c) For the Bloomington site stability analysis in terms of total stresses was carried out prior to construction, using the proposed height of the embankment of 1.5 m. The resulting safety factor of 1.0 indicated a potential rotational shear failure of the fill without the geotextile. By using the critical slip circle and adding a fabric tensile strength of about 30 kN/m, the resisting moment was increased sufficiently so that a safety factor of 1.2 was obtained. During construction the fabrics were subjected to elongation on account of vertical deformations. On the basis of laboratory stress strain curves it was estimated that under the measured elongation the geotextiles mobilized tensile strengths in the order of 35 kN/m. It is therefore postulated that the membrane effect of the geotextile was fully utilized in preventing rotational failures.

(d) At the Bloomington site considerable vertical deformation occurred by lateral displacement of the peat. Such large lateral movements of the foundation soil could have resulted in embankment failure by horizontal splitting (4). There were no tensile cracks observed within the embankment whereas several were observed outside the geotextiles involving the working platform. It may be concluded that the mobilized tensile strength of the geotextiles and the soil fabric friction assisted in restraining the fill from lateral sliding.

ACKNOWLEDGEMENT

The geotechnical design of the road embankments reported in this paper was caried out by Mr. H.S. Elston under the direction of Mr. R.D. Gunter. The co-operation of the research department of Ontario Hydro and the Ontario Ministry of Transportation and Communications (M.T.C.) in the instrumentation and monitoring program is gratefully acknowledged.

The authors are indepted to Mr. F.G. Allen, Executive Director, Highway Engineering Division of the M.T.C., and Mr. F. Kee, Director of Research of Ontario Hydro for permission to publish this paper.

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Fabrics Support Embankment Construction Over Bay Mud

Construction d'un remblai, supporté par des textiles, sur la vase de la Baie de San Francisco

Recent construction practices by Caltrans have included the use of reinforcing fabrics to solve embankment stability over soft muds at the new Dumbarton Bridge. Special construction procedures and the performance of a test embankment supplied guidance in developing specifications for the main approach embankments. Woven reinforcing fabric was specified based on strength, elongation and handling properties. Construction required embankment placement over open water and bay mud. The only failures that occurred were in the west approach containment dike during initial stages of construction. It is believed that major embankment failures would have developed had fabrics not been used. Stability analysis indicated that fabric added about 10% to the overall factor of safety. Fabric provided initial support and helped maintain embankment integrity during periods of high pore pressures and marginal stability.

Les pratiques chantiérs de Caltrans, ces dernières années, utilisent des textiles pour la stabilisation des remblais sur la vasé sous le pont nouveau de Dumbarton. Methodes spéciales et l'etude d'un remblai d'essa: qualifient le calcul de stabilité pour les remblai d'approchement principales. On mesure la resistance, l'elongation des textiles tissées et leur caractéristiques mécaniques dans la pratique. La construction demande qu'on fait le remblais sur l'eau de la Baie et sur la vasé. Il n'avait pas plus qu'une rupture: dans la digue d'approchement a l'este. On croit que tous le remblai s'aurait rompu sans les textiles. L'analyse du stabilité montre que les textiles augmentent par 10% (approximatif) le facteur de sécurité. Les textiles renforcent les remblais au commencement et les soutient durant temps d'accroissant pressions interstitielles et stabilité marginale.

INTRODUCTION

Embankment construction over soft compressible foundations has generally caused difficulty for Transportation Engineers. The initial bearing capacity of these soils has often been too soft to support embankment loading without considerable deformation. Costly failures can also occur due to inadequate.soil strength gain from slow dissipation of pore pressures during the consolidation process.

This type of construction in the past was a particularly difficult task for the California Department of Transportation (Caltrans) where alignment of transportation facilities traversed over soft muds in the San Francisco Bay area. Failures sometimes developed even with close control of embankment placement and a good program of monitoring foundation pore pressures and settlements.

This paper discusses recent construction practices by Caltrans which included special features that have solved these problems by providing embankment and foundation stability during construction over soft muds.

BACKGROUND

Embankment construction for the Dumbarton Bridge replacement project across the southern portion of San Francisco Bay has provided Caltrans with an opportunity during the last two years to evaluate various features to stabilize soft foundation soils during embankment loading.

The alignment for this facility on Route 84 crosses through several commercial salt ponds which lie over former marsh lands. The average depth of water in the salt ponds for the east approach is about 0.8 m (2-1/2 ft). The water near the west approach is a more difficult crossing with depths up to 4.6 m (15 ft).

The entire area is underlain by San Francisco Bay mud, ranging in thickness from 3.1 to 12.2 m (10 to 40 ft). The mud in some areas has an undrained shear strength of less than 4.79 kPa (100 psf) near the surface with strength increasing to over 14.37 kPa (300 psf) at greater depths.

The embankment was planned for an ultimate height of 3.1 to 3.7 m (10 to 12 ft) with constructed heights up to 4.9 m (16 ft) prior to settlement. The bay mud foundation required special treatment and controlled rates of loading to support these embankment heights.

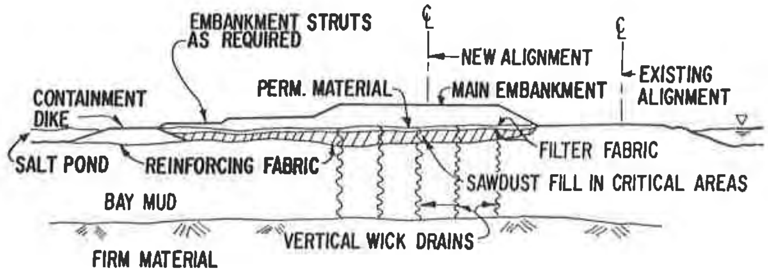

Fig. 1 Cross section showing special construction features used at Dumbarton.

To accomplish the embankment construction within the contract time constraints, several features were incorporated which have had minimal prior usage in the United States. These (Figure 1) include: (1) reinforcing fabric to distribute embankment loading and prevent failure of soft foundation soil during construction and reduce differential settlement, (2) lightweight fill (sawdust) to reduce embankment loading, (3) vertical wick drains to accelerate the consolidation process to less than one year, and (4) filter fabric to prevent contamination of a permeable drain blanket for removal of pore water as part of the consolidation process.

A report on the performance of the above system was presented in 1982 (1).

This paper will emphasize only reinforcing fabric selection, placement procedures and fabric performance at Dumbarton.

FABRIC SELECTION

Stability analysis of critical embankment-foundation sections for design revealed marginal stability for this facility using conventional fill placement. Multiple shear failures and extensive mud waves could result from normal construction techniques. Long term differential settlement and inordinate maintenance requirements could also result. The maximum heights attainable with conventional construction based on previous Caltrans experience was limited to about 2.4 m (8 ft). This is presented by the lower curve in Figure 2. Laboratory studies indicated that conventional consolidation would also require up to 50 years for ultimate settlement of about 1.8 m (6 ft) to occur in the 12.2 m (40 ft) deep bay mud foundation.

Haliburton, et al (2), reported that reinforcing fabric must have three essential properties to provide good performance over soft foundation soil. These properties include: (1) high elastic modulus to prevent elongation of fabric and excessive foundation deformation, (2) high tensile strength to resist embankment failure, and (3) sufficient soil-fabric frictional resistance to prevent lateral movement of embankment slopes.

It was estimated, from computer analyses, that the reinforcing fabric at Dumbarton would

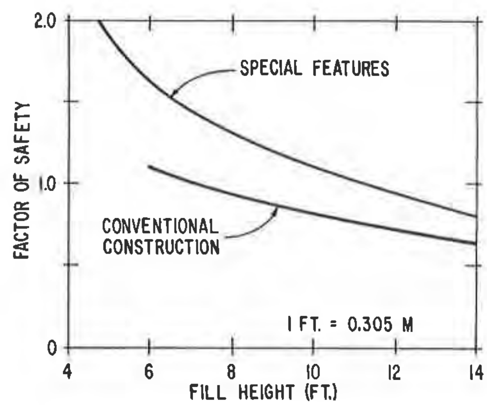

Fig. 2 Factor of safety vs. fill height

add about 10% to the overall stability of the embankment foundation system through additional unit cohesion provided by fabric tensile strength. The upper curve in Figure 2 represents factors of safety with reinforcing fabric in combination with lightweight fill for the instantaneous loading condition.

A 92 m (300 ft) long test section was constructed in 1979 at the east bridge abutment. The test embankment served to evaluate the special features proposed for the main construction and provided information to develop appropriate specifications. The test embankment was constructed to a maximum height of 5 m (16-1/2 ft) with a controlled loading rate of 0.3 m (1 ft) per week to height of 3.1 m (10 ft) and 0.3 m (1 ft) every two weeks thereafter.

Six different reinforcing fabrics were used over the bay mud for the test area. They included both woven and non-woven materials. Table 1 is a listing of these fabrics.

Table 1 Reinforcing fabrics used in test area

Bidim	C-22	Nonwoven
Mirafi	140	Nonwoven
Mirafi	140S	Nonwoven
Mirafi	500X	Woven
Supac	5-P	Nonwoven
Typar	3401	Nonwoven

Extensive instrumentation was installed in the test embankment by Caltrans to monitor settlements and pore pressures during construction loading. Initial construction prior to embankment placement (Figure 3) required the following construction sequence:

1. Placement of reinforcing fabric across 0.8 m (2-1/2 ft) deep open water for support of longitudinal and cross containment dikes. (Reinforcing fabric was placed longitudinal to dike alignment.)

2. End dumping and compaction of earth fill for containment dikes.

3. Dewatering of area contained by dikes and existing roadway embankment and placement of reinforcing fabric on bay mud for main embankment within containment area. The reinforcing fabric in this area was placed transverse to the roadway alignment.

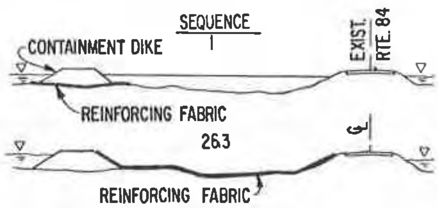

Fig. 3 Construction sequence for placement of reinforcing fabric.

The overall performance of the test embankment was excellent with only minimal settlement cracking. About 2 m (6-1/2 ft) of settlement occurred in the test fill foundation during a one year period.as a result of the wick drain installation.

The excavation for placement of a culvert provided the opportunity to retrieve a sample of the reinforcing fabric after one year of burial under the test embankment. Figure 4 presents a strength comparison of this material to that of new material of the same manufacturer. Note that there is no significant detrimental change in the strength characteristics of the buried fabric.

The reinforcing fabric under the test embankment successfully prevented the development of a mud wave in the soft mud foundation. High pore pressures were recorded during construction which probably would have caused failure if reinforcing fabric had not been used. By contrast, an adjacent staging area constructed by the contractor, by end dumping with no special treatments or controlled rates of loading, produced a large mud wave.

The reinforcing fabrics used under the test embankment exhibited a wide variation in handling characteristics. On the basis of strength, elongation and handling, a permeable woven type reinforcing fabric of either polyester, nylon or polypropylene was specified for the main approach embankments. Table 2 lists the specification requirements.

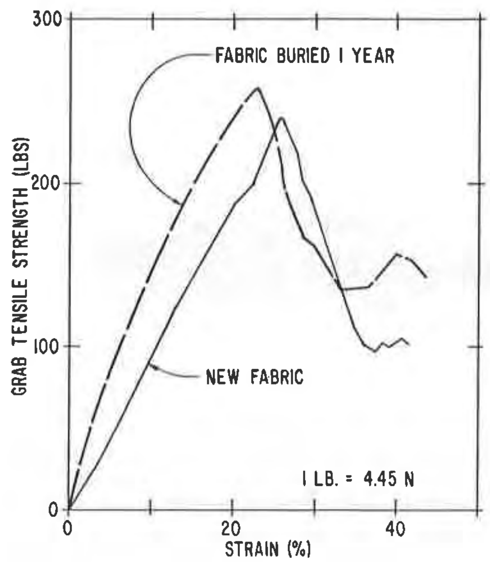

Fig. 4 Strength comparison of fabric buried 1 year.

Table 2 Specification requirements for reinforcing fabric

Characteristic	ASTM Test Designation	Requirement
Grab strength	D1682	890 N (200 lbs) Min.
Elongation to break	D1682	35% Max.
Joint strength	D1682 and D1683	712 N (160 lbs) Min.

The reinforcing fabric would provide support for the main embankment construction and serve as a separator between the bay mud and the embankment. The fabric would also have to be of sufficient strength to support the construction after wick drains were punched through it on 0.92 to 1.83 m (3 to 6 ft) spacings. It was felt that the above test requirements would provide a fabric with the performance desired.

EAST APPROACH EMBANKMENT

The main east embankment construction began August 1980 and progressed in accordance with the techniques developed for the test embankment with modifications as reported by Hannon and Walsh(1). The contractor selected Mirafi-500X, a woven reinforcing fabric supplied by Celanese Corporation.

Photo 1 shows dozer pushing imported fill for the containment dike over reinforcing fabric in 0.8 m (2-1/2± ft) of water. The stitched fabric seam was longitudinal to the dike alignment.

Photo 1 Dike construction in open water

Following dewatering of the subpond, reinforcing fabric was placed with stitched seams transverse to the main embankment.

The fabric was floated on air in large sheets to cover the diked off subponds. Lead corners of the fabric were tied to light trucks which pulled the fabric over the dewatered ponds. One truck operated on the dike and the other from the shoulder of existing Route 84.

Sawdust fill was placed on the fabric (Photo 2) in areas of critical stability to reduce the initial loading on the bay mud and allow embankment progress.

Photo 2 Lightweight fill (sawdust) placement on reinforcing fabric for main embankment.

During the dewatering process prior to placement of reinforcing fabric, it was impractical to completely evacuate all of the fluid muds near the surface. Consequently, much of this material was trapped as mud boils under the fabric. Loading and pushing of the fill materials on the fabric eventually forced the fluid mud out of the fill area, across the dike and into the adjacent salt ponds. During this operation, the fabric was able to contain mud boils in excess of 1 m (3 ft) in thickness as shown in Photo 3.

Photo 3 Soft mud contained by reinforcing fabric.

The embankment was placed with a controlled rate of loading as previously discussed. No failures occurred during construction of the 3.86km (2.4 mile) east approach embankment. However, critical pore pressures were recorded in the bay mud foundation during embankment loading that required waiting periods up to eight weeks. It is believed that failure would have occurred in these areas without reinforcing fabrics using normal construction techniques.

The embankment was completed in January 1982.

WEST APPROACH EMBANKMENT

The construction at the west approach was similar to that employed at the east approach. However, the initial dike construction proved to be more difficult.

The construction began July 1981 and was accomplished by a different contractor, who also selected Mirafi-500X reinforcing fabric.

Several failures occurred during initial construction while placing the containment dike in a marsh area underlain with about 6.1 m (20 ft) of bay mud. Access to the area was convenient to the contractor who elected to use large belly dumps to place the imported fill material. As a result, the foundation soil was loaded too rapidly and failures occurred. This is in contrast to dike placement at the east approach which was accomplished with end dumps and the rate of loading was easier to control.

The dike failures that occurred at the west approach appeared as mud waves on each side of the dike fill (Photo 4). Large cracks also developed along the dike centerline suggesting possible separation at the longitudinal fabric seam (Photo 5).

Photo 4 View showing pushup beyond slope toe of containment dike.

Photo 5 View showing longitudinal crack near centerline of containment dike.

Correction of one failed area for a distance of about 214 m (700 ft) consisted of removing the failed dike embankment down as close as possible, to the failed fabric. This was then followed by placement of a new layer of fabric with sufficient width to overlap the reinforcing fabric to be placed under the main embankment. The embankment for the containment dike was then restored to its planned elevation. Refer to Figure 5.

During initial fill placement for the main embankment, very high pore pressures (equivalent to 60 to 70% of the applied load) were recorded that immediately prompted cessation of loading. This area was very critical because of the previous dike failure. The embankment during this period gave the dynamic action of a large waterbed. This was noted at one location as haul vehicles passed over the main embankment. Approximately 0.15 m (1/2 ft) of instantaneous vertical compression and rebound was observed under truck wheel loading. It was obvious that the reinforcing fabric was helping to hold the embankment intact and preventing complete collapse. The embankment loading was able to resume following dissipation of high excess pore pressures. No further difficulty occurred in this area.

A second critical area of construction required fill placement in about 4.6 m (15 ft) of water overlying about 9.2 m (30 ft) of bay mud. Photo 6 shows the difficulty associated with placement of the reinforcing fabric and the dike embankment. No failures occurred in this area but stability was closely monitored with slope inclinometers to measure lateral movements and piezometers to measure pore pressures in the bay mud.

Photo 6 View showing dozers pushing dike embankment forward over fabric in open water. Fabric is being spread by small barge in right of photo.

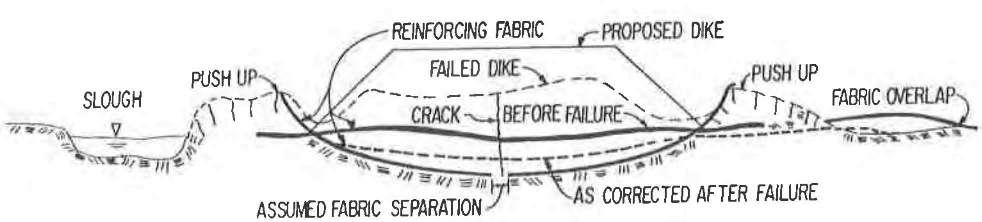

Fig. 5 Cross section of containment dike showing method of failure correction.

About 76 cm (3 in) of lateral movement was monitored over a three-month period but is not considered significant in this soft foundation material. High pore pressures were also monitored. The lateral movements have since stabilized and the excess pore pressures have dissipated sufficiently to allow additional embankment placement.

The main embankment should be completed in the spring of 1982.

ACKNOWLEDGEMENTS

The author expresses his thanks to the various Caltrans technicians and engineers who added to the success of this difficult earthwork project.

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Economic and Technical Aspects of Embankments Reinforced with Fabric
Aspects techniques et économiques des remblais renforcés par le textile

From a comparison of estimated costs it is concluded that reinforcement is unlikely to be worthwhile for permanent road embankments unless fill has to be imported or space restrictions necessitate an embankment with a vertical face. However fabric may enable otherwise unsuitable cohesive soil to be used. The behaviour of fabrics in cohesive soils has therefore been investigated further. In shear box tests adhesion (at zero normal stress) was found to be small. At high normal stresses the fabric-soil shearing resistance approached the shear strength of the soil. In triaxial tests failure usually involved slipping of the fabric, but tensile failure occurredin some circumstances. Finally a computer program based on a slip circle method of analysis has been used to indicate the possible strengthening effect of geotextiles in earth dams and road embankments.

Normalement il ne vaut pas la peine des renforcements pour les remblais de route permanents, sauf dans le cas où il faut importer le sol ou il faut un remblai a pan vertical. Cependant, le textile on peu se servir d'un sol cohésif qui serait autrement peu convenable. On faisait donc des études au sujet du fonctionnement des textiles dans les sols cohésifs. Par suite des essais de cisaillement direct on trouvait que l'adhérence était faible. Aux contraintes normaleset hautes la résistance au cisaillement entre le sol et le textile approchait la force de cisaillement du sol. Dans les essais triaxials l'échec entraînait normalement un glissement du textile mais dans quelques états il y avait des échecs dans la résistance à la traction du textile. Finalement on se servait de la méthode d'analyse de la ligne de glissement circulaire pour indiquer l'effect de renforcement dans la construction des barrages de sol et les remblais de route.

INTRODUCTION

Horizontal layers of fabric may be incorporated in highway embankments and in earth dams to provide increased strength and/or better drainage. This paper is concerned with the strengthening function. The embankments considered are assumed to be constructed on a firm horizontal base.

COSTS OF CONSTRUCTION

To assess the value of fabric reinforcement in embankments with granular fill, estimates have been made of the costs of construction of a variety of reinforced and unreinforced embankments. Reinforcement should permit steeper side slopes to be used with a consequent reduction in the volume of fill and in the area of side slopes and land required. However additional costs are incurred in providing fabric and vertical facing units (if used).

The following cross-sections were compared:-

(a) an unreinforced embankment with $26\frac{1}{2}^{0}$ side slopes, i.e. 2 horizontal : 1 vertical

(b) a reinforced embankment with 45^{0} side slopes

(c) a reinforced embankment with vertical sides formed by turning back each layer of fabric(1)

(d) a reinforced embankment with vertical sides with special facing units similar to the glass reinforced cement units used in the York method of reinforced earth (2). (This is recommendedin preference to type (c) for permanent works).

Two widths of embankment were considered, 10m and 20m at the top, and a range of heights from 3m to 10m. The cost of having to import suitable fill was also evaluated.

The number of layers of reinforcement required for embankments with vertical sides was calculated from Rankine's theory of active earth pressures assuming $\emptyset = 30^{0}$ and $\gamma = 20kN/m^{3}$ for the soil, fabric tensile strength = 50kN/m and that a factor of safety of 3 had to be provided against failure of the fabric in tension. On the basis of some preliminary calculations embankments with 45^{0} side slopes were assumed to have two-thirds of the amount of reinforcement required for embankments with vertical sides.

It was assumed that the fabric extended across the full width of the embankment and that the unit costs of construction (1978 prices) were as follows:

Embankment fill from excavations	£0.52/m^{3}
Imported fill	£3.34/m^{3}
Soiling and sowing slopes	£0.55/m^{2}
Land	£0.25/m^{2}
Fabric	£0.40/m^{2}
Vertical facing units	£13.40/m^{2}

For embankments with a top width of 20m, the estimated costs of construction per metre length are shown in Fig.1.

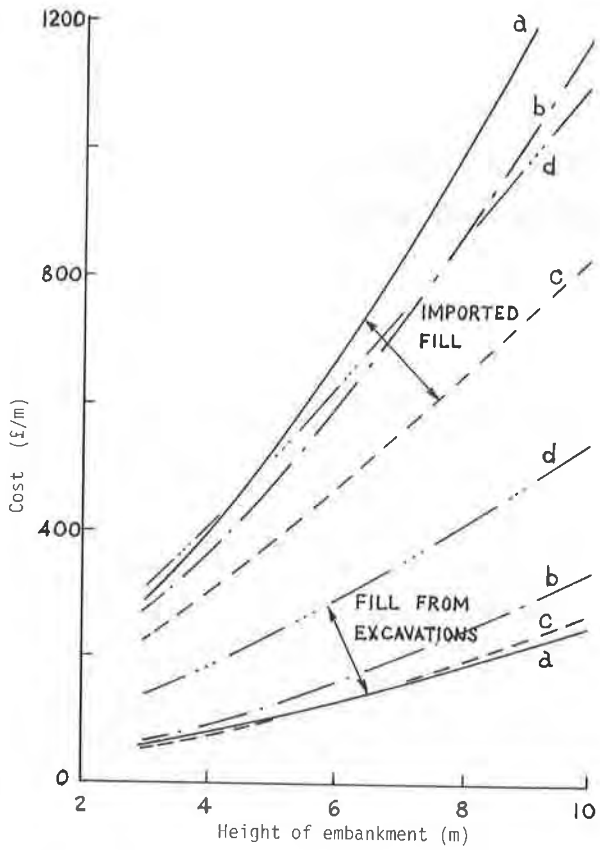


Fig.1 Estimated costs of embankments, top width 20m

(a) Unreinforced, $26\frac{1}{2}^{o}$ side slopes
(b) Reinforced, 45^{o} side slopes
(c) Reinforced, unprotected vertical sides
(d) Reinforced, vertical sides and facing units

The outstanding feature of these graphs is that reinforcement is usually uneconomic if fill can be obtained from the required excavations (e.g. road cuttings) but is worthwhile if the fill has to be imported.

If the top width is reduced to 10m, type (c) reinforced embankments are cheaper than unreinforced embankments for both sources of fill but otherwise the conclusions are unchanged.

There is always a large difference in cost between an unreinforced embankment with imported fill and a reinforced embankment using material from the required excavations. Accordingly a significant saving would result if the addition of reinforcement made an otherwise unsuitable material from the excavations usable as fill. As some cohesive soils may be in this category the remainder of the paper is concerned with some aspects of the use of fabrics in conjunction with such soils.

SHEAR BOX TESTS

To obtain further information about the adhesion and skin friction which can be developed between fabrics and cohesive soils, the shear box tests described in a previous paper (3) have been extended to cover a wider range of soils, moisture contents and fabrics.

Soils included London clay (W_L = 69%, W_p = 29% Proctor optimum moisture content = 26%) tested at moisture contents from 24.0% to 31.7%, well-graded Edinburgh sandy clay (W_L = 28%, W_p = 17%, optimum moisture content = 14.7%) at moisture contents from 10.3% to 19.6%, and Peterhead clay (W_L = 51%, W_p = 25%) at a moisture content of 29.6%.

In addition to the three fabrics tested previously (A - woven polypropylene, B - spunbonded polypropylene - nylon, C - wire-reinforced jute scrim) the following (stronger) fabrics have been used:-

Fabric D : Knitted polyester, Terram RF/12, tested in direction of maximum tensile strength.

Fabric E : Woven polyester, Terram W/20-20

Fabric F : Woven polyester, Terram W/5.5-5.5

Fabric G : Woven tape polypropylene, Lotrak 56/46.

In general the soil-fabric shearing resistance was slightly higher for Fabric C because of its rough texture but otherwise differences between fabrics were insignificant.

With the sandy clay the soil-fabric angle of skin friction was approximately equal to the angle of shearing resistance of the soil. Both decreased slightly as the moisture content of the soil was increased. However, the influence of moisture content was less on soil-fabric adhesion (shearing resistance at zero normal stress) than on the cohesion of soil. Consequently the difference between the total soil-fabric shearing resistance and the shear strength of the soil was smaller at high moisture contents (Fig.2).

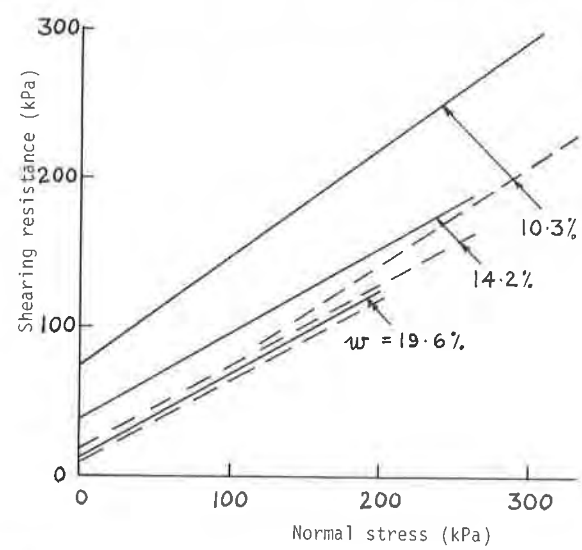


Fig.2 Results of shear box tests for sandy clay at various moisture contents.
———— Soil alone
– – – – Soil in contact with Fabric E

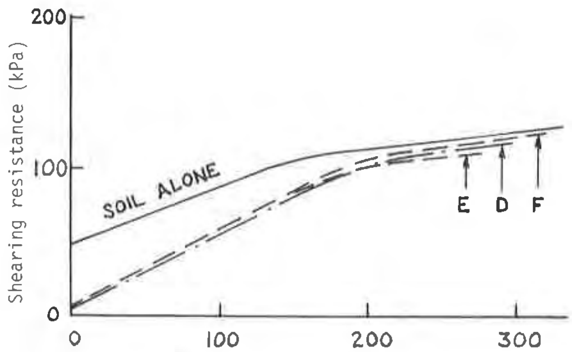


Fig.3 Shear box tests for London clay, alone and in contact with Fabrics D, E, and F.
(w = 29.8%)

The highly plastic clays exhibited a non-linear relationship between shearing resistance and normal stress (see, for example, Fig.3). The angle of shearing resistance became small as the soil approached saturation at high normal stresses. Based on the initial part of such curves the cohesion and angle of shearing resistance of the soil decreased as the moisture content was increased. Over the same range of moisture content the soil-fabric adhesion and angle of skin friction remained fairly constant. In all cases the adhesion was much less than the cohesion of the soil. Consequently at low values of normal stress the total soil-fabric shearing resistance was less than the shear strength of the soil. However at high normal stresses the fabric developed almost the full shear strength of the soil. (At a normal stress of 300 kPa) the latter amounted to 270 kPa for specimens of London clay compacted at a moisture content of 24% and 88 kPa for moisture content = 31.7%)

TRIAXIAL COMPRESSION TESTS

The reinforcing effect of fabric layers in cohesive soils has been investigated by means of a series of triaxial compression tests on specimens 102mm in diameter and 203mm in height (4).

Specimens were made with either London clay or Edinburgh sandy clay at various moisture contents in the region of their Proctor optimum moisture contents. Both reinforced and unreinforced specimens were tested. In the former case, reinforcement was provided by Fabrics C, E and F at various height intervals.

For each reinforced specimen, soil was compacted in 6, 5 or 9 equal layers to suit the placing of 2, 4 or 8 discs of fabric respectively. The number of hammer blows was adjusted to give approximately the same amount of compaction per unit volume as in the standard Proctor compaction test. Each layer of fabric was placed horizontally and covered the full diameter of the specimen. Undrained triaxial tests were carried out at cell pressures of 128, 283 and 421 kPa. A constant rate of vertical deformation of 2mm/min was used.

During the application of the deviator stress it was observed that, although some bulging occurred between layers of fabric, overall barrelling of the specimen was prevented by the fabric. In all cases the deviator stress at failure (defined as the peak stress or as

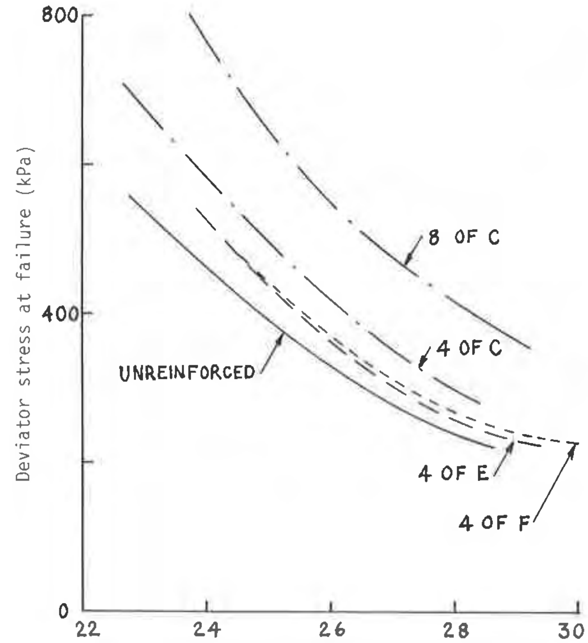


Fig.4 Effect of reinforcing discs on deviator stress at failure in triaxial tests on London clay (σ_3 = 283 kPa).

the stress corresponding to 20% vertical strain if no peak was reached earlier) was increased by the presence of fabric, increasingly so with more layers of fabric (Fig.4).

Failure almost always involved some slipping of the fabric. Exceptions to this occurred in tests with Fabric C in conjunction with Edinburgh sandy clay which had been compacted at moisture contents more than 2.5% below optimum. Under these conditions the soil-fabric shearing resistance was high and tensile failure of the fabric occurred centrally.

Mohr-Coulomb strength envelopes were constructed for both soils at various moisture contents with and without reinforcement. The shear strength parameters obtained from these graphs are given in Table 1.

TABLE 1 Undrained shear strength parameters for compacted soils with and without reinforcing discs of Fabric C.

Moisture content	Unreinforced		Reinforced with 4 discs		Reinforced with 8 discs	
	c	Ø	c	Ø	c	Ø
%	kPa	deg	kPa	deg	kPa	deg
London clay:						
24	185	5	245	5	295	6
26	125	5	160	5	220	6
28	110	2	130	2	135	8
Edinburgh sandy clay:						
11.5	170	18	405	20		
12.7	125	16	260	22		
14.7	110	6	145	16		
16.7	70	1	72	11		

These indicate that the inclusion of fabric in London clay led to an increase in apparent cohesion, especially with closely spaced reinforcement, but the angle of shearing resistance remained low.

With Edinburgh sandy clay compacted at 3.2% below optimum, c was greatly increased by the fabric but only a slight increase in Ø occurred. In this case failure was controlled by the strength of the reinforcement. On the other hand at 2% above optimum only Ø was increased.

While these tests demonstrate the reinforcing effect of fabric layers in cohesive soils, the parameters given in Table 1 are not directly applicable in design problems.

The tests involving tensile failure of the fabric have been analysed further in an attempt to relate the tensile strength of the fabric to an equivalent confining pressure.

Fig.5 shows an element of a horizontal reinforcing disc when this is subjected to a radial body force = R per unit volume. This produces stress resultants N_r and N_ψ at radius r. From conventional theory of circular plates (5) the equation of equilibrium is

$$r^2 \frac{d^2N_r}{dr^2} + 3r \frac{dN_r}{dr} + hrR(4 + 2\nu) = 0 \qquad (1)$$

where h = half-thickness of the disc
and ν = Poisson's ratio.

The body force arises from transfer of stress by shear from the radially expanding soil to the restraining disc. As a first approximation it will be assumed that this is constant. In this case the solution to equation (1) is

$$N_r = \frac{hR}{3}(4 + 2\nu)(r_2 - r) \qquad (2)$$

also

$$N_\psi = \frac{hR}{3}(4 + 2\nu)(r_2 - r) + 2hrR \qquad (3)$$

where r_2 = outer radius of the disc.

From these equations the maximum stress resultant occurs at the centre and has a value of $hRr_2(4 + 2\nu)/3$.

Failure occurs when this is equal to the tensile strength of the reinforcement, α_f.

i.e. when $R = 3\alpha_f/\{hr_2(4 + 2\nu)\}$

The total force transferred to the reinforcement is

$$2hR\int_0^{r_2}\int_0^{2\pi} r\,d\psi\,dr, \text{ i.e. } 6\pi\, r_2\alpha_f/(4 + 2\nu) \text{ at failure.}$$

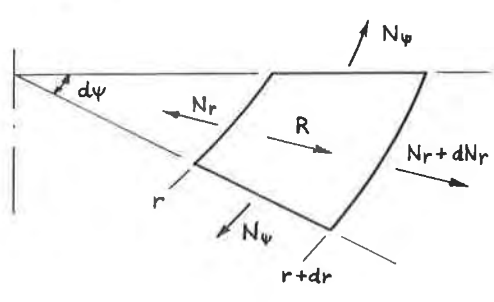


Fig.5 Element of disc subjected to radial body force R.

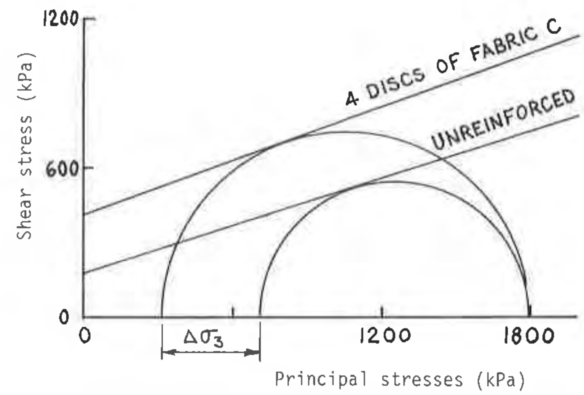


Fig. 6 Mohr-Coulomb strength envelopes for reinforced and unreinforced sandy clay (w = 11.5%)

In terms of the Mohr-Coulomb diagram the reinforcement has the effect of altering the value of cell pressure required for a particular value of maximum principal stress by $\Delta\sigma_3$ (6) as indicated in Fig.6. If the vertical spacing of the reinforcing discs is S_v the reduction in the confining force is $2\pi r_2 S_v \Delta\sigma_3$.

Equating this to the force transferred to the reinforcement gives

$$\Delta\sigma_3 = 3\alpha_f/\{(4 + 2\nu)S_v\} \qquad (4)$$

For specimens containing four layers of Fabric C, for which α_f = 27.5kN/m this gives $\Delta\sigma_3$ = 430 kPa whereas the observed value (Fig.6) is 400 kPa.

ANALYSIS OF EMBANKMENT STABILITY

A design procedure for horizontally reinforced embankments was given in an earlier paper (3). This included a slip circle method of analysis which is relevant to cohesive soils. Dividing the mass above an assumed circular slip surface into vertical slices the following approximate expression was obtained:-

$$F = \frac{\Sigma\{c'l + (W\cos\alpha - ul + Td\sin\alpha)\tan\emptyset' + Td\cos\alpha\}}{\Sigma W\sin\alpha} \qquad (5)$$

where F = factor of safety (assumed equal for shear failure of the soil and tensile failure of the reinforcement)

c', Ø' = shear strength parameters of the soil in terms of effective stresses

T = tensile strength of reinforcement per unit height

W = total weight of a slice

u = pore-water pressure on base of slice

l = length of base of slice

α = slope of base of slice

d = l sinα

In this equation T has units of stress. For example, if fabric having a tensile strength of 40kN/m is laid at vertical intervals of 0.5m, T = 80kPa.

If the lowest part of the slip circle is within rather than on the surface of the embankment the effect of reinforcement should be applied only where d is positive.

A computer program has been written to solve equation (5) for any number of assumed slip circles and hence obtain the critical value of F for a proposed design. For problems involving steady seepage conditions (for example, earth dams with reservoir full) pore-water pressures can be included by specifying a piezometric surface. Alternatively for large embankments during construction one or more pore-pressure ratios r_u can be specified where

r_u = u/total vertical stress.

The same program can be used for an analysis in terms of total stresses by putting u = 0 (artificially) and substituting total stress parameters c and Ø for c' and Ø'. This may be appropriate for smaller highway embankments which can be constructed rapidly and are almost undrained at the end of construction if the permeability of the fill is low.

The computer program has been used to analyse a range of embankment designs. The results provide a guide to the effectiveness of reinforcement under various conditions.
The following range of parameters was covered:-

Height of embankment (H)		10,20m
Angle of side slope		33.7^o,45^o
Tensile strength of reinforcement/m height		0-240kPa
Soil properties:	γ = 20 kN/m^3	
Total stress	c = 10 - 100kPa, Ø = 0	
Effective stress	c'= 0, Ø'= 25^o - 40^o, r_u = 0 - 0.4	

For example, Fig.7 shows some results for a 20m high embankment with 45^o side slopes. In this case a total stress analysis has been used with Ø = 0, representing a saturated clay under undrained conditions. For an unreinforced embankment F is proportional to c/γH. For a given cross-sectional geometry

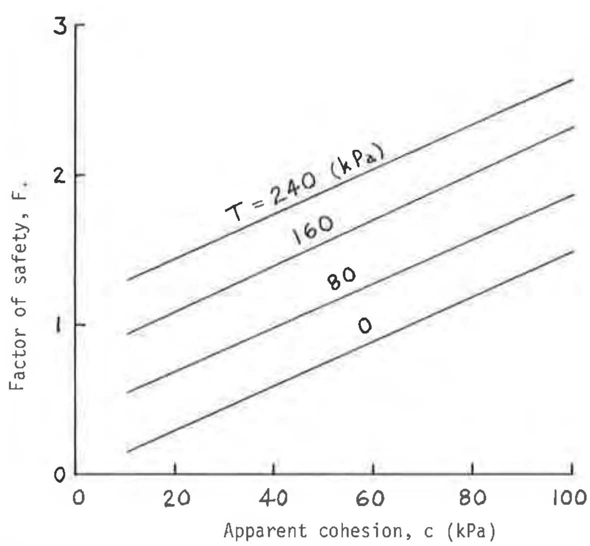
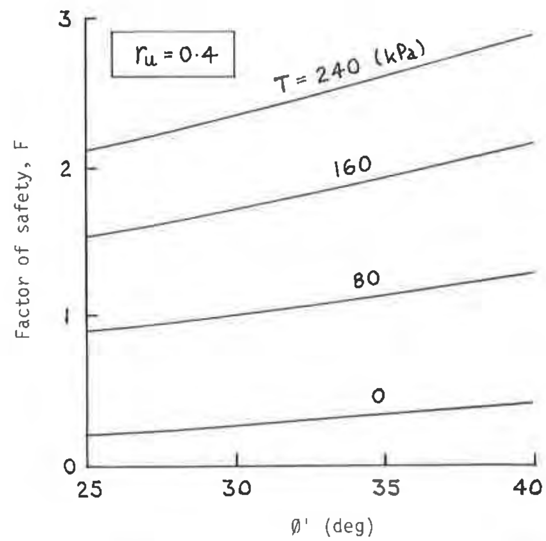


Fig.7 Effect of various amounts of reinforcement (T) on factor of safety of embankments with 45^o side slopes (total stress analysis, Ø = 0).

the increase in F due to reinforcement is directly proportional to its strength. The effect of reinforcement can also be expressed as equivalent to an increase in c. Such an increase is also proportional to T and amounts to 0.32T. This equivalence is constant over the full range of embankments considered in terms of total stresses.



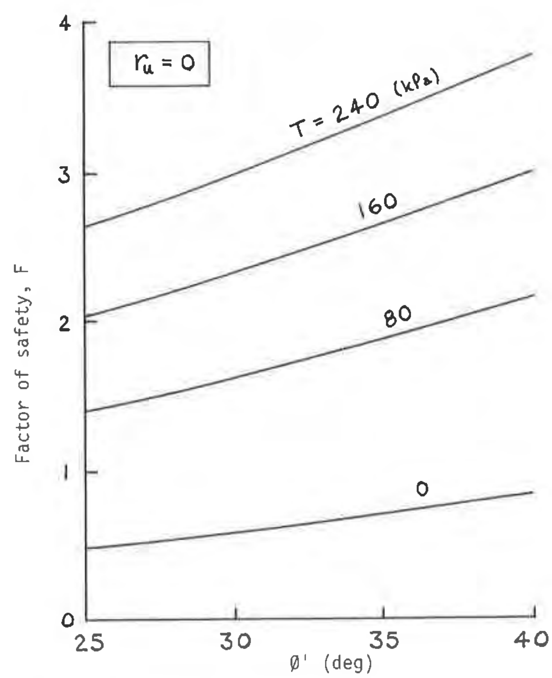


Fig.8 Effect of various amounts of reinforcement (T) on factor of safety of embankments with 45^o side slopes (effective stress analysis, c' = 0).

Fig.8 shows some results based on effective stresses for embankments 20m in height with 45° side slopes. In the example considered c', which is often small in practice, has been neglected. The results shown are for fully-drained embankments ($r_u = 0$) and for embankments with a pore-pressure ratio of 0.4. For an unreinforced embankment there is a linear relationship between F and r_u (7). This applies also to the results for reinforced embankments. Consequently results for other values of r_u can be interpolated from Fig.8. Similar results were obtained for embankments with 33.7° side slopes, for which values of F were about 0.2 to 0.4 higher.

The relationship between the other variables, including height of embankment, is more complicated than in the total stress analysis. However in all cases the factor of safety against this mode of failure is substantially increased by the reinforcement.

In practice, design calculations must also cover the following possibilities:

(1) The shear box tests indicated that the shearing resistance between fabric and cohesive soils is usually less than the shear strength of the soil, especially at low normal stresses. The possibility of a non-circular slip surface passing partly along the interface between a layer of fabric and soil must therefore be considered.

(2) Unless the fabric extends across the full width of the embankment adequate bond length or anchorage must be provided beyond any likely slip surface. A factor of safety equal to the value considered to be acceptable in the slip circle calculations should exist when the pull-out force equals the tensile strength of the fabric.

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Theoretical Design Considerations for Fabric-Reinforced Embankments

Les considérations théorétique de dessein pour des remblais armé avec tissu

The development and verification of embankment design, construction criteria, and theoretical consideration for fabric-reinforced embankments provide basic information necessary for estimation of the unbalanced forces to be carried by geotechnical fabric for known foundation conditions, embankment material properties, embankment heights, and side slopes. Two case histories of fabric-reinforced embankment construction are analyzed and presented. The paper discusses the theoretical design considerations for successful design and construction of fabric-reinforced embankments constructed on soft foundations and correlates the measured fabric elongation and strength with the theoretical computation for each embankment. Nomographic design curves, equations, and design criteria for successful test sections and prototype design and construction are also presented.

Le développement et vérification de dessein des remblais, des critère de la construction, et des considérations théoretique pour des remblais armé avec tissu se prémuni des renseignements de base indispensable pour l'estimation des forces non équilibré a été porté par un géotextile pour des conditions de fondation connu, des proprietes materiele des remblais, l'hateur des remblais, et des pentes du talus. Deux dossiers historique de la construction des remblais armé avec tissu sont analysé et presenté. Ce papier discute des considérations théoretique de dessein pour dessein et construction avec succès des remblais armé avec tissu constructé sur des fondations mou et mis en correlation la mesurage de l' elongation et la force du tissu avec la calcul théoretique de chaque remblai. Des courbes de dessein abaque, des equations, et des critère du dessein avec succès des sections d'essai et du dessein et construction des prototypes sont présenté aussi.

INTRODUCTION

In the past, conventional construction of embankments across extremely soft foundations have principally been displacement sections that contained two to three volumes of material below the surface for one volume above grade. This ratio could easily be much greater even when the embankment base widths are very large. Therefore attempts have been made to design fabric-reinforced embankments that float on these soft foundation materials. Geotechnical fabrics alleviate many soft ground embankment construction problems because they provide equipment mobility, allow expedient construction, and also temporarily suspend several laws associated with foundation bearing failure, allowing construction to design elevation without failure. This paper contains some theoretical design considerations important in the design and construction of a fabric-reinforced embankment. The report also presents the design parameters and analytical procedure used for design and fabric selection in construction at Pinto Pass, Mobile, Ala., and an embankment test section in Holland.

POTENTIAL EMBANKMENT FAILURE MODES

Design and construction of geotechnical fabric-reinforced embankments on soft foundations have been found to be a technically feasible, operationally practical, and cost-effective alternative to conventional soft ground construction in many embankments. To successfully design a fabric-reinforced embankment on a very soft foundation, three possible failure modes must be investigated: (1) horizontal sliding/lateral spreading, (2) rotational slope/foundation failure, and (3) excessive foundation displacement (see Fig. 1) (1). The fabric must resist the unbalanced forces necessary

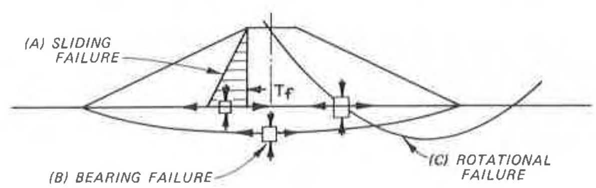

Fig. 1. Potential failure mode that might occur for fabric-reinforced embankments

for embankment stability and must develop moderate to high tensile forces at low to moderate strains. The fabric tensile forces resist the unbalanced forces and the fabric tensile modulus controls the vertical and horizontal displacements of the embankment. Other considerations are development of adequate soil-fabric friction to transfer embankment loads to the fabric as tensile stresses, and the use of proper construction sequence to develop fabric tensile forces at small fabric elongation or strain.

Design Data and Assumptions, Pinto Pass Embankment

A cross section of the Pinto Pass embankment constructed at Mobile, Ala., by the U. S. Army Engineer District, Mobile, is shown in Fig. 2. In addition to

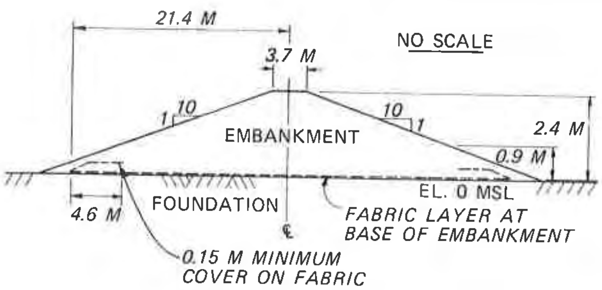

Fig. 2. Simplified fabric-reinforced embankment section at Pinto Pass.

the embankment design cross section, the following detailed data and/or assumptions were used for the analysis:

1. Settlements were computed assuming normally consolidated soils, 12.2 m of sediment thickness, an average initial void ratio of 2.7 and compression index $C_c = 0.8$. Based on these values and a dike height of 2.4 m, settlement was computed to be about 0.9 m.

2. A crest width of 3.7 m was designed to allow future vehicle traffic.

3. To allow for future dike raising, side slopes were 1 vertical and 10 horizontal.

4. Moist unit weight of the cohesionless backfill was estimated to be 1600 kg/m^3 above the permanent water table and 960 kg/m^3 below the water table.

5. It was determined from laboratory tests that the minimum angle of friction $\emptyset$ sf = 30° for the loosely placed backfill and the soil-fabric friction was essentially the same.

6. Field vane shear tests and laboratory tests indicated that the unconsolidated undrained shear strength of the foundation materials prior to construction were cohesion c = 2395 Pa from the surface to a depth of el -3.7 m and increasing linearly from about 4790 Pa to 7185 Pa at a depth of 12.2 m where a dense sand was encountered (see Fig. 3).

Horizontal Sliding/Lateral Spreading of Embankment, Pinto Pass

Resistance to horizontal sliding criteria assumed that, although the soil-fabric frictional resistance of the embankment may be sufficiently greater than the lateral earth pressure necessary to cause sliding, the tensile strength of the fabric may not be great enough and failure may result in fabric tearing and outward sliding of the embankment along the soft foundation.

It was assumed that the horizontal force that might cause lateral sliding could be approximated by Mohr-Coulomb active pressure. The lateral load calculated for the end of construction is:

$$P_a = 0.5\ \gamma_m H^2 \tan^2\left(45° - \frac{\emptyset}{2}\right)$$

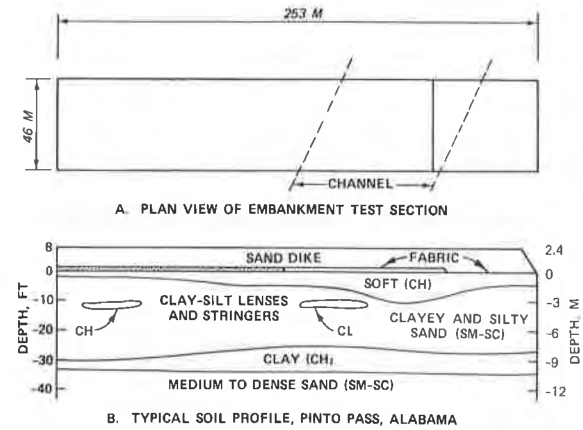

Fig. 3. Plan and soil profile, embankment test section.

where γ_m = density of embankment sand, 1600 kg/m^3; H = maximum embankment height at sta 5+00 at end of construction, 2.5 m; $\emptyset$ = frictional resistance of embankment sand, 30°

or

$$P_a = 0.5\ (1600\ \text{kg/m}^3)(2.5\ \text{m})^2 \tan^2\left(45 - \frac{30°}{2}\right)$$

$$P_a = 1674\ \text{kg/m-width}\ ,\ 16.4\text{KN/m-width}$$

while sliding resistance along the fill material-fabric interface was approximated by

$$P_r = \frac{HL}{2}\ \gamma_m \tan \emptyset\ \text{sf}$$

where P_r = resultant of resisting force (N/m-width), and
$\emptyset$ sf = soil-fabric friction angle = 30°

$$P_r = 0.5 \times 2.5\ \text{m} \times 21.3\ \text{m} \times 1600\ \text{kg/m}^3 \times \tan 30°$$

$$P_r = 24{,}600\ \text{kg/m-width}\quad 241\ \text{KN/m-width}$$

and the factor of safety against sliding was defined as the ratio (P_r/P_a), assuming the fabric tensile strength is not exceeded. By inspection, the controlling parameter was fabric tensile resistance due to lateral active earth pressure.

The horizontal sliding resistance necessary to resist the active pressure would be the ultimate stress of the fabric. Observations made during construction and inspection of the vertical and horizontal settlement plate data indicated that horizontal sliding had occurred with a fabric elongation of about 4.0 percent and a fabric tensile stress of about 14.6 KN/m-width for the fabric. If a minimum safety factor of 2.0 is chosen against sliding, the fabric would provide an ultimate tensile strength, $T_f = 2.0 \times P_a$, or 2.0 × 16.4 KN/m-width or 32.8 KN/m-width, which would exceed the measured tensile stress of 14.6 KN/m-width. This very close agreement of measured and calculated tensile stress indicates this potential unsatisfactory mode controlled the sliding behavior of the test section.

To develop fabric tensile forces and to prevent lateral spreading failure, fabric strain must occur during embankment spreading. Fabric tensile modulus controls the amount of lateral spreading and a factor of safety of 2.0 is normally recommended to determine the minimum required fabric tensile modulus. When the fabric tensile strength T_f is used to determine the required fabric tensile modulus, E_f, a factor or safety of 2.0 is included, and the required fabric tensile modulus is expressed as

$$E_f = T_f/E_{max}$$

where E_f = minimum required fabric tensile modulus (N/m-width), and
E_{max} = maximum fabric strain along the fabric centerline (dimensionless)

The maximum fabric strain over the enbankment width is assumed to be twice the average strain. Lateral spreading of 5 percent has been found to be a reasonable limiting value from previous construction and fabric testing (1). Therefore if 5 percent strain is used as an average value, then the maximum expected strain would be 10 percent and the required tensile modulus would be

$$E_f = T_f/0.10 = 10T_f = (10)(32.8 \text{ KN/m-width})$$
$$E_f = 328 \text{ KN/m-width}$$

This value is the minimum required fabric tensile modulus to prevent lateral embankment spreading failure.

Localized Foundation Bearing Failure and Rotational Subsidence of the Pinto Pass Embankment

This potential unsatisfactory behavior was analyzed by the simplified Bishop slope stability analysis for estimating the ultimate fabric tensile strength necessary to provide a factor of safety against rotational slope failure of a sand embankment on a soft cohesive foundation (Fig. 2b) and the following assumptions were considered in the analysis:

1. Full fabric tensile strength is developed before slope failure.

2. Consideration of shear strength in the embankment was neglected as tensile cracks may occur.

3. The critical slip circle passes through the embankment behind the crest and is tangent to the assumed foundation strength change layer at el -3.7 m where c = 2,395 Pa. Critical failure arcs for the embankment with and without fabric reinforcement are considered to be identical.

4. The embankment and fabric are placed on the foundation simultaneously, and foundation cohesion and ultimate fabric tensile strength are mobilized simultaneously.

5. The likelihood of internal embankment slope failure is minimal because the factor of safety against failure was F = tan 30°/tan 5.7° = 5.8 (where 30° = Ø and 5.7° is embankment slope).

6. The fabric strength is equivalent to the strength of a cohesive clay layer uniformly distributed along the failure arc or plane, and the angle of internal friction is zero.

7. Only the end-of-construction case is considered with the groundwater assumed to be at the same elevation as the fabric reinforcement layer.

Treatment of Fabric Strength, Pinto Pass Embankment

The fabric was laid flat on the soft underlying foundation materials beneath the base of the embankment and the potential failure plane extended through the toe of the embankment. Resistance was provided by the tensile strength of the fabric embedded beneath the embankment and was assumed to act uniformly along the length of the embedded arc length beneath the fabric. Therefore, the total resistance may be mathematically expressed as the sum of the resistance contributed by the fabric and cohesive resistance of the soil or:

$$c = c_f + c_u$$

where c = total cohesive strength, Pa
c_f = equivalent fabric cohesive strength, Pa
c_u = soil cohesive strengths, Pa

The relationship between the fabric tensile strength, T_f , and the equivalent fabric cohesion, c_f , is determined by the following expression:

$$T_f = c_f L_f$$

where L_f = length of the failure arc embedded in the foundation materials beneath the fabric reinforcement.

Therefore, it can be seen that the total resistance, R, for each linear foot of the fabric and foundation soil is as follows:

$$R = L_f c$$

$$R = L_f\ (c_f + c_u)$$

$$R = L_f \left(\frac{T_f}{L_f} + c_u\right)$$

$$R = T_f + L_f c_u$$

Parameter Investigation

To study the influence of the various parameters such as height of the embankment H, the thickness of the soft foundation layer h, and the variables c_f and c_u defined earlier, it was necessary to develop design charts to illustrate their behavior and relationship to one another. Therefore, it was necessary to introduce dimensionless numbers by combining the above parameters as follows:

1. The depth ratio (D) is the sum of the embankment height (H) and foundation layer (h) divided by H, or D = (H + h)/H. A reference line is drawn horizontal and tangent to H at the top of the embankment crest, and dimensions are taken from this line.

2. In conventional slope stability problems, the stability number (N) is defined as $Nc/\gamma_m H$ where γ_m is the moist density of soft foundation soil and c and H are as defined previously.

For a given set of parameters, a critical arc is established first; then the factors of safety and total cohesion are determined for a given arc. All computations for each set of conditions were conducted with the use of the U. S. Army Engineer Waterways Experiment Station (WES) computer.

Design Curves

Since the geometry of the test section was constrained by various design considerations, it was decided to include two sets of design charts that included the design parameters in a dimensionless form. Dimensionless design curves 1 and 2, prepared from several computer runs are shown in Figs. 4 and 5 and

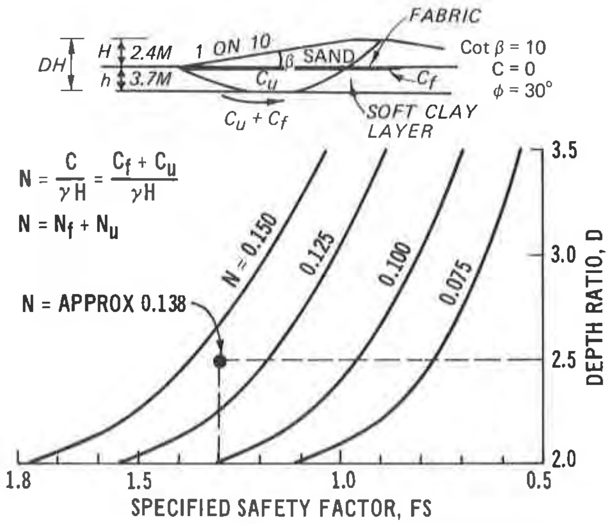

Fig. 4. Design chart 1 for determining stability number N

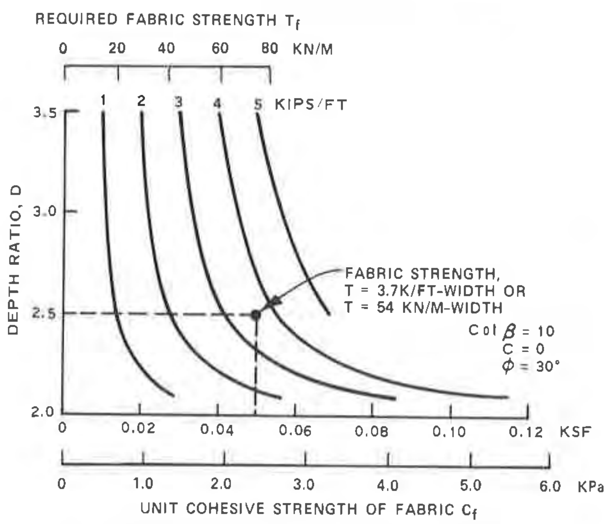

Fig. 5. Design chart 2 for determining fabric strength T_f

were used to determine the proper fabric strength T_f necessary to provide embankment equilibrium and to prevent failure. A sample after construction problem for embankment height H = 2.4 m for the Pinto Pass embankment is as follows:

Given three geometrical parameters (refer to drawing on Fig 2):

(1) Embankment slope 1:10 or Cot B = 10

(2) Embankment height H = 2.4 m

(3) Soft foundation layer h = 3.7 m

and three soil parameters:

(1) Density of embankment materials γ_H = 1600 kg/m^3

(2) Density of soft foundation materials γ_h = 1440 kg/m^3

(3) Cohesive strength of soft foundation soil c_u = 2.4 kPa or KN/m^2

Specified safety factor FS = 1.3

Required: fabric tensile strength T_f

Solution: Find depth ratio D

$$D = \frac{H + h}{H} = \frac{2.4 + 3.7}{2.4} = 2.5$$

From Chart 1, Fig. 4, for a given safety factor FS = 1.3, the total stability number N equals 0.138. The component number N_u for the soil cohesion is:

$$N_u = \frac{c_u}{\gamma_h H} = \frac{2.4 \text{ KN/m}^2}{1440 \text{ kg/m}^3 \text{ (2.4 m)(9.81 N/kg)}}$$

$$N_u = 0.069$$

Therefore, the component number N_f for the fabric is:

$$N = N_u + N_f$$
$$0.138 = 0.069 + N_f$$
$$N_f = 0.069$$

Then, the unit cohesion c_f of the fabric is:

$$c_f = N_f \gamma_h H$$

$$c_f = 0.069 \text{ (1440 kg/m}^3\text{)(2.4 m)}$$

$$c_f = 238 \text{ kg/m}^2 \text{ or 2.3kPa}$$

From Chart 2, Fig. 5, the required fabric tensile strength was determined to be T = 54 KN/m-width.

Results of this investigation indicated that the fabric ultimate tensile strength required to prevent circular arc failure was 32 KN/m-width at an FS of 1.0. This fabric strength requirement is twice as large as the fabric strength required to resist the horizontal sliding mode and also about twice the actual fabric stresses measured after construction. To prevent rotational subsidence an FS of 1.1 to 1.2 was recommended (2), but because this behavior is one of the most difficult to measure, an FS of 1.3 would be more conservative and the chances of success more probable.

There was no evidence of sliding or slumping that might have resulted in a localized bearing failure or stress concentration in the fabric in the embankment test section. A circular arc rotational type failure that resulted in deformation of the embankment and resulting failure in the fabric at the point of sliding was observed in a test section constructed in Holland and reported by Risseeuw (3). This type of failure was documented and data supporting this type of failure are provided and will be discussed later.

Until additional data from controlled tests (such as those in Holland) or prototype structures of this type of behavioral mode become available, it would be expedient to use the fabric strengths determined by the modified Bishop method of analysis. Identifying and measuring the stress in the fabric where these rotational failures may occur, especially at localized points of possible high fabric stress concentration, are very important to these analyses, and every effort should be made to document this type of potential unsatisfactory behavior in future projects. However, based on observed behavior for the test embankment, classic slope stability analysis overpredicts the needed fabric strength by a factor of about 32 KN/m/14.6 KN/m-width or 2.2. Thus, this assumed mode of failure was not critical for the Pinto Pass test section.

Fabric Tensile Stress Developed by Embankment Deformation-Pinto Pass

It has been postulated (2) that once the foundation bearing capacity was exceeded by the embankment bearing pressure, bearing failure and resulting deformation of the foundation would occur. To avoid this type of failure, insofar as possible, the fabric was placed, covered, and anchored by the embankment material before excessive deformations could occur. Bearing capacity of the foundation was exceeded when the embankment height exceeded 0.9 m or (q = 3.8 × 2395 Pa for soft foundations) about 9,100 Pa, and it was assumed that the fabric would carry the remaining weight of the dike (39.8 kPa - 9.1 kPa, or 30.7 kPa) and the embankment would tend to slide or spread laterally, causing tension stresses in the fabric.

Effective soil stresses determined from piezometers along the centerline of the embankment at the end of construction near sta 6+00 was 0 Pa. This confirms the rationale of using unconsolidated undrained shear strength for ultimate bearing capacity calculations. The use of the bearing capacity method for determining the required fabric strength to resist the static loads of the embankment was unsatisfactory.

It was estimated that about 0.9 m of consolidation would occur near the centerline of the embankment, but actually only about 0.5 m occurred. If no lateral displacement is allowed, then the percent fabric elongation and consequent fabric stress can be determined geometrically. The percent fabric elongation was calculated to be less than 0.02 percent; therefore, it was concluded that this magnitude of elongation would not produce appreciably large stresses in the fabric and minimal end anchorage would be necessary.

Controlled Fabric-Reinforced Embankment Failure, Holland

A 120-m-long highway embankment controlled fabric-reinforced test section was constructed in Holland to evaluate several woven geotechnical fabrics for incorporation in a highway to be built across very soft foundation material consisting of clay and peat to a depth of 4.4 m.

Underlying foundation materials consisted of a very soft clay layer having an unconfined compressive strength of about 2395 Pa to a depth of about 0.4 m and approximately 4.0 m of peat with about the same strength. The minimum angle of internal friction for the semicompacted sand fill was considered to be about 30° and equal to the soil-fabric friction. The average saturated unit weight of the foundation material was assumed to be about 1280 kg/m^3 to depth of 4.4 m and the fill material was assumed to be about 1600 kg/m^3. Below this depth was a very dense sand deposit.

The embankment was instrumented with piezometers, settlement plates, and strain wires attached to the fabric to determine the behavior of the reinforced embankment. The embankment had been constructed to a height of 2.5 m and a width of 80 m when a shallow rotational failure about 50 m long and 7.5 m wide occurred and created a subsidence of about 1 m. Excavation test pits were dug both at sections A that did not fail and at section D where the embankment failure was located.

A profile view of the embankment test section in Fig. 6 includes both sections A and D for comparison purposes. This figure shows the fabric prior to failure (section A) and after failure (section D). Embankment slope decreased from 1:1 to 1.5:1 after the embankment surface has subsided about 1 m. About 1 m of fabric was missing in the rotational shear zone shown in Fig. 6. A photograph of the fabric after failure and excavation is shown in Fig. 7. The failure is typical of the type of failure observed after a uniaxial fabric test. Strain wires attached to the fabric were set for 5 percent strain (fabric elongation) increments up to 25 percent total strain, and the fabric failed at about 20 percent strain or a fabric strength, based on uniaxial testing results of about 43.8 KN/mm. No attempt was made to fold back the fabric at the embankment toe to act as an anchor, but there was no evidence that this procedure was necessary or would have provided any additional embankment stability in this case. However, if the fabric had been doubled in the area of the rotational failure, the additional layer of fabric might have served to prevent this potential failure mode.

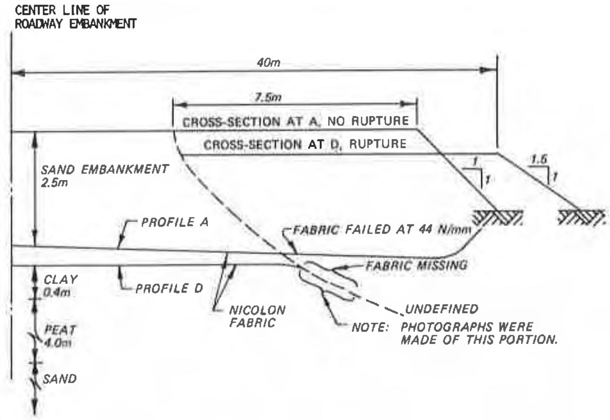

Fig. 6. Profile view of embankment, Holland

Fig. 7. Fabric failure in Holland highway embankment

Analysis of the rotational failure for the highway embankment test section indicated that the failure strength of the fabric was considered to be about the equivalent soil strength necessary to prevent embankment subsidence and subsequent foundation bearing failure. An additional documented failure, such as the one described, has been reported by the Dutch, but to date an account of it has not been published.

Horizontal Sliding/Lateral Spreading of Embankment, Holland

Since the embankments in Holland and at Pinto Pass were almost the same height and exhibited about the same embankment and foundation conditions, it was decided from inspection that this potential failure was not critical. However, further inspection of the slope angle indicated that the probability of internal embankment slope failure is maximized because the factor of safety against failure was $F = \tan 30^\circ/\tan 45^\circ = 0.58$ (where $30^\circ = \phi$ and 45° is the embankment slope). Since the embankment was constructed at the apparent angle of repose of the sand fill, the angle of internal friction ϕ could have been much greater than 30°, and the cohesion of the moist sand could be responsible for supporting these initially steep slopes. The lateral load calculated to be resisted by the fabric at the end of construction was $P_a = 16.4$ KN/m-width, while sliding resistance was $P_r = 454$ KN/m-width, and as at Pinto Pass the controlling parameter was fabric tensile resistance due to lateral earth pressure.

It was further shown as in the previous example analysis at the Pinto Pass test section that the most likely failure mode was the rotational or slip circle type failure. A sample after-construction problem for embankment height, H = 2.5 m for the Holland Roadway embankment is as follows for the rotational failure mode.

Given three geometrical parameters (refer to Fig. 7):

(1) Embankment slope 1:1
(2) Embankment height H = 2.5 m
(3) Soft foundation layer h = 4.4 m

and three soil parameters:

(1) Density of embankment material $\gamma_H = 1600\ kg/m^3$
(2) Density of foundation materials $\gamma_h = 1280\ kg/m^3$
(3) Cohesive strength of soft foundation soil $c_u = 2.4$ kPa or 2.4 KN/m^3

Specified safety factor FS = 1.3; Required fabric tensile strength T , solution; Find depth ratio D:

$$D = \frac{H + h}{H} = \frac{2.5 + 4.4}{2.5} = 2.76$$

From Fig. 4 for a given safety factor FS = 1.3 the total stability number N equals 0.160 and the component number N_u for the soil cohesion is

$$N_u = \frac{c_u}{\gamma_h H} = \frac{2.4\ KN/m^2}{(1280\ kg/m^3)(2.5\ m)(9.81\ N/kg)}$$

$$N_u = 0.076$$

Therefore, the component number N_f for the fabric is:

$$N = N_u + N_f$$
$$0.160 = 0.076 + N_f$$
$$N_f = 0.084$$

Then the unit cohesion C_f of the fabric is

$$C_f = N_f\ \gamma_h H$$
$$C_f = (0.084)(1280\ kg/m^3)(2.5\ m)$$
$$C_f = 269\ kg/m^2 \text{ or } 2.6\ kPa$$

From Fig. 5, the required fabric tensile strength T_f was estimated to be about 65.6 KN/m-width. If the proper fabric strength had been used, the embankment would not have failed at the 43.8 KN/m-width strength estimated from the strain wires attached to the fabric during fabric rupture. The FS would have been about 1.5. An FS of 1.0 was used to estimate the fabric strength of 33.6 KN/m^2, which was about 77 percent of the failure strength estimated in the field test.

SUMMARY AND CONCLUSIONS

Analysis based on field observations and design strengths determined by various design procedures confirmed the need for fabric for reinforcement to prevent embankment failure. Maximum fabric elongation (strain) of 4 percent at 14.8 KN/m-width was recorded in the fabric at the Pinto Pass test section and fabric failure of 43.8 KN/m-width at 20 to 25 percent elongation in the fabric at the Holland embankment.

Data from the Pinto Pass embankment indicated that fabric elongation was within the maximum elongation allowed for the fabric selected during fabric tests in the laboratory. As stated earlier, an average fabric elongation of 5 percent was desired, but fabric elongation of 10 percent would be acceptable in the test section design. The controlled fabric failure in the Holland embankment was predictable when the proper design procedures were used. Comparison of the design procedures used in these analyses indicated that the sliding wedge analysis was more appropriate in that the fabric stress determined for the Pinto Pass embankment by this method was almost identical to the fabric stress measured in the field. The modified Bishop method of analysis using a factor of safety of 1.0 was more conservative, predicting a fabric stress of approximately double that measured in the field at Pinto Pass embankment and about one and one-third times the stress at failure for the Holland embankment. When a factor of safety of 1.0 was used to determine the fabric strength at failure for the Holland embankment, the value determined from the modified Bishop method was only 76 percent of the value measured during failure in the field. The bearing failure method predicted a value of about one-half the field measurements at both the Pinto Pass and Holland embankments. Fabric elongation due to vertical foundation displacement or consolidation was minimal in each embankment.

It is concluded that the modified Bishop method of analysis would provide the most conservative design approach for predicting fabric strength for most embankments constructed utilizing geotexile fabrics on soft foundation materials.

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A Limit Equilibrium Design Method for Reinforced Embankments on Soft Foundations

Méthode d'étudé de l'equilibre limite de remblais armés sur des fondations molles

More widespread and confident applications of mechanical reinforcement in the solution of soils problems are only likely to occur when soundly based calculation methods become available. A method of analysis for reinforced embankments on soft foundations is proposed in this paper. Two important features of the analysis are a clear definition of safety factor and the separation of equilibrium considerations, which can be discussed with confidence, from parameters describing the interaction between the soil and reinforcement. The concepts of reinforcement force required to provide equilibrium and reinforcement force available to do so, are introduced in the paper. Application of the proposed method to the analysis of low embankments on soft foundations is described. Results from back analysis of two field trials using this method are given in an appendix. These lead to the conclusion that the analysis provides a sound basis for the assessment of stability for a reinforced soil embankment.

Il est probable que la méthode d'armature mécanique pour résoudre des problèmes de sols sera appliquée de plus en plus souvent et d'une manière plus sûre au fur et à mesure que des méthodes de calcul correctes deviendront disponibles. Une méthode d'analyse de remblais armés sur des fondations molles est exposée ici. Deux points importants de cette méthode sont une définition claire du facteur de sécurité et la séparation des considérations d'équilibre, qui peuvent être discutées en confiance, à partir de paramètres décrivant l'interaction entre le sol et l'armature. Les concepts de force d'armature requise pour l'équilibre et de force d'armature disponible pour ce faire sont introduits dans cet exposé. L'application de la méthode proposée à l'analyse de remblais bas sur des fondations molles est décrite; Les résultats de deux analyses rétrospectives de deux essais in-situ en utilisant cette méthode sont donnés en annexe. On peut en conclure que la méthode d'analyse fournit une base sûre pour l'evaluation de la stabilité d'un remblai en sol armé.

INTRODUCTION

Mechanical reinforcement can be applied effectively in the solution of a variety of soil problems (1, 2 & 3). These range from uses in steep slopes and vertical walls to provide stability, to uses in embankments on soft foundations to control lateral displacements and short term loss of equilibrium.

Wider and more confident practical applications of mechanical reinforcement are likely to occur, however, only when more soundly based calculation methods become available. These are needed so that the type, strength and distribution of reinforcement, and the security and performance of the reinforced structure can be assessed directly within the existing framework of analyses and definitions currently accepted in geotechnical engineering.

This paper introduces a design method for reinforced embankments on soft foundations. The simplest case of a low embankment reinforced by one layer of reinforcement placed between the embankment and the soft foundation is considered, Fig.1. A limit equilibrium analysis is proposed, which provides a basis for the selection of a compatible embankment geometry and reinforcement layout.

When a soft foundation is loaded both short term stability and long term displacements need to be considered. Attention in this paper is focussed on short term stability only.

Two important features of the analysis are:

- a definition of safety factor consistent with definitions currently accepted in geotechnical engineering, and
- a clear separation in the analysis of equilibrium considerations, which can be discussed with confidence, from parameters describing the interaction between the reinforcement and the soil which are, at the moment, less well defined.

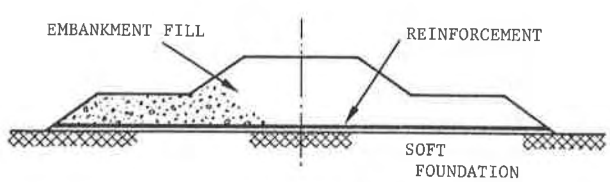

Fig. 1 A reinforced embankment on a soft foundation.

REINFORCED EMBANKMENTS - FIELD STUDIES

The beneficial influence of mechanical reinforcement on embankment performance has been demonstrated in a number of practical trials. On very poor foundations reinforcement is often needed to stabilise the site for plant operation, and so that embankment filling can proceed. This second use is not specifically considered in the paper.

Kerisel (1973)(4) describes a modern application during the construction of a dam across tidal flats in France. A layer of steel mesh reinforcement was placed on the soft foundation surface to protect sand drains from the dumped rockfill being used to form the embankment. The reinforcement helped resist lateral spreading in the foundation and improved overall stability; loss of stability, and the formation of slip surfaces occurred when the reinforcement broke. Two further layers of steel mesh were used to stabilise the remaining construction.

Displacements, which would laterally load piles used to stabilise a sensitive clay foundation to a highway bridge approach embankment in Sweden, were controlled by placing polyester fabric reinforcement on the foundation surface beneath the embankment, Holtz and Massarsch (1976)(5). Inclinometer data showed smaller lateral displacements at the level of the reinforcement than elsewhere.

Four practical demonstrations were presented to the 1st International Conference on the Use of Fabrics in geotechnics (1975) (6, 7, 8 & 9). Comparison of unreinforced and reinforced embankment sections built on the same sites showed that reinforcement reduced settlements and excess pore water pressures (6), increased the maximum height of filling before failure occurred (7), and showed that stiff reinforcement had a more beneficial influence on embankment performance than extensible reinforcement, (9). Bell et al (1977)(8) and Burwash (1980)(10) both describe improved construction of embankments on peat using reinforcement. Burwash had to rely on "judgement" to select an embankment geometry and reinforcement material, "in the absence of accepted analytical procedures".

Two notable case histories have recently been reported by Fowler (1979)(11) and the Study Centre for Road Construction (SCRC) (1981)(12). Fowler describes the testing and selection of fabric reinforcement materials and the site investigation for a low reinforced embankment on soft mud, and discusses construction procedures and the performance of the reinforced embankment during and after construction.(See also Haliburton, Fowler and Langan, 1980 (13)). In the SCRC trials (12) comparative unreinforced and reinforced embankments failed at heights 1.75m and 3.50m respectively. Measurements of soil strength and reinforcement strains are included in the report.

FAILURE MECHANISMS

There are three principal failure mechanisms for a reinforced embankment on a soft foundation, Fig.2. These are:

- Internal stability
- Overall stability
- Foundation stability

The overall embankment height to width ratio is generally governed by the foundation shear strength, and internal stability, Fig.2a, is usually only of secondly importance. However it is necessary to ensure that the upper surface of the reinforcement layer does not provide a discontinuity on which preferential sliding within the embankment can occur. This can be simply checked by using a reduced value of shear strength for the soil at the level of the reinforcement in a routine stability analysis.

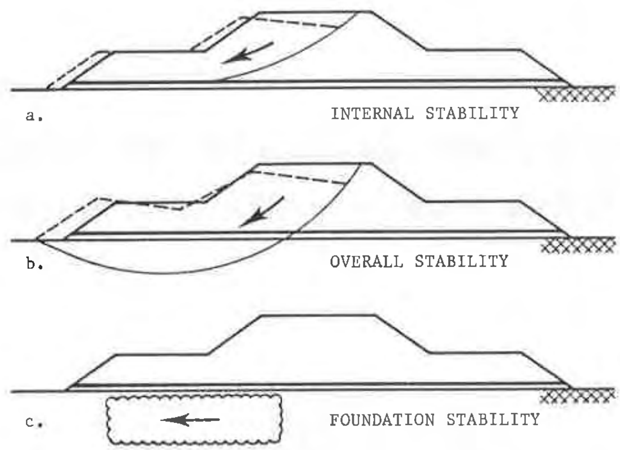

Fig. 2 A schematic view of the three principal failure mechanisms for a reinforced embankment on a soft foundation.

Loss of overall stability, Fig.2b, leads to a block of embankment and foundation soil being displaced along an often well defined failure surface, as observed by Kerisel (4). Strong but relatively extensible reinforcement may prevent the large movements normally accompanying loss of overall stability. In this latter case insufficient reinforcement forces are generated at working deformations to maintain overall equilibrium, which is only re-established after sufficient movement has occurred to generate additional reinforcement forces and restoring soil body forces from embankment settlement and foundation heave. Therefore stiff reinforcement is often desirable.

Loss of foundation stability, Fig.2c, leads to lateral displacements in the foundation soil alone. Squeezing outwards of foundation soil from beneath reinforced embankments has been reported by Kerisel (4) and at the SCRC trials (12).

APPROACH TO DESIGN

In classical soil mechanics stability calculations are normally separated from settlement and deformation calculations. Present knowledge of the role played by effective stresses and improved understanding of the stress-strain behaviour of soils, coupled with the ability to carry out complex numerical computations at a reasonable cost, have provided more precise calculations linking stress and deformation through finite difference, finite element and associated fields techniques (14). In practice, however, simple limit equilibrium stability analyses are still widely used for design.

Limit equilibrium calculations have been suggested for the analysis of reinforced embankments on soft foundations by Wager (1968)(18), Broms (1977)(19), Maagdenberg (1977)(9), Hoedt (1978)(20), Fowler (1979)(11), Bell (1980)(21) and others. In these analyses the reinforcement has either been modelled as a thin, highly cohesive layer or a search has been made for the failure mechanism that requires the greatest reinforcement force for stability, that force being compared to the reinforcement tensile strength.

In contrast to unreinforced soil, it is generally not prudent to ignore soil deformation for reinforced soil even in routine stability analyses. Although an equilibrium calculation, by definition, only examines stresses and forces, it would be potentially misleading to include reinforcement forces in an equilibrium calculation for reinforced soil without questioning whether these forces could be reasonably expected to occur.

Finite element analyses for reinforced embankments on soft soil have been reported by Bell et al (1977)(8), and Brown and Poulos (1980)(15). This work, together with the development of finite element analyses for other reinforced soil applications, for example (16) and (17), will hopefully become more widely available, and used in practice.

OUTLINE OF THE PROPOSED ANALYSIS

The most important aspect of the analysis is clear separation of equilibrium calculations to find the distribution of force along the reinforcement required to provide equilibrium with a specified safety factor, from the assessment of the forces which could be generated in the reinforcement and which are available to provide stability. This separation is desirable because equilibrium calculations are well accepted and can be carried out with confidence, while the interaction between soil and reinforcement, which leads to the generation of forces in the reinforcement and depends in most cases on soil deformation, is currently less well understood and defined.

The key features of the analysis are summarised below.

1. Required forces; a comprehensive series of potential failure surfaces are examined in each case. The objective is not to find the worst failure surface but rather to find the maximum force required at each point along the reinforcement for equilibrium with a specified safety factor. Result; a locus of maximum required force along the reinforcement to provide equilibrium in the embankment with a specified safety factor.

2. Available forces; this calculation is mainly concerned with mobilised soil/reinforcement bond and the relationship between expected soil and reinforcement deformations and strains. Factors considered include embankment geometry, reinforcement layout, water levels, soil strength and deformation characteristics, reinforcement mechanical properties (checking their relevance to in-soil performance and including time effects), soil/reinforcement bond characteristics and the magnitude of the specified safety factor. Result; a profile of available force along the reinforcement.

3. Safety factor; the main safety factor is incorporated in the limit equilibrium calculation of required forces. The conventional definition of safety factor in terms of soil strength is used. The same safety factor can be introduced to derive a mobilised value of soil/reinforcement bond, and material factors on the reinforcement properties can be introduced in the definition of design values for the reinforcement permissible stress and characteristic strength, for the assessment of available forces.

4. Design limit states. Working and ultimate limit states are usually examined for a reinforced embankment. Other cases should be checked as necessary.

The ultimate limit state is a worst case. For an unreinforced embankment an ultimate limit state would exist, for example, if the disturbing forces in the equilibrium equations were increased by the numerical value of the design safety factor. At this ultimate state all the available soil strength would be mobilised to resist collapse. The same argument should apply for a reinforced embankment if the design method is to be consistent with current geotechnical practice. Thus, a locus of required ultimate forces for stability should be calculated as described in 1. for a case where all the disturbing forces are increased by the numerical value of the design safety factor and the full soil shear strength is mobilised. To be consistent with unreinforced designs, the reinforcement must be able to support the ultimate forces without breaking or suffering lack of overall bond.

At the working limit state (working conditions) only a portion of the soil shear strength is mobilised, together with a locus of required working forces to maintain equilibrium.

5. Design criteria. The embankment design is satisfactory if:

a) the required forces at any point along the reinforcement nowhere exceed the profile of available force for all limit states examined;

b) the reinforcement characteristic strength exceeds the maximum value of force that could realistically be generated in the reinforcement.

If the two criteria are satisfied then the reinforced embankment has a minimum overall safety factor not less than the value specified in the calculation of required reinforcement forces at the working limit state.

APPLICATION OF PROPOSED ANALYSIS

The principles described above can be applied to the analysis of end of construction stability for an embankment on soft foundations with a single reinforcement layer in the following way.

Slip Circles

Slip circle failure mechanisms and total stress strength parameters for the foundation soil can be used for simplicity and to be consistent with widely accepted methods for unreinforced embankment analysis (see, for example, Parry, 1971 (22)). A grid of trial circle centres, and a number of trial points evenly spaced along the reinforcement are examined in one analysis, Fig.3.

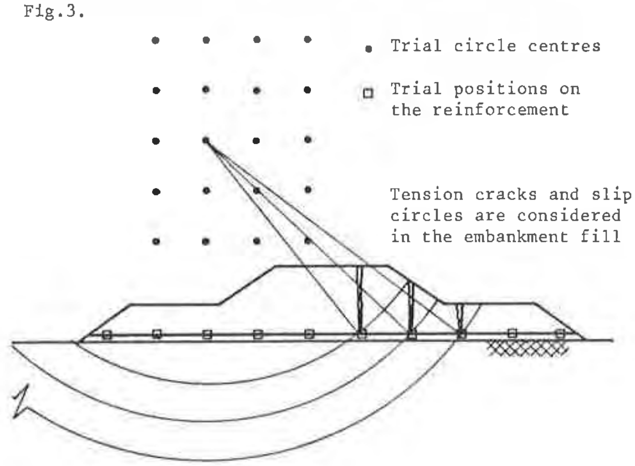

Fig. 3 Slip circle analysis. Results include minimum unreinforced FS and maximum required force for each circle centre, and maximum required force at each trial position on the reinforcement.

In a slip circle analysis both shallow and deep seated failure mechanisms are investigated. A circular surface and a full depth tension crack in the embankment are examined for each trial circle, and the worst of the two cases taken. Typically 100 circle centres and 20 points along the reinforcement might be used.

Slip circles are analysed to give the required reinforcement forces needed for equilibrium at the specified target safety factor on soil strength (typically 1.5). The two most useful forms of output from the analysis are:

- for each circle centre the minimum unreinforced safety factor (and critical circle radius), and the maximum required reinforcement force to give the target safety factor.
- a plot of maximum required reinforcement force at each point along the reinforcement, plotting results for every trial circle for which a force was needed to give the specified target safety factor. A locus of maximum required force along the reinforcement can be constructed from this plot.

It is interesting to note that the minimum unreinforced safety factor and the maximum required reinforcement force for any trial circle centre often do not occur for the same circle.

Contours of minimum unreinforced safety factor and maximum required reinforcement force can be constructed from the results over the grid of trial circle centres. Normally the contours show clearly that the lowest unreinforced safety factor and the highest required reinforcement force fall within the selected grid area of circle centres. The combination of trial circle centres and points along the reinforcement can be refined as desired.

Equilibrium equations

The usual definition of safety for a slip circle is used. The unreinforced safety factor is given by the ratio of restoring moments to disturbing moments.

$$(FS)_{unreinforced} = \frac{M_R}{M_D} \qquad (1)$$

where M_R is the sum of the restoring moments calculated from the soil shear resistance, and M_D is the sum of the disturbing moments, Fig.4.

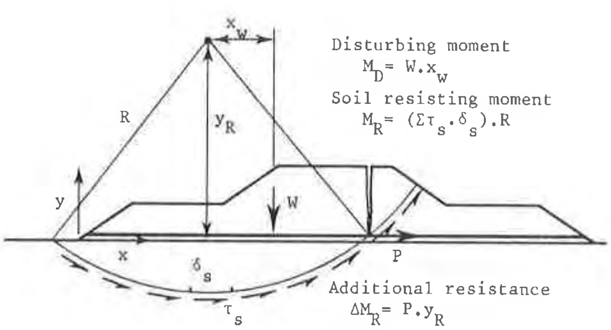

Fig. 4 Definitions and forces for a slip circle analysis.

The effect of reinforcement on equilibrium is calculated by assuming that the reinforcement only modifies the overall stresses carried by the soil. Laboratory investigations of reinforced frictional and cohesive soils at Cambridge University have confirmed that this simple approach provides a good estimate of improved shear strength in reinforced soil (23). The way reinforcement modifies overall stresses can be illustrated by a direct shear test, Fig.5. The reinforcement tension force (when orientated correctly) simultaneously increases the overall normal effective stress and reduces the overall shear stress carried by the soil on the central plane. The shear strength of the reinforced soil is calculated by using the modified overall stresses and a standard failure criterion for the soil.

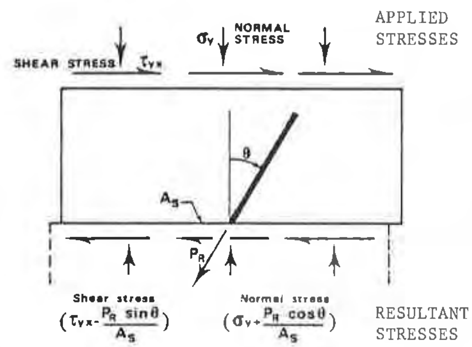

Fig. 5 A schematic illustration of modified stresses in reinforced soil loaded in direct shear.

Two simple and conservative assumptions can be made for the slip circle analysis of low embankments on soft foundation soils.

- the reinforcement force acts in the direction along which the reinforcement was originally placed;
- the reinforcement only reduces the overall shear stresses carried by the soil. (Any additional frictional resistance generated by the increase in overall normal effective stress in the soil due to the reinforcement is ignored).

The reinforcement layer provides an additional restoring moment (ΔM_R),

$$\Delta M_R = P.y_R \qquad (2)$$

where P is the mobilised reinforcement force (KN/m width) at the intersection of the slip circle and the reinforcement, and y_R is the vertical distance between the horizontal reinforcement layer and the slip circle centre.

The safety factor for the reinforced embankment on a given slip circle and with a given mobilised reinforcement force is,

$$(FS)_{reinforced} = \frac{M_R + \Delta M_R}{M_D} \qquad (3)$$

If the magnitude of the overall safety factor for the embankment is initially specified (target safety factor, FT) then the reinforcement force required to satisfy eqn.3 can be calculated.

Required forces

The required reinforcement force at the working limit state (working conditions), P_{WR}, can be defined for any slip circle, eqns.2 & 3.

$$P_{WR} = \frac{M_D - M_R/FT}{y_R} \quad (4)$$

where FT is the target safety factor.

In a simple case, the required reinforcement force at the ultimate limit state, P_{UR}, may be defined by taking the ultimate loading condition as the expected disturbing moment multiplied by the numerical value of the target safety factor, (FT), eqns 2 & 3,

$$P_{UR} = \frac{M_D.FT - M_R}{y_R} \quad (5)$$

All the soil shear resistance and the ultimate reinforcement force is needed on each trial circle to resist this worst loading condition.

The ultimate limit state can also be investigated by combing worst values for soil strengths, soil densities, external loading, water levels, reinforcement location etc. Several analyses may be carried out in a sensitivity study.

Clearly when the simplified approach is used the ratio of ultimate to working required reinforcement force for any slip circle equals the specified target safety factor, eqns. 4 & 5.

Available forces

Three factors which influence the available reinforcement force are,

- the mobilised soil/reinforcement bond
- the distribution of tensile strain in the soil adjacent to and in the direction of the reinforcement
- the load/extension/time or stress/strain/time properties of the reinforcement material in the ground.

Two important reinforcement characteristics are the permissible and ultimate forces or stresses

A profile of maximum available reinforcement force can be derived for any limit state as follows:

A. Select a value of mobilised soil/reinforcement bond stress on each side and at each point along the reinforcement.
B. For anchored reinforcement, select a value of mobilised anchor force at each end of the reinforcement.
C. Select a value of allowable tensile strain in the soil in the direction of the reinforcement at each point along the reinforcement (for simple cases one value might be selected).
D. Determine a load/extension (stress/strain) relationship for the reinforcement in the ground, taking into account the effect of time during which the reinforcement must act.
E. The maximum available force profile can be constructed as follows, working from both ends of the reinforcement. The maximum available force at the reinforcement ends is given by B; away from the reinforcement ends the maximum available force increases at the rate given by the bond stresses in A; the overall maximum force that can be generated is limited by the magnitude of allowable tensile strain in the soil, C, the corresponding reinforcement force being defined by D.

At the working limit state the maximum value of available reinforcement force should not exceed the permissible reinforcement force. At the ultimate limit state the maximum value of available reinforcement force should not exceed the ultimate reinforcement force.

The profile of available reinforcement force defines the maximum value of force at any point on the reinforcement that could realistically be generated. The maximum available reinforcement force profile is a design concept. The shape of the reinforcement force profile that would actually be generated would lie within the available force profile but would equal or exceed the maximum required force at each point along the reinforcement.

Main Design checks

For each limit state two distributions of force along the reinforcement are calculated, Fig.6. The slip circle analysis of equilibrium gives a locus of maximum required force, Fig.6a. The procedure outlined in the previous section gives a profile of maximum available reinforcement force, Fig.6b.

The embankment geometry and reinforcement layout is satisfactory if for each limit state the required force at any point on the reinforcement is less than the maximum available force, Fig.6c.

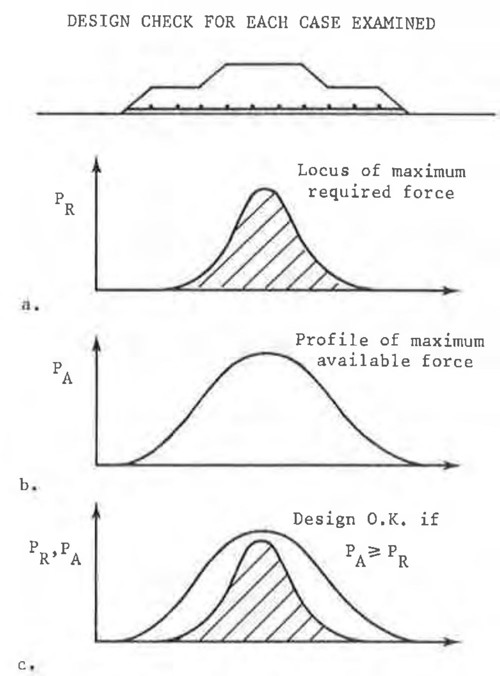

Fig. 6 A schematic view showing the main design check: available forces must exceed the maximum required forces.

Design Cases

Overall stability. The required reinforcement forces are calculated taking the worst of the slip circle or the full depth tension crack in the embankment fill. The available reinforcement force profile is assessed with bond stresses on both the embankment and foundation sides of the reinforcement.

Foundation stability. The required reinforcement forces are calculated using a tension crack over the full depth of the embankment fill (ie the embankment provides only a surcharge loading on the foundation). The available reinforcement force profile is assessed using bond stresses between the foundation and the reinforcement only (ie no bond stresses are mobilised between the reinforcement and embankment fill). This is because during foundation failure, Fig.2c, the block of foundation soil is only restrained by shear between itself and the reinforcement.

Although both the required and available reinforcement forces are generally less for foundation stability than for overall stability, the former condition usually determines the minimum acceptable width for the embankment. The embankment fill can easily be strengthened by reinforcement, but the foundation soil only has reinforcement on its upper surface providing restraint.

PRACTICAL EXAMPLES

The proposed design approach has been applied successfully to the back analysis of the SCRC trials (12) and the embankment test section at Mobile, Alabama (11) and (13). The SCRC trials provide field measurements at an ultimate limit state (failure conditions), while the embankment at Mobile provides data under working conditions.

Results for these two case histories will be reported at the conference, and form an appendix to this paper. They lead to the conclusion that the proposed method of analysis provides a sound and realistic basis from which to assess the performance of reinforced embankments on soft foundations.

CONCLUSIONS

1. The beneficial influence of mechanical reinforcement on embankment performance has been widely demonstrated by practical trials, but there is a need for soundly based calculation methods. In contrast to unreinforced soil, soil deformation needs to be taken into account for the analysis of reinforced soil even for routine stability calculations.

2. A method of analysis is proposed for reinforced soil which clearly separates equilibrium considerations and the calculation of reinforcement forces required for stability, from the assessment of reinforcement forces which could actually be mobilised and are available to provide stability. An application of the method to the analysis of reinforced embankments on soft foundations is described, incorporating a fundamental definition of safety factor consistent with current practice for unreinforced soils.

3. Back analysis of two case histories, one under working conditions and the other at failure has shown that the proposed method realistically models field observations.

ACKNOWLEDGEMENTS

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The Analysis of an Embankment Constructed on a Geotextile

Etude d'un remblai construit sur géotextile

A technique for the analysis of geotextile reinforced embankments is used to examine the factors affecting the performance of a test embankment constructed at Pinto Pass, Alabama. The analysis includes consideration of the development of membrane forces, variable foundation support characteristics, internal embankment arching and load redistribution, plastic failure of the foundation, slip at the soil-fabric interface and simulation of complex construction sequences.

The paper studies the effects of construction sequence, fabric stiffness, underlying soil properties and fill stiffness upon the embankment performance. Particular consideration is given to the effects of the geotextile upon settlements, horizontal movements, membrane forces and embankment stability. The implications of the theoretical predictions and the observed behaviour are briefly discussed.

Nous utilisons une méthode pour analyser les remlais renforcés de fibre geotextile pour étudier les facteurs qui ont pu affecter la performance d'un essai entrepris à Pinto Pass, Alabama. L'analyse considère le développement des forces de membrane, des caractéristiques de support de fondation variables, la redistribution interne de poids, la rupture de fondation en régime plastique, le glissement à l'interface sol-fibre et al simulation de séquences complexes de construction.

Nous étudions dans cette publication les effets de la séquence de construction, de la rigidité du fibre et des propriétés du sol sur la performance d'un remblai. Nous discutons briévement les implications des résultats théoriques ainsi que le comportement observé.

INTRODUCTION

Geotextiles are currently finding acceptance as a means of facilitating the construction of embankments on soft/weak foundations. Much of the literature relating to the effect of geotextiles upon the performance of these embankments is conceptual in nature and, as yet, has not been fully supported by experimental and theoretical studies. Design methods based on these concepts (eg. (5) and others) involve relatively straight-forward extensions of basic engineering concepts relating to earth pressure and slope stability. From an engineering standpoint, these methods are very attractive as they require relatively little input data and the calculations required for fabric selection are extremely simple and straightforward. However, it must be admitted that these approaches do make a number of arbitrary assumptions and it is of some scientific and practical interest to examine the mechanisms affecting the behaviour of these embankments, using more sophisticated analytical techniques.

The Fabric Reinforced Test section constructed at Pinto Pass, Mobile Alabama (4), (7) has played a major role in the development of data and concepts for uses in the design of geotextile reinforced embankments. Consequently in this paper, a finite element soil-structure interaction analysis will be used to examine the behaviour of this test section so as to provide some additional insight into the role of the geotextile.

PRINCIPAL ASSUMPTIONS AND METHOD OF ANALYSIS

The results presented in this paper were obtained using the plane strain, Elasto-Plastic Soil-Structure Interaction Analysis program EPSSIA which is based on the general soil-structure interaction technique proposed by Rowe et al (8). In this analysis, the soil is assumed to be an elastic-plastic material with a Mohr-Coulomb failure criterion (defined in terms of the cohesion c and angle friction ϕ) and a flow rule of the form proposed by Davis (2). For the analyses reported herein, it was assumed that the soil deformed plastically at constant volume, however, provision is made for plastic dilation of the soil if this is considered appropriate. The geotextile was treated as a structural membrane with axial stiffness but negligible flexural rigidity. Provision was made for slip between the fabric and the soil above and/or below the fabric. The displacement of the soil and fabric were assumed to be compatible until the shear stress reached a limiting shear stress defined by a Mohr-Coulomb criterion at the interface. Once this shear stress was attained, slip (ie. differential tangential displacement between the soil and fabric) occurred at this point.

Since the geotextile is considered to develop membrane forces due to deformation, the co-ordinates of the fabric were updated during the analysis. To be consistent, a large deformation analysis was also performed for the soil.

It is noted that the use of this soil-structure interaction approach allows consideration of: the development of membrane forces; variable foundation support characteristics (including local failure); permits the development of internal embankment arching and load redistribution caused by soil displacement;

slippage at the soil-fabric interface if, and only if, the interface shear strength is reached; and permits simulation of complex construction sequences including the placement of the fabric and lapping back of the edge of the fabric to provide anchorage (ie. the approach does not require any arbitrary assumption regarding the anchorage of the fabric). This approach allows the determination of embankment deformations, membrane forces and stability.

PINTO PASS CASE

The design, construction and performance of the Pinto Pass test section has been described in several publications (4), (6) and (7). Briefly, a 2.4 m high, 253 m long and 52.4 m wide test embankment was constructed with a 3.6 m wide crest and 10:1 (horizontal: vertical) side slopes on a foundation of very soft, highly plastic clays and loose clayey fine sands and silts which extended to depths of from 9.75 m to 12 m, where dense clean sand was encountered. In this paper attention will be restricted to a "typical" cross-section of the embankment which, on the basis of limited soils data, may be considered to be underlain by approximately 3.6 m of soft clay (CH), 4 m of clayey and silty sand (SM-SC) and a further 2.15 m of "fairly strong highly plastic clay" (CH) (4) which rested on medium to dense sand. Field vane shear tests indicated undrained shear strengths of approximately 2.4 kPa to a depth of 1.5 m, 4.8 kPa from 1.5 to 2.4 m and 7.2 kPa from 2.4 m to 3.6 m. The embankment was successfully constructed in four sections using four fabrics with properties as indicated in Table 1. The observed settlements of this embankment was typically 0.3 m and in the worst case 0.5 m.

Table 1. Properties of Geotextiles (after Haliburton et al (6). From uniaxial tests using 152 mm wide x 305 mm long samples.

Fabric	Ultimate (kN/m)	Strain at Failure%	Initial Tangent Modulus (kN/m)	Secant Modulus 10%ε(kN/m)
Nicolon 66475	158.0	21	125	634
Polyfiller X	54.5	35	250	180
Advance Type I	44.0	29	613	188
Nicolon 66186	39.6	15	46	190

SELECTION OF PARAMETERS FOR USE IN THE ANALYSIS

The published data relating to the Pinto Pass case involves considerable uncertainty with regard to the soil and fabric properties. Thus to provide some insight into the performance of this embankment, a limited parametric study was performed involving four different representations of the soil profile as indicated in Table 2, and five different values of fabric modulus. It was assumed that: the soil had a unit weight of 14.5 kN/m^3 and K_o = 0.6; the fabric soil interface had a friction angle of 30°; and the soil profile consisted of horizontal layers (so that symmetry could be assumed). Although data supporting this latter assumption is scarce, the approximately symmetric settlements observed beneath most of the embankment would indicate that the assumption is reasonable except at sections 5+00 and 6+00 where the settlements were markedly non-symmetric (this is probably due to the oblique intersection of an old channel with the embankment).

Soil profiles [1], [2] and [4] adopt undrained cohesions c_u based upon the published data to a depth of 5.5 m; there is no direct data below 5.5 m. In profiles [1] and [2] it is assumed that the entire deposit is uniform and normally consolidated and on this basis the undrained shear strength below 5.5 m may be estimated from the available data. In profile [1] the elastic modulus of the soil is assumed to be 1000 c_u and corresponds to undrained conditions.

Strictly speaking, a full effective stress elasto-plastic consolidation (1) analysis should be used to

Table 2. Soil Profiles Examined. (1) ϕ=0°; (2) ϕ=25°

Profile Depth (m)	[1] E kPa	[1] c kPa (1)	[2] E kPa	[2] c kPa (1)	[3] E kPa	[3] c kPa (2)	[4] E kPa	[4] c kPa	[4] ϕ
0 - 0.6	2400	2.4	120	2.4	120	1.9	350	2.4	0
0.6-1.5	2400	2.4	180	2.4	180	1.9	350	2.4	0
1.5-2.4	4800	4.8	240	4.8	240	1.9	500	3.4	0
2.4-3.6	4800	4.8	300	4.8	300	1.9	700	4.8	0
3.6-5.5	7200	7.2	390	7.2	390	2.9	5500	0.5	35°
5.5-7.6	13200	13.2	500	13.2	500	5.75	7000	0.5	35°
7.6-9.75	16800	16.8	600	16.8	600	5.75	7000	16.8	0

obtain the final settlement since this will model the initial undrained response followed by subsequent dissipation of pore pressures. However, it has been found that in predicting the final settlement of embankments of soft soil, it is generally adequate to avoid the consolidation phase by using undrained shear strength parameters and drained elastic parameters (3) (this inconsistent use of undrained and drained parameters is necessary if the approximate approach is to simulate both the plastic strains that develop under undrained conditions as well as the consolidation that occurs as pore pressures dissipate). Thus profile [2] adopts the same shear strength as profile [1] but the elastic moduli are determined assuming a homogeneous deposit to 9.75 m and calculating an equivalent E for the appropriate stress range assuming a uniform normally consolidated deposit with average properties e_o = 2.7 and C_c = 0.8. (Thus profile [2] approximately corresponds to the profile used by Fowler (4, p. 67) for settlement calculations which suggested a settlement of the embankment, without fabric, of 0.91 m (3 ft.). Profile [3] uses both drained shear strengths and modulus values consistent with the soil profile assumed in profiles [1] and [2].

Profile [4] is considered to be the most realistic case. It assumes soft clay to a depth of 3.6 m underlain by 4 m of clayey sand which is assumed to have cohesive frictional properties. The clayey sand is underlain by another 2.15 m of medium clay. The modulus values adopted for this profile were determined from empirical correlations based on the author's previous experience in predicting embankment behaviour.

Table 1 indicated a wide variation in fabric stiffness depending upon the type of fabric and the strain. In addition, it is noted that these results are from uniaxial tests whereas in practice the fabric will be acting in an approximately plane strain mode. Thus despite this available test data, the actual operational stiffness is unknown. To provide an indication of the effect of fabric stiffness, four fabric moduli were considered. The basic case ("E" = 125 kN/m) corresponds to the magnitude at the lower end of the range reported in Table 1. The second (625 kN/m) represents a 5 fold increase and is at the higher end of reported range. The third and fourth cases correspond to a 10 and 100 fold increase in stiffness over the basic case. As a control, analyses were also performed for a fabric with negligible stiffness.

The fill material was assumed to be non-cohesive with a friction angle of 30° and a unit weight of 15.7 kN/m^3. The Youngs modulus E of the fill was defined by an equation of the form $(E/p_a) = K(\sigma_3'/p_a)^n$ in which σ_3' is the minor principal effective stress; p_a is atmospheric pressure; and K, n are two coefficients appropriate for loose material at low confining stresses. Analyses were performed for a number of values of (K, n) namely [A] (57, 0.51); [B] (450, 1.0); [C] (450, 0.5). The first two envelopes were based on triaxial tests performed by the author and his co-workers on loose samples of different sands at appropriate stress levels; the third envelope is based on data from the literature and the density and stress levels may be less appropriate.

Unless otherwise noted, the following results were obtained for the basic fabric (E = 125 kN/m) and fill properties [B] (K = 450, n = 1).

RESULTS

Soil Parameter Selection and Analysis

The effect of the soil profile upon settlements at points on the soil surface located at the centreline and 13.7 m from the centreline, as well as the mobilization of fabric force are shown in Fig. 1. The

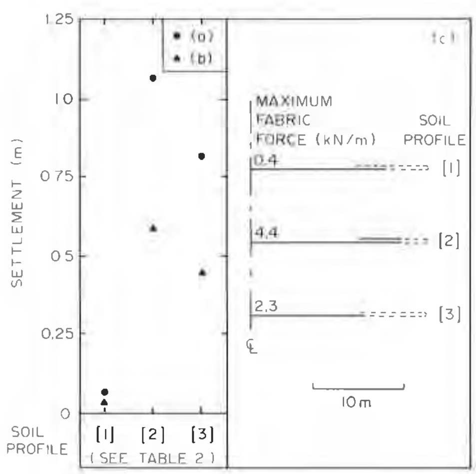

Fig. 1 Variation in Settlement and Fabric Force with Soil Profile (Fabric Stiffness 125 kN/m)
(a) Settlement at Centreline
(b) Settlement 13.7 m from Centreline
(c) Schematic of the Fabric; full line indicates the portion of the fabric where tension is mobilized.

undrained profile [1] gives a relatively small settlement and fabric force. An undrained analysis may be useful in assessing embankment stability, but is unlikely to provide a good indication of the effect of the fabric upon general embankment performance. Because of the much smaller "elastic" modulus of the soil, the fully drained analysis (soil profile [3]) gives much larger settlements and fabric force than the undrained analysis, although the extent of tension in the fabric is slightly less. The drained analysis did not involve any plastic failure within the soil, and hence, all the deformations may be considered to be due to consolidation. This type of calculation provides a lower estimate of the settlement. In practice, local yield will probably occur in the soil mass during construction. Thus the use of the undrained shear strength parameters together with the drained "elastic" parameters (soil profile [2]) provides an indication of undrained stability as well as an upper estimate of the deformations of the embankment (neglecting creep). This analysis does not give the exact final settlement, however, it is considered to be sufficiently accurate (and conservative) for practical purposes; particularly when the uncertainty regarding parameters is considered.

Construction Sequence

Embankment construction was split into 60 steps, and as nearly as possible, followed the construction procedure described by Haliburton et al (7). As construction proceeded, plastic regions developed both within the embankment and within the underlying soil, as illustrated in Fig. 2 for four stages during the construction (soil

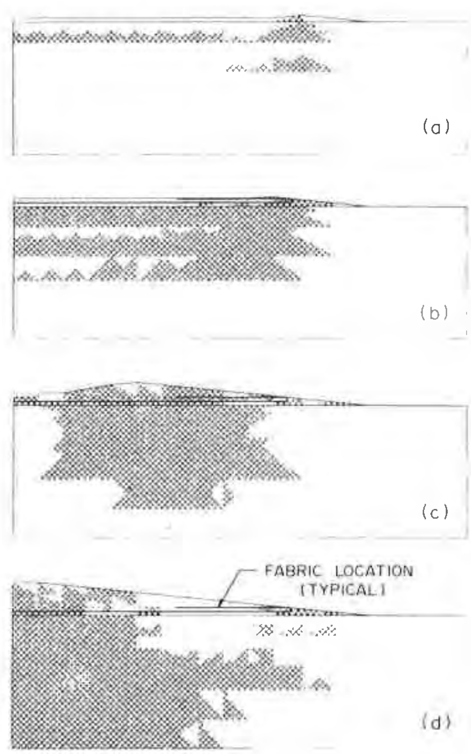

Fig. 2 Plastic Zones at Four Stages During Construction. Soil Profile [2]; Fabric Stiffness 125 kN/m.

profile [2]). Because of the outside-inside construction sequence, some regions of the soil which are plastic at early stages in the construction experience a decrease in deviator stress (and cease to be plastic) at later stages in the construction sequence and this has a beneficial effect upon the general response of the embankment. The plastic region at full construction extends down to the dense sand layer. However, this plasticity is contained and the factor of safety is greater than unity.

A conventional construction sequence involving construction of the embankment in horizontal layers was also considered. No difficulty was encountered in developing the fabric anchorage, although the shear stress was mobilized over a shorter distance. No shear slip at the fabric-soil interface occurred for either construction sequence. However, the construction sequence proposed by Haliburton et al (7) was superior as it gave rise to settlements 10% smaller than obtained using the conventional approach.

Effect of Fabric Stiffness

The vertical settlements at the natural soil surface for points on the centreline and 13.7 m from the centreline are shown in Fig. 3 for a range of fabric stiffnesses

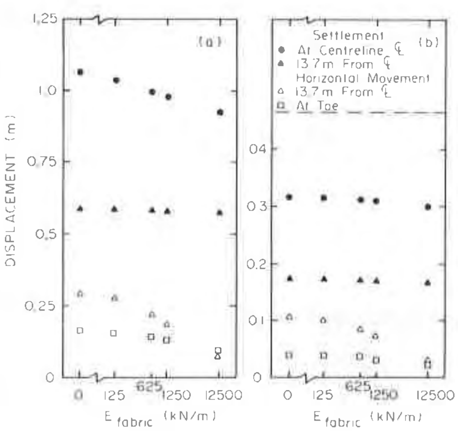

Fig. 3 Variation in Displacement as a Function of Fabric Stiffness for Two Soil Profiles (a) Soil Profile [2], (b) Soil Profile [4].

and two soil profiles. Also shown are the lateral displacements of the 13.7 point and at the toe of the embankment (note that the total lateral spreading will be twice the value shown due to symmetry). Profile [2] gives a centre settlement of 1.06 m for the no fabric case. This is slightly larger but of a similar order to that calculated by Fowler (4) using simple settlement theory and assuming a similar soil profile. However, an inspection of Fig. 3a shows that increasing the fabric stiffness has a very modest effect upon settlements and even the use of an extremely (perhaps unrealistically) stiff fabric only reduces the centreline settlement by 10% to 0.93 m. The differential settlement was also reduced by increasing fabric stiffness but again for this combination of soil and fill moduli, the effect of the fabric was not large. The major effect of the fabric stiffness was upon the horizontal displacements. For the typical range in reported fabric stiffness the horizontal movement was reduced by between 7% and 25%, and by upto 75% over the entire range of stiffness considered. It is of some interest to note that the displacements vary in an approximately linear fashion with the logarithm of the fabric stiffness.

Similar trends were obtained with soil profile [4].

In this case, the settlement of 0.31 m determined without fabric is very close to the observed settlements at Pinto Pass. However, the settlement was reduced by less than 6% for even the stiffest fabric considered. Again the effect of fabric stiffness was greatest for lateral movements which were reduced by between 7% and 21% for the reported range of fabric moduli, and over 70% for the stiffest fabric.

The mobilization of tension within the fabric, and the maximum fabric force are shown in Fig. 4. The soil

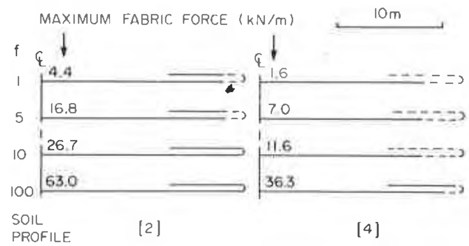

Fig. 4 Development of Tension in Fabric as a Function of Soil Profile and Fabric Stiffness. Tensile forces mobilized in full line region Fabric stiffness = 125f kN/m.

stiffness influences both the degree of tensile mobilization and the magnitude of the maximum tension, however, the results for both the soil profiles suggest that failure in the fabric is unlikely and that the outside edges of the fabric are not stressed (both these findings accord with the observed behaviour). For the typical range of reported fabric stiffness (E = 125 - 625 kN/m) and ultimate tensile capacity (40 - 160 kN/m) the factor of safety against fabric failure ranges between 2.4 and 36 for soil [2] and, more probably, between 5.7 and 100 for soil [4]. The force in the fabric and the degree of mobilization increased with increasing fabric stiffness. Only for the unrealistic stiffness E = 12500 kN/m was the fabric fully tensioned and then only for the large deformations associated with profile [2]. A number of analyses were performed without any fabric overlap and as might be expected, these results were not significantly different from those reported above for the practical range of fabric stiffness.

The foregoing theoretical results would suggest that sufficient fabric anchorage could be mobilized without overlapping the fabric at the edge of the embankment. A similar conclusion was reached by Haliburton et al (7) from consideration of the field performance of the embankment.

The effect of the fabric stiffness upon the development of local yield within the soil is illustrated in Fig. 5 for soil [4]. A comparison of Figs. 5a and 5b indicates that the presence of the basic fabric (E = 125 kN/m) has no significant effect upon the plastic region. In both cases, the embankment would appear to be marginally stable. With the absence of the fabric, the embankment has a factor of safety of approximately 1.13. This factor of safety was determined by reducing the underlying soil shear strength until failure of the embankment occurred and may be compared with the factor of safety of 1.07 obtained from a modified Bishop analysis. This analysis included the shear strength of the embankment (the factor of safety is less than unity without the fill strength). In design it is common, and conservative to neglect the fill strength and stiffness to allow for the possibility of tension cracks, however no tension cracking was reported and the results of this

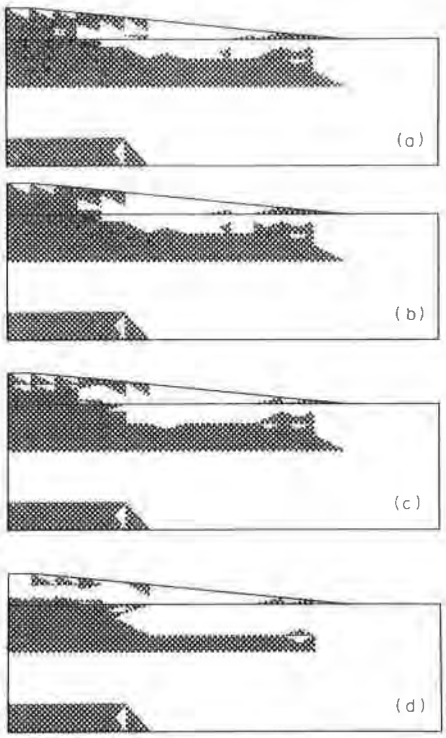

Fig. 5 Plastic Regions After Full Construction for Four Fabric Stiffnesses (a) No fabric (b) E = 125 kN/m (c) E = 1250 kN/m (d) E = 12500 kN/m. Soil Profile [4].

analysis did not indicate the development of tensile failure within the embankment. Hence, for the purposes of interpreting the behaviour of this embankment, it is considered appropriate to include the fill strength in the analysis.

Increasing fabric stiffness above the basic value tends to reduce the extent of local yield although the effect is not particularly evident until the fabric stiffness is 1250 kN/m. (Fabric stiffness has a somewhat greater effect upon the plastic regions for soil profile [2]). The effect of the fabric upon embankment stability may be assessed by reducing the underlying soil strength until uncontained plastic flow occurs. Fig. 6 shows the plastic regions corresponding to a decrease in soil strength such that if failure just occurs, it would imply a factor of safety of 1.25.

Fig. 6 indicates that with a fabric stiffness of 625 kN/m, there is uncontained plasticity and an inspection of the associated velocity field (see Fig. 7) indicated impending failure. The lateral spreading of the embankment has increased more than two fold, however, the tensile capacity has not yet been reached and in that sense, failure has not occurred. Thus a slip circle analysis which incorporates a restoring moment due to the tensile capacity of the fabric would not indicate failure. Nevertheless, the embankment is clearly in severe distress as soil is squeezed out from between the embankment and the stronger underlying clayey sand. Due to the relatively low stiffness of this fabric, extremely large

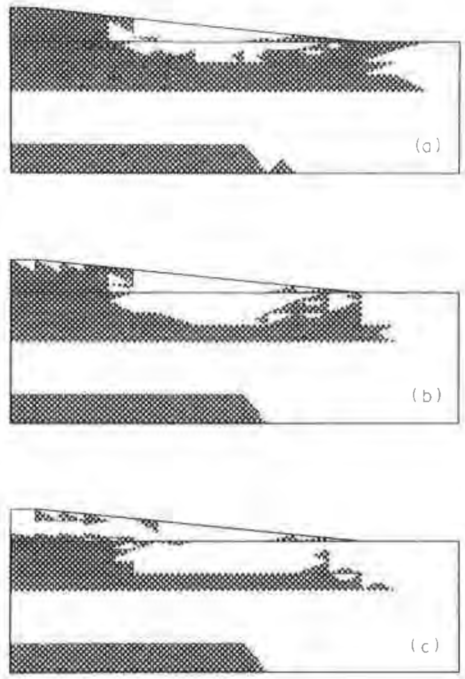

Fig. 6 Plastic Regions at Full Construction for Three Fabric Stiffness (a) E=625 kN/m (b) E=1250 kN/m (c) E=12500 kN/m. Soil Profile [4] except that all shear strengths have been divided by 1.25 [failure at full construction for this case implies a Factor of Safety of 1.25].

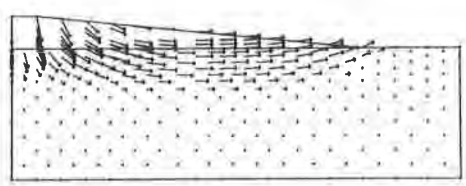

Fig. 7 Velocity Field Corresponding to the Plastic Region Shown in Fig. 6a. Impending Failure

deformations will occur prior to the fabric reaching its tensile capacity and hence for all practical purposes, failure may be deemed to have occurred prior to rupture of the fabric.

The preceding discussion implies that the stiffness of the fabric may be as important as the tensile capacity in assessing the stabilizing influence of the fabric. This is illustrated by comparing the plastic region in Fig. 6 obtained for three fabric stiffnesses. The major role of the fabric is the reduction of lateral spreading, the stiffer the fabric the less the lateral spreading and the greater the confinement of the underlying soil which in turn reduces plasticity in this soil. If failure is considered to have occurred with the onset of uncontained plastic flow (even though rupture of the fabric may not have occurred) then the factor of safety corresponds to the stiffnesses 125, 625, 1250 and 12500 kN/m respectively are 1.18, 1.25, 1.44 and more than 2.25. Failure of the underlying foundation prior to failure of the fabric may be considered to be a ductile failure, while collapse corresponding to failure of the fabric may be expected to be relatively brittle. Thus a knowledge of both the fabric stiffness and tensile capacity determined under plane strain conditions is considered to be very important for the rational design of geotextile reinforced embankments. With this data, it should be possible to design embankments with a primary factor of safety against ductile failure (ie. failure of the foundation) and with a somewhat higher factor of safety against brittle failure (ie. snap of the fabric).

Effect of Fill Stiffness

To illustrate the effect of fill stiffness, analyses were performed for three sets of the parameters (K, n) which define fill modulus as a function of minor principal stress. The results from these analyses indicated that increasing fill stiffness tends to reduce both the vertical and lateral movements. The effect is greatest upon the centreline settlements and least upon horizontal movement at the toe of the embankment. Increasing embankment stiffness tends to enhance the role of the geotextile and, thus, increasing fabric stiffness has a slightly greater effect on deformations for a stiff fill than for a loose fill. However, in all cases the effect of fill stiffness upon deformations is relatively small (ie. typically less than 12%). It was also found that the fill stiffness (assuming the same strength parameters) had negligible effect upon the stability of the geotextile reinforced embankment.

CONCLUSIONS

A technique for the analysis of geotextile reinforced embankments was used to examine the factors affecting the performance of an embankment constructed at Pinto Pass, Alabama. The analysis permits consideration of the development of membrane forces, variable foundation support characteristics, internal embankment arching and load redistribution, slip at the soil-fabric interface and simulation of complex construction sequences. The approach allows the determination of embankment deformations, membrane forces and stability.

The results of this theoretical study indicate that for this case:

1. The outside-inside construction technique proposed by Haliburton et al (7) is superior to normal construction in horizontal lifts even without the use of fabric; however, the role of the fabric may also be enhanced by the technique, particularly for stiff fabrics.
2. The fabric at the edge of the embankment was predicted to be unstressed. Thus, sufficient fabric anchorage could be mobilized without the expense and inconvenience of overlapping of the fabric at the edge of the embankment.
3. The fabric has relatively little effect upon vertical settlements (provided collapse does not occur) but may significantly reduce lateral spreading.
4. For this case, the displacements varied approximately linearly with the logarithm of fabric stiffness. (Additional research is required before the generality of this relationship could be accepted).
5. The fabric does increase the stability of the embankment. However, for fabric with low to moderate stiffness, extremely large deformations may occur prior to the fabric reaching its tensile capacity, and in these cases, failure may be deemed to have occurred prior to rupture of the fabric.
6. A knowledge of both the fabric stiffness and tensile capacity, determined under condition of plane strain, is considered to be essential for the analysis and design of geotextile reinforced embankments.
7. Although some consideration must be given to the fill stiffness, it would appear that for the range of cases considered herein, a precise determination of the fill stiffness is unnecessary.

ACKNOWLEDGEMENT

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An Analytical Study of Geotextile Reinforced Embankments

Une étude analytique de remblais renforcés au géotextile

In recent years great interest has been shown in the use of reinforcement in embankments. Various methods of analysis have been proposed these mainly being adaptations of slip circle analysis including the effects of discrete layers of continuous horizontal reinforcement To be effective such reinforcement should be aligned in the principal tensile strain directions. Since these directions are not always horizontal the blind application of horizontal reinforcement could lead to instability. This tendency can be minimised if the soil at the free face of the embankment slope is encapsulated by the reinforcement. Assuming such a reinforcement geometry two methods of analysis are presented. The first, based on the well established "Infinite Slope" analysis, may be employed to assess the stability of superficial planar slips. The second, based on the Bishop Routine method, may be applied to superficial and more deep seated circular slips. In both cases tentative graphs are presented which subject to verification through field trials, might form the basis of a simple design method.

Au cours des dernières années on a marqué un très vif intérêt à l'égard de l'utilisation du renforcement pour les remblais. Diverses méthodes d'analyse ont été proposées ces dernières étant principalement des adaptations de l'analyse de bande circulaire où figurent les effets de couches discrètes de renforcement horizontal continu Pour être efficace un tel renforcement doit être aligné sur les principales directions de déformation due à la traction. Comme ces directions ne sont pas toujours horizontales l'application aveugle du renforcement horizontal pourrait conduire à l'instabilité.Cette tendance peut être minimisée si le sol sur le parement libre de la pente du remblai est enveloppé par le renforcement.En supposant une telle géométrie du renforcement réalisée deux méthodes d'analyse se présentent.La première,basée sur l'analyse bien établie de la "Pente Infinie",peut être utilisée pour juger de la stabilité des bandes planes superficielles.La seconde,basée sur la méthode de la Routine Bishop,peut s'appliquer à des bandes circulaires superficielles et à des plus profondes.

INTRODUCTION

In the last five years interest in reinforced fill embankments has been renewed, this interest being bolstered by the potentially enormous savings to be made. Various methods of analysis have been proposed, these mainly involving an adaptation of slip circle analysis with the reinforcement modelled as a strong cohesive layer, Volman et al (1), or an equivalent tensile force developing an additional restoring moment, Christie & El-Hadi (2), Phan et al (3), Ingold (4). In executing these analyses the reinforcement is invariably assumed to be arranged in horizontal layers. This configuration is eminently suited to standard embankment construction techniques where the fill is also placed in horizontal layers. However, it must be remembered that the reinforcement should be installed to efficiently resist tension and as such needs to be oriented along or close to the lines of principal tensile strain, Bassett & Last (5). This assertion has been borne out by laboratory tests reported by Snaith et al (6) and McGown et al (7). These findings are of significance to the designer of extensively reinforced embankments. Studies by Bassett and Horner (8) and Jones and Edwards (9), reported by Sims and Jones (10), show that whilst principal tensile strain directions are substantially horizontal in the main body of the embankment they tend to fan-out about the toe of the embankment in the embankment foundation. As a corollary to this potential failure surfaces also exhibit horizontal sections in the embankment foundation. It follows that if horizontal reinforcement is blindly applied, especially near the toe it could well induce, rather than prevent, a potential failure (McGown (11).

1 LOCATION OF REINFORCEMENT

The strain components in any two dimensional strain regime can be conveniently represented by a Mohr circle of strain. Figure 1 shows a regime with a compressive vertical strain ε_z and a compressive horizontal strain ε_x. From knowledge of the shear strain $\frac{1}{2}\gamma_{xz}$ associated with the normal strain ε_x it is possible to construct the Mohr circle of strain. Since the strains $\varepsilon_x, \frac{1}{2}\gamma_{xz}$ are associated with a vertical plane it is found that a vertical line drawn through the point $(\varepsilon_x, \frac{1}{2}\gamma_{xz})$ on the Mohr circle will cut the upper part of the circle at a point Op which defines the origin of planes. A line drawn through Op and a second point on the strain circle will define the inclination of the plane on which the

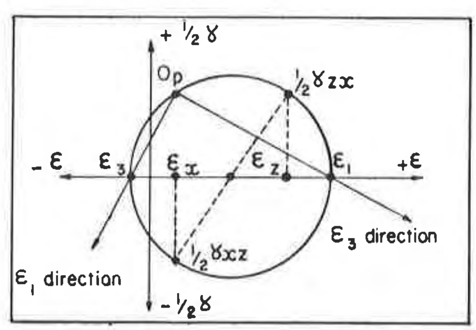

Fig. 1. The Mohr Strain Circle

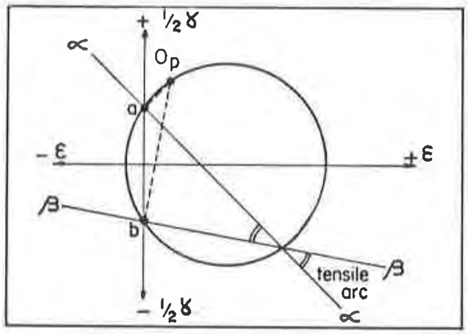

Fig. 2. The α and β Characteristics

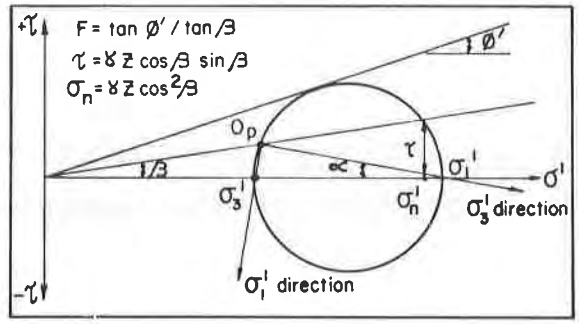

Fig. 4. The Infinite Slope Analysis

stresses, defined by the co-ordinates of the second point, are deemed to act. Thus a line joining Op and ε_3 will define the plane on which ε_3 will act, the direction of ε_3 being normal to this direction. Similarly the inclination of the ε_1 plane is defined by a line joining Op and ε_3. This demonstrates, as shown in Figure 1 that a vertical plane subjected to a compressive horizontal strain ε_x and a shear strain $\frac{1}{2}\gamma_{xz}$ can lead to a negative, or tensile, minor principal strain ε_3 which will be inclined to the horizontal. This construction can be extended to define arcs between which normal strains will be negative. Details of the construction can be followed from Figure 2 where lines are drawn to connect Op with the two points, a & b, where the strain circle cuts the ordinate axis. These two lines represent planes on which the normal strains are zero. Consequently, the zero normal strain directions are at right angles to these two lines which are termed the α and β zero extension characteristics. The physical importance of these lines is that within the arc segment containing the minor principal strain direction all normal strains are tensile Bassett and Last (5). Several research workers have concluded that the α and β zero extension characteristics also represent the potential slip or failure plane, Shields (12), Bransby (13), Lord (14), Milligan (15). The significance of these characteristics to reinforced fill is, quite simply, that tensile reinforcement must, to be effective, be placed in the direction of tensile normal strains, ideally in the direction, and along the line of action, of the minor principal tensile strain ε_3.

The next logical step in applying this theory to reinforced embankment design is to determine the likely principal tensile strain directions in embankments without reinforcement. Taking principal stress and principal strain axes to be coincident this can be achieved by reference to stress and strain distributions published by Clough & Woodward (16) and Valera & Chen (17). The resulting idealized reinforcement layout is shown in Figure 3. As can be seen although horizontal layers of reinforcement would be correctly aligned under the crest of the embankment they would have inappropriate inclinations under the batter, especially at the toe. A similar conclusion has been drawn by McGown (11) who has also pointed out the dangers of reinforcing outside the tensile zones where the reinforcement may have a weakening effect, McGown et al (7). The problem of determining appropriate reinforcement inclinations is exacerbated when the slope angle β is greater than the effective internal angle of shearing resistance which would generally be the case. This problem can be crudely illustrated by application of infinite slope analysis, Haefeli (18). Reference to Figure 4 shows the Mohr stress circle relating to this condition where a slip surface at depth z runs parallel to a slope inclined at β to the horizontal. For a purely frictional soil the normal stress and shear stress acting on the failure plane are $\gamma z\cos^2\beta$ and $\gamma z\cos\beta\sin\beta$ respectively. This leads to an origin of planes Op, as shown in Figure 4 which is associated with the minor principal stress direction rotating α from the horizontal. This rotation is related to the slope angle by equation (1)

$$\tan 2\alpha = \frac{2\tan\beta\sec 2\alpha}{(Kp-1)} + (\sec 2\alpha - 1)\tan\beta \qquad (1)$$

It can be deduced from equation (1) that as the slope angle increases, and the factor of safety drops, the rotation of the minor principal stress increases. This has been confirmed by Charles (19) who measured an increase in α for decreasing factor of safety in soil elements in the upper reaches of an earth dam slope. Analysis becomes meaningless when $\beta > \phi'$ since, as can be seen from Figure 4 the problem falls outside the normally accepted Mohr-Coulomb failure envelope. The problem becomes even more intractable once reinforcement is added since this causes a further rotation of principal stresses. Even though an ideal reinforcement layout might be determined it would be impractical if it took the form of that shown in Figure 3. Consequently the layout shown in Figure 5 has been adopted.

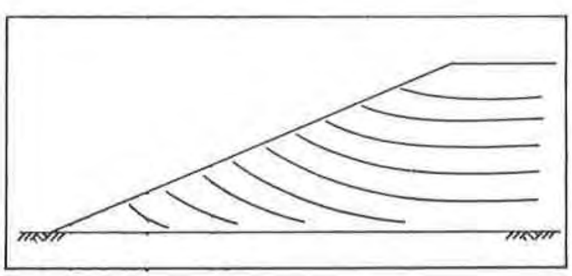

Fig. 3. Idealized Reinforcement Orientations

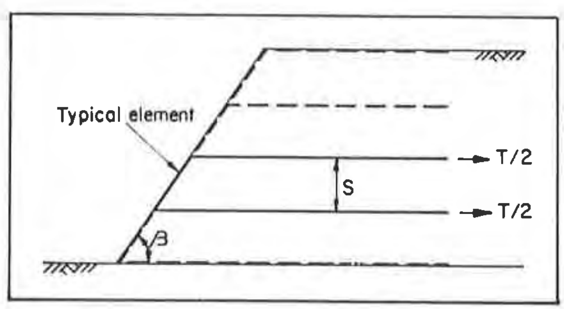

Fig. 5. Encapsulating Reinforcement

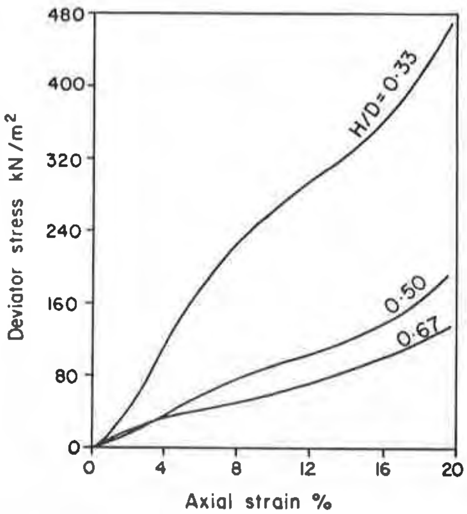

Fig. 6. Test Results for Encapsulated Sand

Using this encapsulating technique the soil close to the face of the batter is strengthened by the normal stress exerted by the reinforcement in contact with the batter face. The mechanism of the strength improvement close to the slope surface defies simple analysis, however, it will almost certainly involve the development of a stabilising stress analogous to an increase in minor principal stress the effects of which have been demonstrated, Ingold (20). It can be seen from Figure 5 for example that with a sufficiently rigid reinforcement with a vertical spacing S the increase in horizontal stress is approximately $\Delta\sigma_x \cong T/S$ where the reinforcement fails in tension and has an ultimate tensile strength of T kN/m. This possibility has been confirmed by triaxial compression tests carried out on sampes of sand contained in cylindrical bags of knitted polyethylene. The bags which were 150mm in diameter, were filled with Boreham Pit sand, d_{50} =425μm, d_{60}/d_{10}= 2.8, compacted to a dry density of 1.70Mg/m³. Three heights of sample, namely 50mm, 75mm and 100mm, were tested to explore the effects of reinforcement spacing. Compression tests were carried out unconfined. Test results, up to axial strains of 20%, are given in Figure 6 which indicates that deviator stress increases with decreasing sample height. In evaluating these results it should be remembered that since testing was carried out without the application of a cell pressure unreinforced samples would be associated with a near zero strength.

2 THE REQUIREMENTS OF A REINFORCING SYSTEM

One purpose of reinforcing an embankment is to enable the use of much steeper side slopes than those pertaining to unreinforced embankments whilst retaining the integrity of the reinforced mass. This entails designing against both superficial and more deep seated failures. Limiting considerations to a dry uniform cohesionless soil it can be shown that stability reaches a limiting condition as the slope angle approaches the internal angle of shearing resistance. For example infinite slope analysis indicates a factor of safety of unity when $\beta = \phi'$, thus if an unreinforced embankment could be constructed with $\beta > \phi'$ there would be failure. If it is assumed for the moment that the embankment is constructed on a competant foundation then failure would involve a series of slips on surfaces passing through the slope or toe of the embankment. As the slip debris is repeatedly removed there would be more slipping until ultimately a

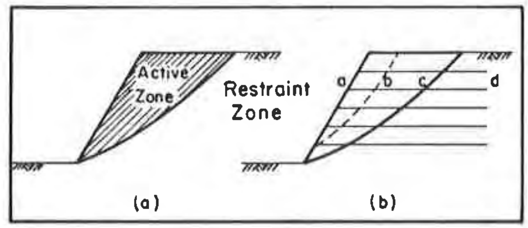

Fig. 7 Modes of Failure

stable condition is reached. This condition is illustrated in Figure 7a which shows the active zone where instability will occur and the restraint zone in which the soil will remain stable. The required function of any reinforcing system would be to maintain the integrity of the active zone and effectively anchor this to the restraint zone to maintain overall integrity of the embankment. In essence this requirement may be achieved by the introduction of a series of horizontal reinforcing or restraining members as indicated in Figure 7b. This arrangement of reinforcement is associated with three prime modes of failure, namely, tensile failure of the reinforcement, pull-out from the restraint zone or pull-out from the active zone. Using horizontal reinforcement that does not encapsulate the soil it would be difficult to guard against the latter mode of failure. Even ignoring the fact that the reinforcement may not be aligned in the appropriate tensile strain arc there is the problem of obtaining adequate bond lenghts. This can be illustrated by reference to Figure 7b which shows a bond length ac for the entire active zone. This bond length may be adequate to generate the required restoring force for the active zone as a rigid mass, however, the active zone contains an infinity of prospective failure surfaces. Many of these may be close to the face of the batter as typified by the broken line in Figure 7b where the bond length would be reduced to length ab and as such be inadequate to restrain the more superficial slips. This reaffirms the soundness of using encapsulating reinforcement where a positive restraining effect can be administered at the very surface of the slope. This reiterates the necessity to develop analytical techiques to assess both superficial and more deep seated instability.

3 INFINITE SLOPE ANALYSIS

Infinite slope analysis may be applied to make some assessment of the possibility of minor slope instability Reference to Figure 8 shows a typical reinforcement arrangement with horizontal encapsulating reinforcement set at a vertical spacing S. Consider first a planar

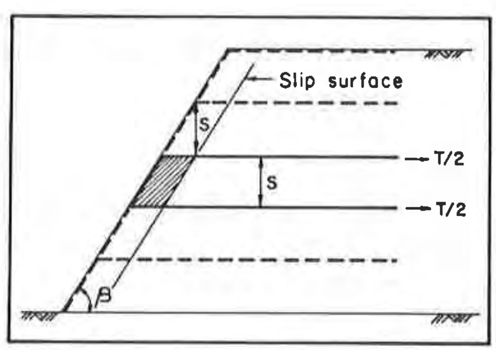

Fig. 8 Infinite Slope Analysis

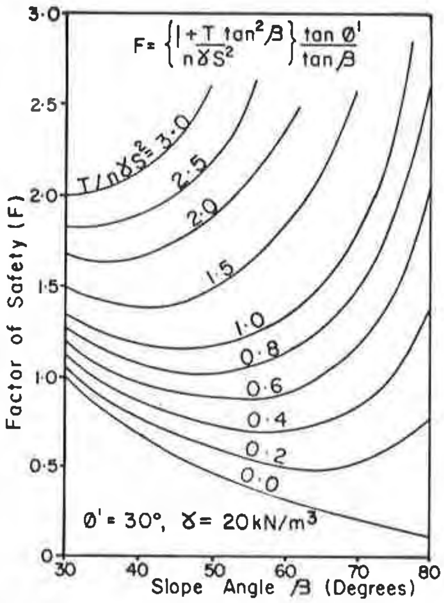

Fig. 9 Infinite Slope Analysis Results

slip surface parallel to the slope batter at a depth S below the slope surface of a dry cohesionless fill. Limiting consideration initially to the stability of the soil mass of weight W contained by two consecutive reinforcements and the slip surface, the hatched area in Figure 8, the disturbing force is $W\sin\beta$. For a soil of unit weight γ then $W=\gamma S^2\cot\beta$. For deeper slip surfaces this weight would increase. Restricting the depths of slip surface investigated to multiples of the reinforcement spacing S then in general $W=n\gamma S^2\cot\beta$ giving rise to a disturbing force $n\gamma S^2\cos\beta$. Restoring forces will be generated by the soil, $n\gamma S^2\cot\beta\cos\beta\tan\phi$, and the reinforcement. The tensile force in the reinforcement may be resolved into the components parallel and normal to the slope. The former component is ignored since to be effective it must be tranmitted through the unstable soil mass in the form of a shear stress. Assuming a tensile failure mode the normal component, $T\sin\beta$, would be mobilised provided the reinforcement is sufficiently stiff. In this case the restoring force is simply $T\sin\beta\tan\phi$. Taking the factor of safety to be the ratio of restoring forces to disturbing forces, equation (2)

$$F = \frac{n\gamma S^2\cot\beta\cos\beta\tan\phi + T\sin\beta\tan\phi}{n\gamma S^2\cos\beta} \tag{2}$$

On rearrangement equation (2) reduces to equation (3)

$$F = \left\{1 + \frac{T}{n\gamma S^2}.\tan^2\beta\right\}\frac{\tan\phi}{\tan\beta} \tag{3}$$

The expression in equation (3) has been evaluated for a range of slope angles and is given in the form of a design chart in Figure 9. As will be seen this, or similar charts, would not be used for the main design per se but merely to check that there is an adequate factor of safety against superficial instability.

4 CIRCULAR SLIP ANALYSIS

The circular slip analysis developed by Bishop (21) has been chosen as the basis for reinforced fill embankment analysis. As a first stage the embankment is analysed unreinforced to define a range of critical factors of safety for various circle geometries. These factors of safety, denoted F_o, are factored to permit determinations of ΔF which is the deficit between F_o and the desired factor of safety F. Reference to the Bishop routine method summarised in equation (4) for a cohesionless soil shows that F_o is a function of $1/m_\alpha$.

$$F_o = \frac{\Sigma[W(1-r_u)\tan\phi'/m_{\alpha o}]}{\Sigma W\sin\alpha} \tag{4}$$

where

$$1/m_{\alpha o} = \sec\alpha \Big/ \frac{(1 + \tan\phi'\tan\alpha)}{F_o}$$

Thus to determine ΔF due allowance must be made for the fact that the final factor of safety operating in the soil will be F as opposed to F_o. This has the effect of reducing the mobilised shear strength of the soil and so leads to a value of ΔF greater than $(F-F_o)$. It follows then from equation (4) that ΔF may be represented by equation (5) where $\bar{m}_\alpha$ relates to the required factor of safety F and $\bar{m}_{\alpha o}$ relates to the factor of safety of the unreinforced embankment F_o. In all cases the $\bar{m}_\alpha$ values are the average values for a particular circle.

$$\Delta F = F - F_o\bar{m}_\alpha/\bar{m}_{\alpha o} \tag{5}$$

Having determined a value of ΔF for a particular slip circle the next step is to determine what reinforcement is necessary to fulfill this requirement. This might be assessed on a trial and error basis by assuming that the horizontal reinforcement generates a restoring moment ΔM which is the sum of the product of the individual tensile force developed in each reinforcing layer and its lever arm about the centre of the slip circle under consideration. That is $\Delta M=\Sigma TR\cos\alpha/F_R$, where F_R is the factor of safety against tensile failure of the reinforcement. The arrangement for a single layer of reinforcement is shown in Figure 10. Contrary to the assumptions of Binquet and Lee (22) the mobilised reinforcing force, T/F_R for each reinforcement, is assumed to act horizontally rather than tangentially. This assumption leads to a lower bound solution, however, this is not thought to be unduly conservative since for T/F_R to act tangentially would require significant movement along the the slip surface and in fact for reinforcement stiff in bending the tangential condition may never be achieved. The effect of the reinforcement may be quantified by modifying the Bishop analysis as set out in equation (6)

$$F = \frac{\Sigma W(1-r_u)\tan\phi'/m_\alpha + \Sigma T\cos\alpha/F_R}{\Sigma W\sin\alpha} \tag{6}$$

In this analysis it is presupposed that the reinforcement fails in tension. This assumption does not lead to any complication since the length of each reinforcement embedded in the restraint zone can be adjusted to resist, with an appropriate factor of safety, any designed pull-

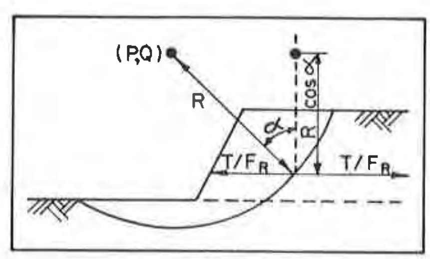

Fig. 10. Circular Slip Analysis

out force. The factor of safety against pull-out or tensile failure may be set at some arbitrary value. However, this does not mean that the maximum value of factor of safety of the reinforced fill embankment per se is limited to this same value. Obviously the greater the amount of reinforcement with a chosen local factor of safety, the greater the global factor of safety becomes.

5 THE DESIGN METHOD

The proposed design method is based on the philosophy presented in the preceding section, namely, to determine what reinforcement is required to obtain a specified factor of safety F for the reinforced fill embankment. Some indication of a simple approach was given by an initial series of dimensionless analyses which showed that, for a slip circle and slope of given geometry and soil properties, the restoring force, ΣT, for a given factor of safety varies in inverse proportion to the product γH^2, equation (7)

$$\Sigma T/\gamma H^2 = \text{Constant} \tag{7}$$

Using this relationship an extensive dimensionless analysis was carried out for a series of embankment slopes reinforced with N layers of reinforcement of allowable tensile strength $T/F_R = H^2$ placed in the lower third of the embankment. Primary reinforcement was restricted to the lower reaches of the embankment since it is in this region that the reinforcement has the largest lever arm and is therefore the most efficient. The material of the embankment was assumed to have weight but no shear strength thus the resulting calculated factor of safety could be attributed to the reinforcement alone and is in fact the value ΔF cited earlier. By running a parallel series of analyses for unreinforced embankments of the same geometry but with fill material having finite strength it was possible to define pairs of values of ΔF and critical values of F_o. These particular values of ΔF, for given values of D/H as defined in Figure 11 have been plotted in the normalised form of $\gamma\Delta F/N$ against slope angle β in Figure 11. The use of the design chart can be illustrated through the example shown in Figure 12. The embankment was first analysed unreinforced for a range of values of P, Q and D/H. This led to the minimum values of F_o and consequently the ΔF values indicated in Table 1. In this particular case the value of ΔF has been calculated

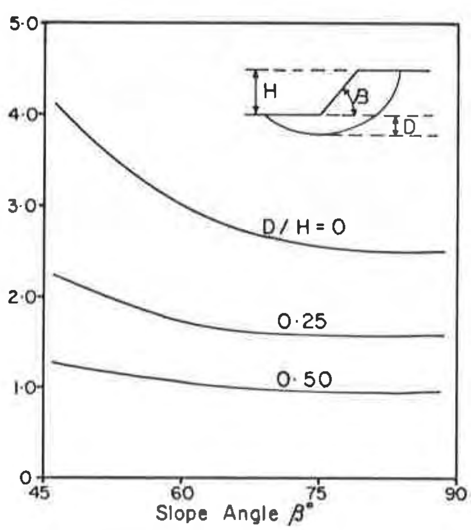

Fig. 11 Embankment Design Chart

assuming a required final factor of safety, F, of 2.

Table 1. Analytical Results

D/H	P	Q	F_o	ΔF	$\gamma\Delta F/N$	N	F
0	0	10.0	1.11	1.05	3.00	7.0	3.61
0.25	-2.5	12.5	1.00	1.07	1.72	12.4	2.42
0.50	-2.5	12.5	1.28	0.75	1.04	14.4	2.12

For the required value of β=60° Figure 11 is entered for D/H=0 whence a value of $\gamma\Delta F/N$ =3.00 is obtained. This is repeated for D/H=0.25 and 0.50 to render values of 1.72 and 1.04 respectively.

Knowing the required values of ΔF, the unit weight of soil, γ, and remembering that $T/F_R=H^2$, which in this case is 100kN/m, it is possible to evaluate N from the respective pairs of values of $\gamma\Delta F/N$ and ΔF. Table 1 shows for example that for D/H=0 it requires 7.0 layers of reinforcement with an allowable tensile force of 100kN/m to obtain the required factor of safety of two. Similarly 12.4 and 14.4 layers are required for D/H values of 0.25 and 0.50 respectively. Obviously the embankment must be reinforced for the worst case examined which occurs when D/H=0.50. It should be pointed out that in final selection of the primary reinforcment it is the product NT/F_R that must be adhered to. In this case 12 layers of reinforcement were adopted with an allowable tensile strength of 14.4x100/12=120kN/m. Since the primary reinforcement is to be restricted to the lower third of the embankment the required spacing is 10/(3x11) =0.3m. Using this reinforcement the embankment was reanalysed using an adapted Bishop routine method which incorporates the effects of the reinforcement. This resulted in values of final factor of safety F shown in the last column of Table 1. As would be expected the reinforcement requirement derived from the design chart renders high factors of safety for D/H values of zero and 0.25, however, for the most critical case occuring at D/H=0.50 the recalculated value of 2.12 is very close to the required value of two.

The above analysis has only considered primary reinforcement, namely that distributed in the lower third of the embankment and as such does not guard against more superficial failures that can occur in the upper two-thirds of the embankment. This can obviously be guarded against by the introduction of appropriate reinforcement. It is useful at this stage to invoke the relationship between tensile strength and the effective embankment height, H', defined in equation (7). On this basis the reinforcement spacing may be maintained at 0.3m but the strength reduced. Bearing in mind that the above case need only be analysed for D/H' of zero the strength should be reduced from that determined for the original D/H value of zero, namely (7x100)/12=58kN/m. Thus the middle third of the embankment where H'=6.7m is reinforced at 0.3m c/c with reinforcement with an allowable

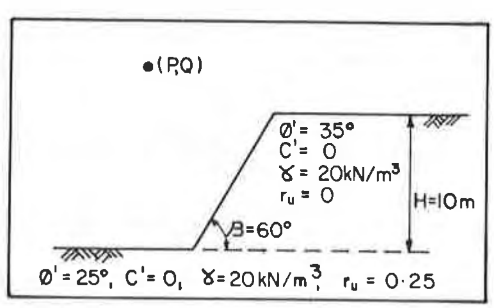

Fig. 12 Trial Analysis

tensile strength of $(6.\beta)^2/(10)^2 x58=26kN/m$ with the upper third reinforced at the same spacing with reinforcement having an allowable tensile strength of $(3.3)^2/(10)^2 x58=6.5kN/m$. When these sections of embankment were reanalysed using the modified Bishop routine method it was found that a factor of safety in excess of two was obtained. It is useful to make a final check on superficial instability using equation (3). This indicates that the upper reinforcement, with an allowable tensile strength of only 6.5kN/m may be a little light since a factor of safety of two only prevails for strips up to 0.82m deep. If this strength is increased to the intermediate value of 26kN/m then the stabilised depth increases to the more acceptable value of 3.3m, also for a factor of safety of two.

So far consideration has been limited to tensile failure of the reinforcement. To prevent bond failure it is necessary to ensure that sufficient length of each reinforcement extends into the restraint zone shown in Figure 7. To accomplish this the Bishop routine analysis has been adapted to calculate the total length of reinforcement in each layer for each circle analysed. This total length includes a bond length, with any specified factor of safety, sufficient to resist the allowable tensile force generated in the reinforcement, in other words a balanced design. Once the maximum required reinforcement lengths have been evaluated using the modified analytical method a final analysis should be made of slip circles passing through the free ends of the reinforcement. These circles will of course not benefit from any reinforcing effect. It is vital that these circles should have an adequate factor of safety against failure without which the reinforced mass might suffer a rotational block failure. Similarly a check should be made against lateral block failure. Additionally if the reinforcement is continuous across the width of the embankment due allowance must be made for tensile loads induced by settlement.

6. CONCLUSIONS

The foregoing presents an analytical techique to design against rotational embankment failure. No account has been taken of settlement induced stress in the reinforcement, however, provided the reinforcement does not run the full width of the embankment these stresses might be negligible. It must be emphasised that this is a purely analytical exercise which has yet to be calibrated by field trials.

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Factors Influencing the Selection of Woven Polypropylene Geotextiles for Earth Reinforcement

Facteurs influegant le choix des géotextiles en polypropylène tissés poive le renforcement des sols

The growth of geotextiles in the construction industry has been rapid over the last decade and many types of geotextile are now well accepted in a wide range of civil engineering tasks.

Among current new possibilities is the use of geotextiles as earth reinforcement. The paper discusses the main properties required of geotextiles in this area of application, with particular reference to woven polypropylene materials. Some suggestions as to geotextile specifications are included.

L'importance des geötextiles dans le domaine de la construction s'est rapidement accrue pendant ces dix dernierès années. diverses sortes de ge'otextiles sont de nos jours utilises dans bien des travaux divers du genie civil.

Parmi les nouvelles possibilities actuelles, l'on trouve l'usage des ge'otextiles pour le renforcement des sols.

Le présent exposé traite des principales propriétés requises des ge'otextiles dans ce domaine d'application; l'on a fait particulièrement référence aux materiaux en polypropylène tissés. L'on a inclus quelques ouggestions en ce qui concerne les spécifications du ge'otextile

1. INTRODUCTION

Textile materials are now used extensively in civil engineering applications and have properties which perform the functions of separation, filtration reinforcement, and drainage in and through the plane.

Initially in Britain carpet backing type materials in both woven and non-woven form were used in simple applications such as temporary roads. These textiles were adequate for non-critical uses having physical properties such as weight, length and width in the roll at a cost acceptable to the user. They could be regarded as "first generation" geotextiles, and no particular design of the textile specific to their application was involved.

More use was made of such textiles over the next few years and as confidence grew more critical applications were tried successfully in the construction industry. It was realised that stronger fabrics gave greater insurance against failure and that the filtration function was also becoming more important. To meet these requirements heavier fabrics were used and more discretion was exercised in their design and application. It was generally appreciated that woven fabrics, weight for weight, performed a reinforcement function better than non-wovens but equally it was thought that the thicker non-woven provided a more efficient filtration function. Such fabrics could be termed "second generation" geotextiles.

At this stage of development, agreed specification and test methods were sought by both textile and construction industries. It was recognised that a range of well-designed geotextiles would be necessary to meet the range of applications and soil types. Manufacturers began to build into their products the physical properties demanded of a particular end use as dictated by the construction industry. For example composite fabrics are now available with separate elements providing strength and filtration functions in a bonded structure. Such fabrics could be termed 'third generation' geotextiles.

The above development has taken place in the last five to ten years and there is still much debate and study being made into the effective role of a geotextile. Standards, test methods and specifications have still to be agreed but at least some standards committees have been set up to study the problems. Meanwhile research into the use of geotextiles continues and as field experience grows more use is being made of these new building materials.

At present soil reinforcement using geotextiles is widely being examined, and field experience is being gained in a number of applications. Woven geotextiles are often favoured for soil reinforcement, and various raw materials and processing techniques are adopted. Although polypropylene is a cheap and relatively inert polymer it tends to be viewed less favourably than other polymers when reinforcement geotextiles are being considered. This paper attempts to show the potential of woven polypropylene geotextiles as soil reinforcement, on the basis of experience derived from extensive laboratory testing coupled with the construction of a trial geotextile-reinforced embankment (1, 2).

In order for a geotextile to act successfully as reinforcement in soil it must satisfy a range of criteria. It should be well suited, both physically and from a cost viewpoint, to the type and height of the earth mass in which it is to be placed. It clearly must be effective as a reinforcement, and should also have sufficient in-plane permeability to prevent local high moisture content zones in the earthfill. During construction it should be sufficiently robust to withstand normal site conditions yet should be capable of being handled easily. In addition, during the design life of the reinforced soil mass the geotextile reinforcement should not be susceptible to degradation, and continuing strains (creep) should be very small. The importance of these factors with regard to the potential of woven polypropylene geotextiles as soil reinforcement is discussed below.

2. POSSIBLE AREAS OF APPLICATION

It is important to recognise that there are a range of types of earth reinforcement. Consequently it should be expected that some geotextile reinforcements may be unsuitable for some of the more demanding jobs, while others may be too expensive for some of the less demanding jobs.

Several reinforcement applications are currently being investigated. These can be divided into two areas-those in which reinforcement is placed internally within earthfill, and those in which reinforcement is placed externally on an earthfill boundary. The former areas include reinforcement in edges of embankments to allow full compaction (3) reinforcement to allow the use of steepened sideslopes thus saving in fill volume and land area (1, 2), reinforcement in Reinforced Earth type retaining walls, (4) and reinforcement in poor quality fills. External reinforcement usually centres around reinforcement of soft foundation strata beneath embankments.

At one extreme these applications require only temporary reinforcement from the geotextile. For example edge reinforcement of embankments requires little more than local short-term strengthening of side slopes. Once construction is complete the stresses imposed on the geotextile are relatively small. Similarly the construction of embankments on soft ground can be facilitated using geotextile sheets placed on the subsoil surface beneath the fill. Once consolidation of the soft ground has occurred as a result of this loading, it may itself support the construction and the geotextile essentially becomes redundant (eg 5). In such applications the geotextile should have suitable tensile strength, stress-strain characteristics and adhesion or friction with the soil to enable it to reinforce successfully.

At the other extreme, however, the geotextile will be required to function continuously as effective reinforcement during the design life of the structure, for example, as with reinforced earth walls. In these applications not only will suitable tensile strength, stress-strain and adhesion characteristics be necessary, and with more stringent specifications, but also the long term preformance must be adequate. Creep or stress relaxation with time should be kept within certain limits, and degradation should be negligible.

3. WOVEN POLYPROPYLENE GEOTEXTILES AS EARTH REINFORCEMENT

The Lambeg Industrial Research Association (LIRA) and the Queen's University of Belfast (QUB) have for the last four years been examining the possibility of reinforcing soil with polypropylene woven tape geotextiles. Initially work was confined to the laboratory, and the basic relevant mechanical and hydraulic properties of sixteen different woven tape fabrics were examined. Laboratory research was then concentrated on the reinforcement effects obtained using polypropylene geotextile inclusions in samples of compacted clay.

It was found that significant reinforcement was possible, but was critically dependent on inclination of the fabric plane relative to shear plane direction (2) and also on other factors, notably the adhesion developed between the geotextile and the soil (1).

Since adhesion was of importance, a shearbox study was carried out using both stiff compacted clay and firm clay consolidated from a slurry. Under both drained and undrained conditions it was found that adhesion was in all cases greater than 70% of the soil control shear strength for normal stresses greater than 100 kN/m^2. For the tests each fabric sample was placed with the weft direction coinciding with the direction of shear because it had been found that adhesion resistance was greatest for this orientation.

Tests at lower normal stresses for the compacted clay revealed very variable adhesion with soil, in some cases less than 80% of the soil control strength. However tests with the much more compressible firm clay indicated much higher values. A most relevant finding (Figure 1) was that the adhesion resistance depended to some extent on the weft secant modulus* although other factors associated with geotextile manufacture are also thought to be important. Further details of the laboratory test series can be found in refs (1,2,6).

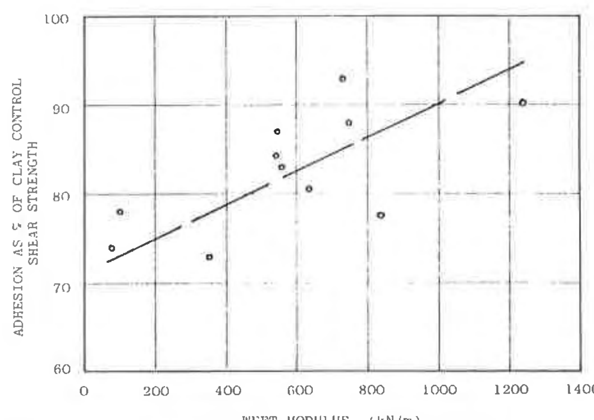

Figure 1

As the laboratory work progressed it became increasingly clear that if possible a field reinforcement trial should be attempted. This was felt necessary to investigate at full scale the viability of earthfill reinforcement with geotextiles. Consequently a full scale reinforced embankment trial was planned, and successfully constructed with the assistance of the Royal Engineers at a site within a military complex at Ballykinler, Co Down, Northern Ireland

* Footnote : The strip tensile stress-strain behaviour for the tested fabrics was linear to very good approximation. Therefore secant modulus = tensile strength ÷ breaking strain is considered a useful representation of deformation behaviour

The embankment was constructed to a height of 4m using cohesionless fill (uniform sand) available on site, and incorporated two geotextile-reinforced sections with side slopes of approximately 70° to the horizontal and two control sections with side slopes of approximately 30° to the horizontal. Laboratory testing demonstrated that the friction developed between the proposed geotextile reinforcement and the sand fill was likely to be very high (1).

In the reinforced sections of the embankment the geotextiles were placed in horizontal layers continuously across the embankment. In order to most efficiently resist the lateral stresses, following from the laboratory shearbox tests, the weft direction was placed laterally with the warp direction therefore coinciding with the long axis of the embankment. This arrangement made site positioning simple since the warp direction is the roll direction for woven tape fabrics. Overlaps of approximately 1m were used with no bonding used between sheets. A low strength geotextile (strength of 17.5 kN/m weft and 17.1 kN/m warp) was employed in the north reinforced section with 0.5m vertical spacing. In the south reinforced section a stronger geotextile (tensile strength 42.0 kN/m weft and 56.4 kN/m warp) was used with 1m vertical spacing.

The embankment performance has been very satisfactory as a clear demonstration of the beneficial reinforcement effects derived from woven polypropylene geotextiles. It was found that tensile strains developed in the reinforcement were set up during compaction. Subsequent bank height increases modified these strains only slightly, indicating the large importance of compaction stresses relative to the overburden stresses. Further details of the trial can be found in references (1,2,7).

4. TENSILE PROPERTIES

In reinforcement applications the tensile performance of the geotextile is of obvious importance. Tensile strength, modulus, extension at break and time-dependency of the tensile stress-strain relationship for the geotextile reinforcement will all influence the behaviour of the reinforced soil mass.

McGown et al (8) have drawn attention to the possible differences in geotextile behaviour when stressed 'in-isolation' as in common laboratory tests, and 'in-soil' as in soil reinforcement applications. As a result, a new laboratory method of determining the tensile performance of geotextiles whilst subjected to normal compressive stress acting through a soil medium has been developed by McGown et al (eg (8). Tests on a polypropylene woven tape fabric indicated that the load-extension behaviour was as would be expected, influenced by the orientation of the test specimen (8). However there was no influence arising from confining pressure.

This suggests that the tensile properties in warp and weft directions, while the geotextile is buried in soil, can be determined using suitable in-isolation laboratory tests. For many reinforcement applications-plane strain conditions apply. The geotextile will be placed so that the width is required to resist high stresses and the length is relatively unstressed. As pointed out previously in this paper the width direction of a woven tape fabric will correspond to the weft direction. Consequently it seems likely that a simple in-isolation laboratory test which can be used to determine weft (and warp) tensile behaviour may be useful not only in quality control but also in the design of certain types of soil-reinforced structure. Clearly more data and field experience is required before this can be confidently accepted. Nevertheless the following discussion, which relates mainly to strip tensile laboratory testing may be of more direct relevance than previously thought.

The upper limit of warp and weft tensile strength for woven tape geotextiles is often determined by the type of loom employed during weaving. For many manufurers polypropylene tapes of up to 3000 denier can be handled, and this sets an approximate upper limit on tensile strength for the completed geotextile of approximately 60 kN/m width.

The maximum height of geotextile-reinforced soil structure which can safely be constructed using such materials can be conservatively estimated using a simple expression, as used for example by Jones (9):

$$\frac{T_{Max}}{F} = K_a \cdot \gamma \cdot H \cdot \Delta H$$

where K_a - coefficient of active earth pressure.
γ - bulk unit weight of soil (kN/m^3)
H - height of structure (m)
T_{Max} - Tensile strength of geotextile kN/m)
ΔH - vertical spacing of reinforcement
F - Factor of Safety against tensile failure of the reinforcement.

Adopting a factor of safety of 3, and conservative values for K_a and γ it is shown that for a vertical spacing of 0.5m structures of 6m to 7m in height depending on fill type can be built. For a vertical spacing of 0.33m structures of up to 10m in height can be built. Taller structures could be constructed with composite-type geotextiles employing mainly polypropylene tapes but with a percentage of tapes made from another polymer. Alternatively double thickness fabrics could be used, although other considerations such as bonding or permeability may prevent this.

A lower limit on geotextile strength for reinforcement applications can be estimated with reference to the effect of stresses set up during compaction of earthfill placed on the reinforcement. The influence of compaction on lateral pressures behind retaining structures has been discussed, for example, by Ingold (10) and its influence has also been noted in connection with reinforced earth structures (11).

Experience gained from the geotextile reinforced embankment at Ballykinler also showed that during compaction significant tensile strains occurred in the geotextile sheets (1,2,7).

Development of geotextile extension with time allowed an estimate of the effects of compaction to be made, and at Ballykinler it seems to have been equivalent to at least 2m of fill. Factors of safety against tensile failure arising from compaction can therefore be determined by examining the performance of reinforcement placed at a level of 2m above the ground surface.

Since the laboratory strip tensile tests revealed approximately linear load-extension relationships for the woven tape fabrics used as reinforcement in the Ballykinler trial, factors of safety against tensile failure could be estimated by comparing the breaking strain from the laboratory tests with the in-situ strain recorded from the geotextile in the embankment.

For the section of embankment reinforced with the weaker fabric the min-factor of safety against tensile failure at the 2m level was 1.8. For the section reinforced with the stronger fabric the min-factor of safety was 2.3. Hence tensile stress developed in the fabrics as a result of compaction, T_c, can be estimated as follows:

(a) weaker fabric : T_{Max} = 17.5 kN/m
(weft) Breaking Strain = 1.2%
T_c = 17.5/1.8 = 10 kN/m approx.

(b) stronger fabric: T_{Max} = 42 kN/m
(weft) Breaking Strain = 6.9%
T_c = 42/2.3 = 18 kN/m approx

It can be seen that the stronger, less extensible fabric apparently develops more tensile stress during compaction than the weaker, more extensible fabric. Assuming a factor of safety of 2 against compaction stress failure suggests lower limits on tensile strength of 20 kN/m (more extensible geotextile) and 36 kN/m (less extensible geotextile).

Several factors will control compaction-induced stresses in geotextile reinforcements. These will include the type and weight of compaction plant, number of passes, type of fill and its friction or adhesion with the geotextile, and the tensile stress-strain behaviour of the geotextile. Consequently the estimates made above can only have approximate general relevance. The less extensible geotextile would be more suitable as a reinforcement material and a minimum strength of about 40 kN/m is therefore suggested.

The tensile deformation behaviour of a reinforcement geotextile is also of concern. This has been discussed by McGown et al (12) who suggests that the brittleness of the reinforced soil is to a certain extent controlled by the relative extensibility of the reinforcement. The laboratory tests conducted by the LIRA/QUB team using both sand and clay soils have shown little evidence of brittleness of soil samples reinforced with woven tape polypropylene geotextiles. These materials had weft extensions at break, measured from strip tensile tests in the range 6% to 20%. However a smaller range, perhaps 6%-12% would seen more appropriate for practical reinforcement purposes to limit extension.

As mentioned previously the weft secant modulus exerts some control over adhesion with clay, and from the laboratory tests (Figure 1) this should be in the range of 600 kN/m to 1000 kN/m. For a fabric with a weft tensile strength these imply extensions at break of 6% to 10%.

5. CREEP BEHAVIOUR

It should be noted that when a fabric is subjected to a load the deformation within the fabric has two forms -

(a) an immediate extension
(b) a slow time-dependent extension or 'creep'.

The creep component of extension is obviously an important parameter where reinforcement is of prime concern. Yet remarkably little has been published in connection with the creep behaviour of geotextiles in general and polypropylene geotextiles in particular.

In a well-conducted experimental study Finnigan (13) examined the creep behaviour of polyester and polyamide yarns and a woven polyester fabric. Van Leeuwen (14) also reports on the results of creep tests on woven fabrics made from polyester and polyamide. Table 1 summarises creep coefficients obtained from data produced from static tests in these studies.

TABLE 1

Polymer	Creep Coefficient (% extension per log 10t)	% Breaking load (approx)	Reference
Polyamide	0.4 (woven fabric)	50	Van Leeuwen
"	0.22 (yarn)	48	Finnigan
"	0.14 (yarn)	27	"
Polyester	0.2 (woven fabric)	50	Van Leeuven
"	0.28(woven fabric-warp)	50	Finnigan
"	0.17(yarn)	41	Finnigan
"	0.14(yarn)	23	Finnigan

Although not included in his original paper Van Leeuwen's creep data was later supplemented (ref 15) by information on a polypropylene fabric which suggested a creep coefficient of about 2.0 was relevant, in comparison with the much lower values recorded for the polyamide and polyester materials.

This data is frequently employed to argue against the use of woven polypropylene fabrics as soil reinforcement, particularly for permanent structures. However, as pointed out by Finnigan (13) and Klobbie (15) heat stretching can improve the creep properties of polymers.

More recently investigations into the creep characteristics of polypropylene yarns and fabrics have been conducted at the Lambeg Industrial Research Association (LIRA) and at the Queen's University of Belfast (QUB).

The work at LIRA (16,17) compared typical polyester and polyamide yarns used in geotextiles with polypropylene tape yarns. It was shown that by correct choice of raw materials and processing conditions it was possible to significantly improve the creep characteristics of polypropylene yarns, see Table 2.

Initial studies, using static load testing have been conducted at QUB, using a strong woven tape polypropylene geotextile which was selected off the shelf and which had not been processed specially to reduce creep. As can be seen from Table 2 the creep coefficients in both warp and weft directions are much lower than might be expected on the basis of yarn creep.

That for the weft direction, which is often more relevant in reinforcement applications, shows good comparison with the reported values from other polymers, see Table 1, although it should be noted that it was stressed at only 20% of its normal (strip tensile) breaking load.

TABLE 2

Polymer	Creep Coefficient (% extension per log 10t)	Material	% Breaking load
Polyamide	0.29	Yarn	40
Polyester	0.15	Yarn	40
Polypropylene	1.50	Yarn (untreated)	40
"	0.40	Yarn (treated)	40
"	0.73	woven geotextile (warp)	20
"	0.41	woven geotextile (weft)	20

Another factor which should be considered in relation to creep performance of reinforcement geotextiles is their in-soil behaviour. The tests mentioned were all done in isolation from the soil environment. There seems little doubt that creep behaviour will be modified by the confining influence of the soil, which itself is time dependent for fine-grained soils. Although creep rates might be expected to be lower in soil than in-isolation there is almost no relevant data currently available and research in this area is needed. Experience with the Ballykinler embankment trial has been less definite than hoped for owing to the very large percentage of geotextile strain gauges which failed to function both during and subsequent to construction. However for periods up to 50 days after construction increases in tensile strain with time were very small or, as in many cases, were non-existent. It is presently almost two years since construction was complete and there is certainly no visual evidence of movement in the embankment which could be related to fabric creep.

6 OTHER RELEVANT FACTORS

6.1 Degradation of Polypropylene

Degradation of textile materials can be classified into the following areas.

(a) Chemical Degradation
(b) Thermal Degradation
(c) Degradation due to exposure to ultra-violet visible radiation
(d) Other Environmental Degradation

The following notes summarise the current state of the art in the above areas.

(a) Chemical Degradation

It is well known that polypropylene has a high resistance to chemical attack. The effects of many chemicals on polypropylene have been investigated in both laboratory and field trials. The results of such experiments are well documented elsewhere (18). For civil engineering applications chemical attack save in the case of accidents would not normally be a problem. Generally polypropylene is only attacked by solvents such as Xylene, trichloroethylene (dry cleaning solvent), turpentine, gasoline and aviation fuel etc. Such chemicals are known to cause swelling of polypropylene at low temperatures, but for extensive damage to occur temperatures in excess of 100°C are required.

(b) Thermal Degradation

Polypropylene is generally regarded as being thermally stable for long periods at ambient temperatures. Work has shown that commercially produced polypropylene material (with no surface lubrications) exposed at 80°C for periods up to one year show no appreciable loss in tensile properties. On the other hand application of lubricants (to assist with further processing ie weaving) can accelerate the thermal oxidation of the materials, with resultant loss in tensile properties. Results show that such materials exposed for a one year period at 80°C can lose up to 50% of their original strength (19)

(c) Degradation due to exposure to ultra-violet and visible radiation

Unstabilised polypropylene is very susceptible to attack particularly by ultra-violet (UV) radiation, which causes rapid breakdown and loss in tensile properties. However this problem can be easily overcome by the correct use of stabilisation systems. Research work carried out over a long period of time has shown that even under extreme conditions of outdoor exposure (in Australia) it is still possible to stabilise the material in order that its lifetime is satisfactory. It was shown that samples exposed to these extreme conditions still retained in excess of 50% of their original tensile properties after a two year period of exposure (20). Thus for civil engineering applications long term UV stability should not be a problem. If care is taken with on-site storage and use, UV degradation should not be a problem with adequately stabilised fabrics.

(d) Other Environmental Degradation

Studies have shown that polypropylene is not attacked by chemicals or micro organisms commonly found in the environment. A detailed study of the effect of normal seawater and modified high alkaline seawater on woven polypropylene fabrics has been carried out (21). Results from this work show that, in the main, all fabrics retained more than 90% of their original strength after a one year period.

The above studies have shown that as far as woven polypropylene in civil engineering fabrics is concerned any environmental problems can be relatively easily overcome by a suitable choice of stabilisation system. However it must be remembered that such systems will add to the cost of the material.

6.2 Permeability

In general this will have little direct relevance to reinforcement and all that will normally be required of the geotextile is that it should not prevent the free passage of water while embedded in the fill. The woven geotextiles tested as part of the LIRA/QUB programme of research were adequate from this viewpoint. The minimum measured cross-plane permeability for these materials, using a simple 100 mm static head test, was 5 litres/m^2/sec.

6.3 Site Handling

Consideration should be given to the packaging and means of transport of geotextile rolls to the construction site. Once on-site the geotextile fabric should be given adequate storage since continuous exposure to the elements may cause weakening of the material as indicated elsewhere. Geotextile rolls are supplied with protective covers and these should not be discarded until the fabric is to be used.

Physical damage to the geotextile may be caused by the use of paper core tubes on which the geotextile is wrapped in the factory. If such paper cores are exposed to continuous wetting and drying they will degrade and possibly collapse if poorly stacked on-site. In particular, heavy rolls containing, say, 200 or more meters of geotextile should be provided with plastic cores which will allow mechanical handling methods to be used. It has been found that roll sizes greater than 100 m cannot be easily handled by site labour.

The geotextile manufacturers literature should be carefully studied with regard to storage conditions since reported damage may be subject to debate if poor site storage and handling methods are practiced.

For example during placement plant should not run directly on the geotextile. If cohesive fill is being used not less than 150 mm should be placed on the fabric before the passage of plant.

All geotextiles for reinforcement should be tough enough to withstand the rigours of site handling and placement. For example this includes the possibility of being torn by large angular stones occurring in the earthfill. Unfortunately it is difficult to devise a single test which would equate with this robustness requirement. Burst tests, grab tensile tests, wing tear tests, cone drop tests and wear tests have all been suggested. As in several other areas there is a need for generally agreed relevant tests for'robustness'.

7 CONCLUSIONS

Laboratory tests have shown, that woven polypropylene geotextiles have considerable potential as soil reinforcement. Fabric inclination was shown to be critically important.

A field trial has shown that the encouraging laboratory results could be reproduced at full scale. In addition the important influence of field compaction in initially stressing the fabric was shown to be important. This modifies to some extent the laboratory conclusions concerning inclination.

Application areas for woven polypropylene geotextile reinforcement (suggested specification below) which are currently feasible include edge strengthening and slope steepening of road and railway embankments, in waste or tailings embankments, and as'load spreaders' beneath low embankments on soft ground.

A suggested specification for woven polypropylene tape geotextiles is as follows:

		Weft	Warp
Tensile Strength	≮ :	40 kN/m	Similar
Tensile Modulus	:	600-1000 kN/m	Similar
Extension at Break (%)		6-12	Similar
Creep Coefficient (% log 10t)	≯	0.4	0.6
Cross Plane Permeability (litres/m^2/sec)	≮	5	
roll width	≮	5m	
roll length	≯	100m (if labour only)	

Notes concerning site handling and placing should be carefully followed and in particular the warp direction should be placed in the long axis of the embankment, thus ensuring that the weft is placed in the load-carrying direction.

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The Finite Element Method of Analysis Applied to Soil-Geotextile Systems
La méthode d'element fini d'analyse appliqué aux systèmes de sol géotextiles

The paper describes the nature of the elements used to represent soil-geotextile systems for the purpose of predicting their stress-strain behaviour. The stiffness matrices used to characterise each of these are given and the stress-strain laws adopted to represent the material properties are detailed. A finite element mesh so derived is then applied to the prediction of the behaviour of a footing resting on dense sand with or without a single layer of geotextile placed at different depths in various tests. It is shown that if the soil properties can be correctly represented in the soil elements, good correlations between predicted and measured data is obtained up to about 85 per cent of peak load. Beyond this, the finite element method is inappropriate as local failures in the soil occur which cannot be accommodated in the finite element procedures.

L'article donne une description du genre d'éléments employés pour représenter des systèmes de sol-géotextiles afin de prédire leur fonctionnement en ce qui concerne contrainte et deformation. On donne les matrices de rigidité employées pour caractériser chacune de ces systèmes et l'on expose en détail les lois de contrainte et deformation qui ont été adoptées. Un montage d'éléments finis ainsi trouvé est appliqué ensuite à la prédiction du fonctionnement d'une fondation qui repose sur du sable compact avec ou sans une seule couche de géotextile placée a des profondeurs différentes en divers essais. Cela demontre que si les propriétés du sol sont representées d'une manière exacte dans les éléments du sol, une bonne correlation est obtenue entre les données prédites et les données mesurées, jusqu'à 85% environ de charge maximum. Au-dessus de cela, la méthode 'élément fini' est peu appropriée parce qu'il y a la possibilité d'échecs locaux du sol qui ne sont pas inclus dans les procédures 'élément fini'.

INTRODUCTION

The behaviour of soil structures containing tension resistant inclusions such as geotextiles, depends not only on the properties of the soil and the inclusions but on the interaction of the soil and inclusions at interfaces. The success of any method of analysis for such a system will to a large extent be determined by its ability to represent these various material properties and interactions. At the present time, the finite element method approach involving discrete representation of the different constituents of the system offers a possible if somewhat complex method of analysis. Perhaps due to its complexity, few previous investigators have attempted to apply it to soil-geotextile systems (1, 2 and 3). In this paper, the work carried out at Strathclyde University, to develop a suitable finite element method approach for soil-geotextile systems which does allow such relative momement is described. Also its accuracy and limitations are identified by making comparisons between finite element method predictions and observations for laboratory scale prototypes of strip-footing loading systems.

FINITE ELEMENT REPRESENTATION OF SOIL-GEOTEXTILE SYSTEMS

The finite element procedure used is basically that for solving continuum load-displacement problems, (4, 5 and 6). It consists of three main steps: the discretization of the continuum; the derivation of element stiffness matrices and the analysis of the element assemblage.

Discretization of the Continuum:

In order to extend the finite element procedures for continuum problems to deal with soil-geotextile systems, particular types of elements have been developed to represent the soils, the geotextiles and the soil-geotextile interaction. These elements may be described as follows:

a) Soil elements: The soils are represented by triangular and quadrilateral elements, as shown in Fig. 1. The triangular elements are constant strain elements in which the displacement function is linear. The quadrilateral elements are considered to be formed from four triangular elements. Each triangular element stiffness is derived with two degrees of freedom allowed for any given point in the elements (6). The relationship between strains and small displacements is used (7) and the stiffness matrix is derived using the principle of minimum potential energy, which is a variational approach. The stiffness matrix may be given in the generalised form:

$$[Ke]\{\delta e\} = \{Pe\} \qquad \ldots\ldots(1)$$

where $[Ke]$ is the stiffness matrix of the elements, $\{\delta e\}$ is the vector of nodal displacements and $\{Pe\}$ is the vector of nodal forces.

The stiffnesses of the quadrilateral elements are derived by calculating the global stiffness of its four constituent triangular elements. By eliminating the degrees of freedom of the internal node of the quadrilateral elements, i.e. making use of a procedure called condensation (8), the stiffness matrices can be reduced to 8 x 8 matrix configurations without loss of accuracy.

b) Geotextile elements: The geotextiles are represented by straight line elements which have no bending stiffness and are of the form shown in Fig. 1. These elements can deform only in the axial direction and their displacement functions are linear (6). The stiffness matrix used is shown here in an incremental form:

$$\begin{Bmatrix} \Delta F_{x1} \\ \Delta F_{y1} \\ \Delta F_{x2} \\ \Delta F_{y2} \end{Bmatrix} = \frac{AE_t}{L} \begin{bmatrix} C^2 & SC & -C^2 & -SC \\ SC & S^2 & -SC & -S^2 \\ -C^2 & -SC & C^2 & -SC \\ SC & -S^2 & SC & S^2 \end{bmatrix} \begin{Bmatrix} \Delta U_1 \\ \Delta V_1 \\ \Delta U_2 \\ \Delta V_2 \end{Bmatrix} \quad \ldots\ldots(2)$$

which may be rewritten as:

$$\{\Delta F\} = \left[Ke\right] \{\Delta\delta\} \quad \ldots\ldots(3)$$

where $\{\Delta F\}$ is the vector of nodal force increments, $\{\Delta\delta e\}$ is the vector of nodal displacement increments and $[Ke]$ is the element stiffness matrix. Also E_t is the element instantaneous elastic modulus, A is its cross sectional area and L its length. S is sin α and C is cos α, where α is the inclination of the element to the x-axis.

c) Soil-geotextile interface elements: Soil-geotextile and soil boundary interactions are assumed to be purely frictional and simulated by spring elements of zero length connecting the nodes of soil elements to those of inclusion elements (9), as shown in Fig. 1, and its stiffness matrix may be written in incremental form as follows:

$$\begin{Bmatrix} \Delta F_{x1} \\ \Delta F_{y1} \\ \Delta F_{x2} \\ \Delta F_{y2} \end{Bmatrix} = \begin{bmatrix} K_h S^2+K_v C^2 & K_h SC-K_v SC & -K_h S^2-K_v C^2 & -K_h SC+K_v SC \\ & K_h C^2+K_v S^2 & -K_h SC+K_v SC & -K_h C^2-KS^2 \\ & & K_h S^2+K_v C^2 & K_h SC-K_v SC \\ (\text{symmetrical}) & & & K_h C^2+K_v S^2 \end{bmatrix} \begin{Bmatrix} \Delta U_1 \\ \Delta V_1 \\ \Delta U_2 \\ \Delta V_2 \end{Bmatrix} \quad \ldots\ldots(4)$$

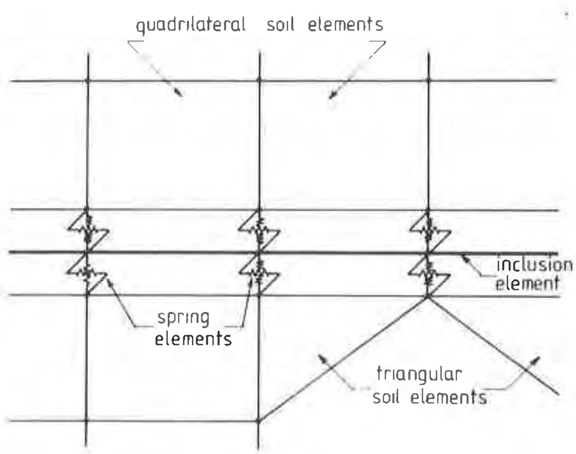

Fig. 1. Elements used in the analysis

which may be rewritten as:

$$\{\Delta F\} = \left[Ke\right] \{\Delta\delta e\} \quad \ldots\ldots(5)$$

where $\{\Delta F\}$ is the vector of nodal force increments, $\{\Delta\delta e\}$ is the vector of nodal displacement increments and $[Ke]$ is the element stiffness matrix. Also K_h is the spring stiffness in the plane of the geotextile and K_v is the spring stiffness perpendicular to it. Again S is sin α and C is cos α where α is the inclination of the element to the x-axis.

From a comparison of equations (2) and (4), it can be seen that the spring element stiffness can be constructed by superimposing the stiffness matrices of two line elements, one parallel to the inclusion and the other perpendicular to it. During the analysis, the spring element is permitted to elongate, or compress, but only in the direction parallel to the geotextile plane. This allows the development of relative sliding between the soil and the geotextile. Relative movement perpendicular to the geotextile is restricted by assigning a very large stiffness (K_v) to the spring element in this direction.

Derivation of Element Stiffness

To obtain the stiffness of each element the stress-strain behaviour of the various components and interfaces must be represented in mathematical form. To do this different mathematical techniques were adopted for the soils, the inclusions and their interfaces. Taking them in turn:

a) Stress-strain model of soil behaviour: To represent the soil, the non-linear elastic hyperbolic model is used (10, 11). It is also assumed that the initial tangent modulus (E_i) is related to the confining pressure (σ_3) as follows:

$$E_i = KPa\left(\frac{\sigma_3}{Pa}\right)^n \quad \ldots\ldots(6)$$

where Pa is atmospheric pressure, K and n are dimensionless constants, (12).

Thus the instantaneous tangent modulus can be expressed as

$$E_t = \left[1 - \frac{R_f\,(1-\sin\phi)(\sigma_1-\sigma_3)}{(2c\cos\phi + 2\sigma_3\sin\phi)}\right]^2 KPa\left(\frac{\sigma_3}{Pa}\right)^n \ldots\ldots(7)$$

where R_f is the ratio of deviator stress in the soil at failure to the ultimate deviator stress, c is the soil cohesion and ϕ is the angle of soil friction.

To include the effect of the curvature of the Mohr failure envelope in cohesionless soils, the following is added:

$$\phi = \phi_0 + \Delta\phi\,\log_{10}(\sigma_3/Pa) \quad \ldots\ldots(8)$$

where ϕ_0 is the value of ϕ for σ_3 equal to atmospheric pressure Pa, and $\Delta\phi$ is a material constant equal to the reduction in ϕ for a 10 fold increase in σ_3, (13).

The above equations provide all the information required for the evaluation of the instantaneous tangent modulus at any stress level and any confining stress during the incremental finite element analysis. To obtain the instantaneous Poisson's ratio, the expression used was:

$$\mu_t = \frac{G - F\log\left(\sigma_3/Pa\right)}{\left[1 - \dfrac{d(\sigma_1-\sigma_3)}{KPa\left(\frac{\sigma_3}{Pa}\right)^n\left[1 - \dfrac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]}\right]^2} \quad \ldots(9)$$

where G equals the value of initial Poisson's ratio μ_i

at one atmosphere, F equals the reduction in μ_i for a ten fold increase in σ_3 and d is a constant.

The value of Poisson's ratio cannot exceed 0.5 as it will violate the condition of positive strain energy, hence, volume expansion is not taken into account in this analysis. Also a Poisson's ratio of 0.5 cannot be used as it would lead to an infinite bulk modulus. To overcome this, when a value of 0.5 or more is encountered, a value of 0.49 is assigned to the element. To relate the stresses and strains in the soil, the incremental stress-strain equation proposed is:

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{Bmatrix} = \begin{bmatrix} Mb + Md & Mb - Md & 0 \\ Mb - Md & Mb + Md & 0 \\ 0 & 0 & Md \end{bmatrix} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{Bmatrix} \quad .....(10)$$

where $\Delta\sigma_x$ and $\Delta\sigma_y$ are the incremental stresses in the x and y directions respectively and $\Delta\tau_{xy}$ is the incremental shear stress. $\Delta\varepsilon_x$ and $\Delta\varepsilon_y$ are the incremental strains in the x and y directions respectively and $\Delta\gamma_{xy}$ is the incremental shear strain (14).

Also Md is the instantaneous shear modulus and

$$Md = \frac{E_t}{2(1 + \mu_t)} \quad .....(11)$$

with Mb the instantaneous bulk compressibility and

$$Mb = \frac{E_t}{2(1 + \mu_t)(1 - 2\mu_t)} \quad .....(12)$$

These modulii are used assuming Mb is a constant for a given confining pressure σ_3, but varies with σ_3 according to the corresponding initial tangent modulus E_i and initial Poisson's ratio μ_i (15). Thus Mb is calculated as follows:

$$Mb = \frac{E_i}{2(1 + \mu_i)(1 - 2\mu_i)} \quad .....(13)$$

With the above equations then the stiffness matrices for the soil elements may be evaluated.

b) Stress-strain model of geotextile behaviour: As the load extension relationships for geotextiles are usually non-linear, they cannot be accurately simulated by linear relationships. Polynomial functions have therefore to be used and the function that is chosen has the form:

$$T = A\sigma = a_1\varepsilon + a_2\varepsilon^2 + a_3\varepsilon^3 ... \quad .....(14)$$

where T is the axial load applied to the geotextile, A is the initial cross sectional area, σ is the "apparent axial stress", calculated by assuming the geotextile does not change its cross-sectional area during axial straining, ε is the axial strain and a_1, a_2, a_3... are polynomial constants.

By differentiating equation (14) with respect to ε, the product of the instantaneous tangent modulus E_t and the initial cross sectional area A, can be directly evaluated from strains as follows:

$$\frac{dT}{d\varepsilon} = \frac{d(A\sigma)}{d\varepsilon} = \frac{Ad\sigma}{d\varepsilon} = AE_t = a_1 + 2a_2\varepsilon + 3a_3\varepsilon^2 \quad ...(15)$$

The product AE_t is then used directly to evaluate the stiffness matrices of the geotextile elements as indicated in equation (2). The constants a_1, a_2, a_3... are determined by the least square curve fitting technique of measured load-extension curves (16)

c) Shear stress-deformation model of soil-geotextile interface friction: A hyperbolic model is adopted to simulate the interface friction behaviour of soil-geotextile and soil in contact with boundaries. The model used is as follows:

$$\tau = \frac{\delta r}{b_1 + b_2\delta r} \quad .....(16)$$

where τ is the shear stress and δr is the relative displacement at the interface for a given normal stress σ_n (15). Also b_1 and b_2 are constants equal to $(1/k_i)$ and $(1/\tau_{ult})$ respectively, where k_i is the initial tangent stiffness per unit area and τ_{ult} is the asymptotic value of the shear stress at infinite displacement of the hyperbolic curve. The initial tangent stiffness per unit area (k_i) is assumed to be related to the normal stress (σ_n) by the following expression:

$$k_i = K_1 Pa\left(\frac{\sigma_n}{Pa}\right)^{n_1} \quad .....(17)$$

where K_1 and n_1 are constants.

In a manner similar to that for the instantaneous tangent modulus of soil, the instantaneous tangent stiffness of the interface can be expressed as

$$k_t = \left[1 - \frac{Rf_1\tau}{\sigma_n \tan\delta}\right]^2 K_1 Pa\left(\frac{\sigma_n}{Pa}\right)^{n_1} \quad .....(18)$$

where Rf_1 is the ratio of the shear stress at failure to the ultimate shear stress on the interface and δ is the angle of friction between the soil and the geotextile. The instantaneous stiffness of the spring element (K_t) then becomes:

$$K_t = k_t - A \quad .....(19)$$

where A is the field area of the element.

Analysis of the Element Assemblage

After evaluating the stiffness matrices for the individual elements, the stiffness matrix for the whole system is assembled. The basis of the assembly is that the value of the displacement at a node is the same for each element sharing the node (5). The assembly process results in a set of simultaneous equations of the same form as the equations for the individual elements, as follows:

$$\left[K_g\right]\{\delta\} = \{P\} \quad .....(20)$$

where $\left[K_g\right]$ is the global stiffness matrix, $\{\delta\}$ is the vector of all the nodal displacements and $\{P\}$ is the vector of the nodal forces.

The next step is to modify equation (20) to account for boundary conditions which may include zero or non-zero displacements. This is followed by the solution of the simultaneous equations for the unknown displacements. Finally, from the now known nodal displacements, the strains and stresses for each element are calculated to yield the complete solution.

In its present form the program can only deal with systems in which the initial stresses are either known or are calculated from the at-rest condition. Increments of load are then applied and the instantaneous tangent modulii for each increment of load are evaluated according to the position reached on the stress-strain curve. This is achieved using a two stage load cycle technique based on the mid-point integration method, (5).

SYSTEMS AMD MATERIALS MODELLED

Several soil-geotextile systems have now been modelled at Strathclyde University using the finite element procedure including the prediction of the measured behaviour of laboratory scale embankments (3). In these embankments the deformations and stresses in the system were relatively small and good correlation between measured and predicted behaviours were obtained.

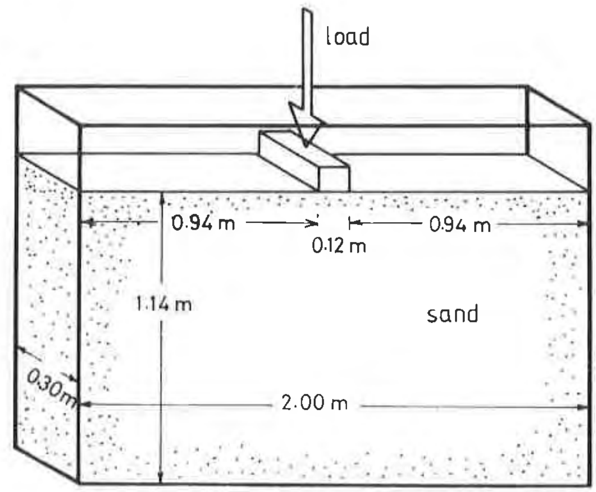

Fig. 2. Layout of apparatus

In this paper, comparisons are made between measured and predicted behaviour of laboratory scale soil-geotextile strip-footing loading tests which have been taken to failure and therefore involve large deformations and stresses. Prior to making these comparisons the nature of the system tested and the materials used are described.

System Tested:

The strip-footing tests were conducted in a large tank with 12.5 mm thick glass sides within a rigid steel frame. The principal dimensions and layout of the apparatus are shown in Fig. 2. The strip-footing used was 120 mm wide and made of rigid smooth steel plates. The load was applied to the footing by means of a motorised 10 tonne capacity screw jack at a constant rate of [illegible] per hour.

The sand used in both tests was Leighton Buzzard sand which is a sub-rounded, mainly quartzite sand with a particle size range of 0.3 - 2.0 mm and mean diameter of 0.85 mm. The uniformity coefficient was 1.22. It was placed in a dense state with a porosity of 34 per cent by an air activated spreader (17). The soil was tested under drained triaxial test conditions and the values of the hyperbolic parameters determined from these tests are shown in Table 1.

TABLE 1. HYPERBOLIC PARAMETERS FOR LEIGHTON BUZZARD SAND

Parameters derived from triaxial tests									
K	K_{ur}	n	c	$\emptyset_o$	$\Delta\emptyset$	R_f	G	F	d
1573	1890	1.05	0	42.7°	3.2°	0.9	0.46	0.17	24
Parameters assumed and used in analysing the strip footing model tests									
K	K_{ur}	n	c	$\emptyset_o$	$\Delta\emptyset$	R_f	G	F	d
1573	1890	1.05	0	49°	0	0.9	0.45	0	0

The geotextile used as the inclusion was a non-woven melt-bonded material made from 67 per cent polypropylene and 33 per cent polyethylene, manufactured by ICI Fibres and known as "Terram 1000". It possessed a mass per unit area of 140 g/m^2 and average thickness of 0.7 mm. The load-extension properties of the geotextile were established in the in-soil test apparatus (18). In the strain range and normal confining stress range applied in these tests, it was considered that a single average curve could be used to describe the load deformation behaviour of the geotextile. The polynomial coefficients used to model the curve are as shown in Table II.

TABLE II. POLYNOMIAL COEFFICIENTS FOR TERRAM 1000

a_1	a_2	a_3	a_4	a_5
76.4	-1347.4	20804	-187422	982038
a_6	a_7	a_8	a_9	a_{10}
-2965793	47181650	-3183505	0	0

Shear box tests were carried out to determine the interface properties between the geotextile and the sand and the footing and the sand. In these the geotextile and the steel and the footings were placed in the bottom half of the shear box level with the plane of sliding with the top half. The top half was filled with dense sand at a porosity of 34 per cent. The parameters obtained from these tests which were used in the analysis are given in Table III.

TABLE III. HYPERBOLIC PARAMETERS FOR INTERFACE FRICTION

Leighton Buzzard sand in contact with Terram 1000			
K_I	n_I	R_{f_I}	δ
1300	0.30	0.8	36.3°
Leighton Buzzard sand in contact with footing base			
K_I	n_I	R_{f_I}	δ
800	0.05	0.49	11.3°

PREDICTED AND MEASURED STRIP FOOTING LOADS

Sand Alone:

To prove the reproducibility of test data from the apparatus, two tests were carried out on the footing placed on sand alone. As can be seen in Fig. 3 in dimensionless form, the load settlement relationship for these tests is typical of that for a strip footing resting on dense sand and shows very little variation between tests. Back analysis of the peak bearing pressures using classical bearing capacity theories suggests that the operational angle of friction for the sand lies between 48° and 50°. Now the level of stressing of the soil is much greater in these tests than it was in the embankment model tests previously reported (3), thus side friction in the apparatus is bound to be much greater. This would lead to an apparently higher value of the operational angle of friction. Also, at strain levels in the sand approaching peak stresses, the differences between the behaviour in plane strain and triaxial test conditions becomes more marked than at low strain levels. In fact it has been suggested that the angle of friction to be used in plane strain bearing capacity calculations should be 10 per cent higher than that measured in triaxial tests in order to account for this, (19, 20). A combination of these factors could therefore well explain the average 12 per cent difference between measured triaxial values and the back analysed values from footing test data.

The finite element mesh used to model the strip footing on sand alone consisted of 285 rectangular soil elements each formed of four constant strain triangular elements with, on this occasion, only 5 spring elements

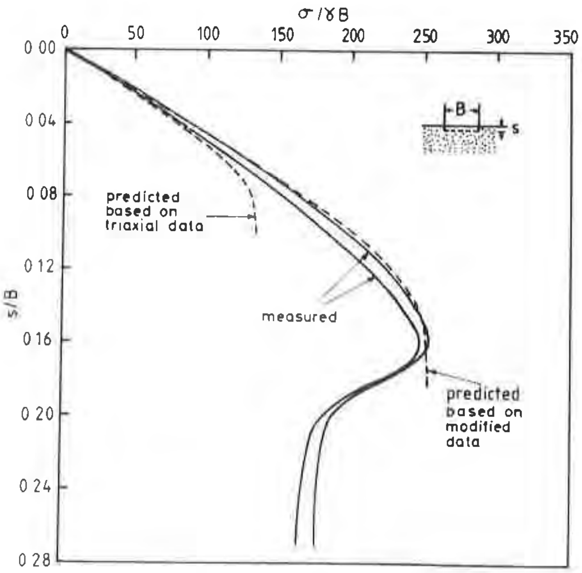

Fig. 3. Sand alone - bearing pressure vs settlement (dimensionless)

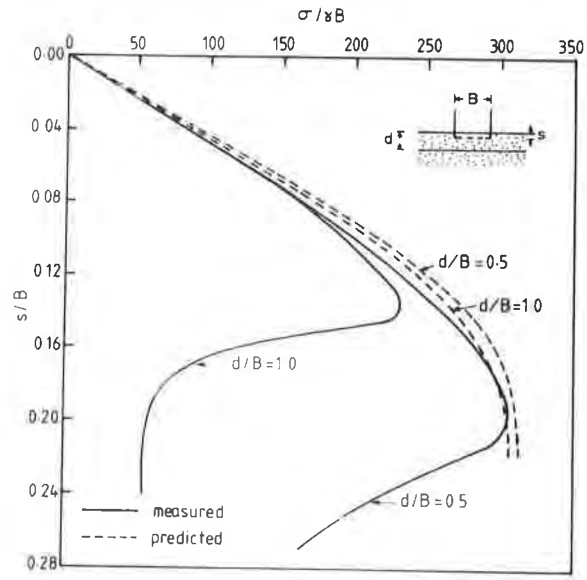

Fig. 4. Sand with inclusion - bearing pressure vs settlement (dimensionless).

used to represent the soil-footing interface. The loading was applied in steps which corresponded to a constant displacement increment of 1 mm. Geometrical changes of the elements during strain were ignored.

Using the parameters indicated in Tables I, II and II, the predicted load settlement relationships for the strip footing resting on the sand were as shown in Fig. 3. This indicates a peak footing stress approximately half that measured although the initial load settlement relationship is in reasonably good agreement. This indicates that the value of angle of friction used for the sand is most probably the parameter in error. To overcome this and to simplify the analysis somewhat, it was decided to adopt a constant value for the angle of friction of the sand equal to the average back analysed test data value of 49°. Also to provide less erratic patterns of stresses beneath the footing, a constant Poisson's ratio value of 0.45 was adopted. With these changes, the predicted load-settlement curve became much more accurate, as shown in Fig. 3, although it did not indicate the measured reduction in bearing capacity at post peak strains. This is to be expected as strain softening is not included in the finite element analysis.

Sand with a Single Geotextile Inclusion Layer:

Tests were conducted on the strip-footing resting on sand containing a single layer of geotextile placed at different depths as it was known from small scale tests that the depth of the geotextile was an important factor, (21). Predictions were made of the behaviour of these systems using the finite element procedure. The mesh used was the same as for the sand alone but with the addition of 14 line elements to represent the geotextile and 30 spring elements to represent the soil-geotextile interfaces. The reorientation of the line elements during straining was allowed for. The parameters used to model the materials and their interfaces were the modified parameters used for the tests on soil alone.

The measured and predicted data for two cases are given in Fig. 4. They show that for the case where the geotextile was placed at a depth equal to half the breadth of the footing (0.5 B), the measured and predicted behaviours were within 10 per cent up to peak footing load. For the case where the footing was placed at a depth equal to the breadth of the footing (1.0 B), the measured and predicted behaviours were again within 10 per cent but only up to 85 per cent of the measured peak load. Beyond this, the predicted behaviour rapidly diverges from the measured behaviour and grossly overestimates the bearing capacity of the footing. Examination of the strain fields obtained from measured displacement data shows that sliding along the geotextile occurred in this test at strains approaching peak loads and that locally the soil above and in close proximity to the geotextile was subject to large strains. Thus post peak strain softening was probably occurring in this region and this is not allowed for in the finite element analysis.

It is very interesting to note from the measured and predicted data that the influence of the geotextile on the load-settlement behaviour of the strip footing was very limited up to settlements equal to approximately 8 per cent of the footing breadth. This suggests that up to that level of settlement, strains in the soil were insufficient to mobilise significant tensile load in the geotextile.

DISCUSSION

The finite element method of analysis involving discrete representation of the different constituents within soil-geotextile systems has been described in this paper and shown to be a valuable analytical method. It has been found to be limited to the analysis of the behaviour of soil-geotextile systems prior to failure developing in any of their constituent materials, even locally. Whenever such a failure developed, the predicted behaviour was found to rapidly diverge from the actual behaviour since the mathematical laws used in the analysis to represent the stress-strain behaviour of the constituents do not allow for any strain softening that occurs. The finite element procedure does, nevertheless, provide up to this failure stage, a good prediction of system behaviour.

The principal difficulty of representing the materials behaviour is with the soil. It appears from the work so far undertaken that triaxial test data can underestimate the strength of soils in plane strain. Thus in the analysis reported in this paper it was found necessary to increase the value used in the analysis by 8 to 10 per cent over that from triaxial tests to allow for this. A further 4 to 2 per cent was added to allow for side friction in the plane strain test apparatus, to give an average overall increase of 12 per cent.

Although the finite element procedure described in this paper is limited to pre-failure conditions, these are likely to be appropriate to the range of operational stress and strains in many soil-geotextile systems. Thus this method is likely to be a valuable means of testing the significance of varying different soil and geotextile properties. Also it will allow the significance of varying construction procedures to be established. Much development is still required for this finite element procedure but the results so far obtained suggest that it will be worthwhile pursuing this analytical method.

ACKNOWLEDGEMENTS

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Stress Reduction in Flexible Culverts Due to Overlays of Geofabric

Diminution des contraintes dans les conduits souterrains souples grace à un recouvrement de géotextile

Increasing number of flexible metal culverts of different geometries and shapes are being used successfully to bridge across streams and canals. These flexible metal culverts derive a considerable portion of their load carrying capacity through interaction with the surrounding backfill. By including layers of geofabric, the effectiveness of these backfill materials can be improved and the stresses in the culvert can be reduced. This paper reports the results of finite element analyses of a box culvert to provide a quantitative assessment of the reduction in stresses due to the inclusion of geofabrics. The nonlinear and stress-dependent stress-strain behavior of the backfill and the actual sequence of construction operations are taken into consideration in the analyses. The bending moments, axial forces, and deflections in the structure with and without the use of the geofabric are compared for the performance of the structure under the weight of backfill and traffic loads.

De plus en plus de ponceaux métalliques fléxibles de différentes formes et géometries remplacent avec succès des structures plus rigides. Ces ponceaux métalliques flexibles obtiennent une grande partie de leur capacité partante de l'interaction avec le remblai avoisinnant. L'inclusion de couches de geotextiles augmente l'efficacité du remblai et reduit les contraintes induites dans le ponceau. Cet article présente les résultats d'analyses d'éléments finis d'un ponceau fermé et une évaluation quantitative de la réduction des contraintes due à l'inclusion de geotextiles dans le remblai. La relation contrainte-dèformation (non-lineáire et dépendante des contraintes) et les étapes de constructions sont utiliseés dans les analyses. Les moments de flexion, forces axiales et déformations du systéme soumis avec charges de remblai et de circulation automobile sont comparés pour les cas avec et sans géotextiles.

INTRODUCTION

Geofabrics are being used successfully for many engineering applications and the details are given in (1, 2, 3, 13, 14, 17, 18, 19, 20). Soil reinforcement is one of the more common applications of geofabrics. There have been extensive research on the behavior of soil-geofabric reinforcement and the extent to which the geofabric-soil interaction will improve the load carrying capacity of the soil is well understood. Although there have been a tremendous increase in the use of flexible metal culverts in highway bridge projects, no attempt has been made up to date to study the desirable effects of overlays of geofabric placed in the backfill used around and over the culvert. These corrugated culverts derive a considerable portion of their stiffness and load-carrying capacity through interaction with the surrounding backfill. Therefore the quality of the backfill used around and over the structure will determine the performance of these culverts under backfill and live loads. By using several layers of geofabric, embedded in the surrounding backfill, the effective modulus of the backfill can be increased by a substantial amount and the stresses induced in the culvert can be reduced. If the stresses in the culvert can be reduced by an appreciable amount, much thinner culvert structural sections can be employed and a greater economy in use of corrugated flexible metal culverts can be achieved. Because of the complex nature of the soil-structure-geofabric interaction, design of these structures thus requires a method of analysis which is capable of taking this interaction into account, so that the loads in the structure due to both backfill and traffic may be determined. In this paper, the finite element method of analysis is used to study the effects of the presence of geofabric on the loads carried by the culvert. Several series of analyses were performed to provide a quantitative assessment of the reduction in stresses in the culvert structure for varying properties of fabric and loading conditions. The increases in the factor of safety, against development of plastic hinges in the culvert section, due to the use of geofabric were calculated using the bending moments and the axial forces determined from the finite element analyses. The deflections of the structure with and without the use of the geofabric were also calculated as part of this study.

GEOMETRY AND FINITE ELEMENT MESH

A cross-section through the backfill and the aluminum box culvert used in this study is shown in Fig. 1, and the finite element mesh used for the analyses is shown in Fig. 2. The culvert was modeled by linearly elastic beam elements and the geofabric was included in the analyses using a series of linearly elastic bar elements with no compressive strength. The backfill was modeled by two-dimensional isoparametric elements. A no-slip condition was simulated at the interfaces between the culvert and the backfill and between the geofabric and the backfill.

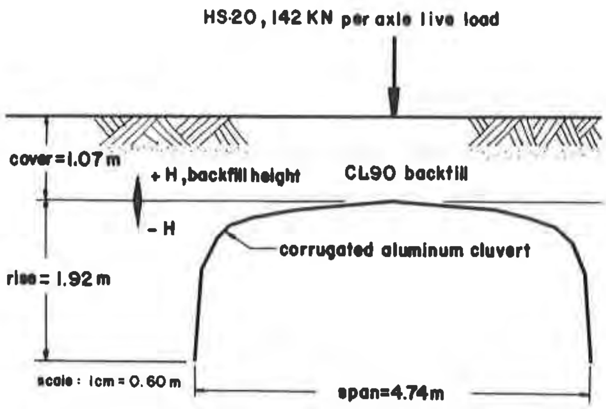


Figure 1. Cross-Section of Box Culvert.

MATERIAL PROPERTIES

The properties of the low plastic clay backfill employed in the analyses are listed in Table 1. These properties correspond to clay compacted to 90% of the maximum dry density as determined by the Standard AASHTO compaction test. The properties of the aluminum structural plate and the structural plate with angle stiffners used respectively in the side section, haunch and crown sections of the culvert are listed in Table 2. The properties of the geofabric used in the analyses are summarized in Table 3.

REPRESENTATION OF TRAFFIC LOADS

Two-dimensional analyses of the type discussed in this paper, represent a slice of unit thickness through the culvert and the backfill. In these analyses it is assumed that the slice analyzed is representative of any section along the length of the structure, and that

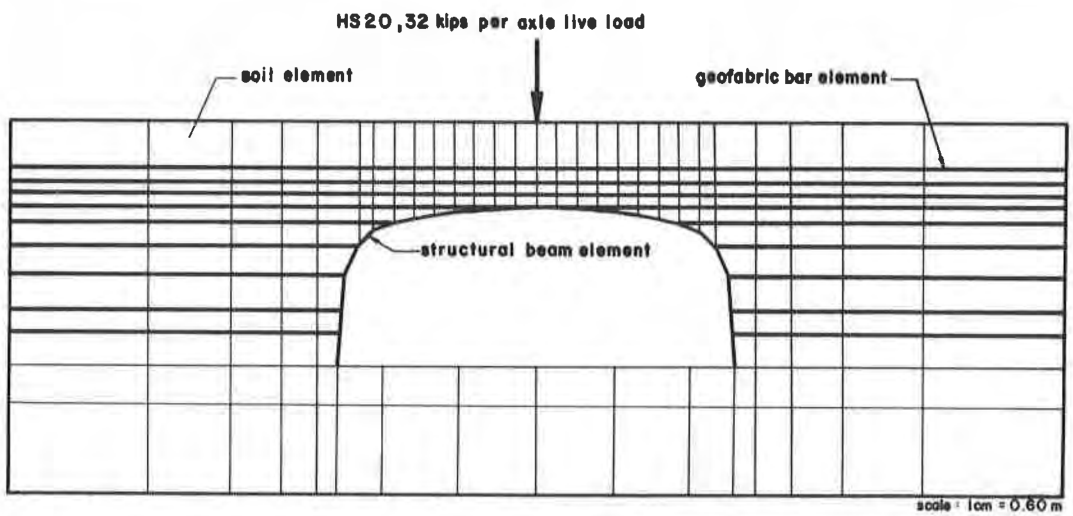


Figure 2. Finite Element Mesh

ANALYSIS PROCEDURE

The interactions between the flexible metal culvert, the surrounding backfill, and the overlays of geofabric were studied using the finite element analysis procedures. The finite element method has been successfully applied to culvert structures including many box culverts of the type chosen for the present study (5, 6, 8, 9, 10, 11, 12, 15). The analyses described in this paper were performed using a hyperbolic stress-strain relationship for the backfill material. This relationship, which is described in detail in (4, 7), model the nonlinear stress dependent stress-strain behavior of soils incrementally, by varying the values of Young's modulus and bulk modulus in each element according to the previously calculated stresses. The analyses herein were performed incrementally, simulating the field construction operations. The backfill was placed around and over the structure one layer at a time, followed by one overlay of geofabric at a time as shown in Fig. 2. The analyses were performed with and without the placement of overlays of geofabric and an AASHTO HS-20 live load was simulated subsequent to the completion of construction,

Table 1. Properties of Backfill Material used in Finite Element Analyses

Property	Values Used
Unit weight, γ (kN/m^3)	1.8
Cohesion, C (kN/m^2)	2.9
Modulus Number, k	90
Modulus Exponent, n	0.45
Bulk Modulus Number, k_b	80
Bulk Modulus Exponent, m	0.2
Failure Ratio, R_f	0.7
Angle of Internal Friction, ϕ (degrees)	30

Table 2. Properties of Culvert Sections used in Finite Element Analyses

Property	Values used for Side Section	Values used for Haunch Section	Values used for Crown Section
Cross Sectional Area (m^2/m)	0.0558	0.0100	0.0091
Moment of Inertia (m^4/m)	0.0239×10^{-4}	0.1448×10^{-4}	0.1646×10^{-4}
Plastic Moment M_p (kNm/m)	16.5	57.4	61.9
Plastic Axial Force P_p (kN/m)	858.2	2023.0	1757.4
Young's Modulus (kPa)	70,355,520		

Table 3. Properties of Geofabric used in Finite Element Analyses

Property	Values Used
Cross Sectional Area (m^2/m)	0.0030
Young's Modulus (kPa)	4800 to 4800000

loads are continuous along the length of the structure. Therefore, to perform two-dimensional analyses of live load effects, it is necessary to represent the actual traffic load by an equivalent line loading which is continuous along the length of the structure. Therefore, for the chosen HS-20 AASHTO permit load vehicle configuration and wheel loads, it was necessary to determine the equivalent line loading which produces the same peak vertical stress at the crown of the structure using an elastic theory of stress distributions. As shown in Fig. 3 and Table 4, the magnitudes of these equivalent line loads decrease as the depth of soil cover over the crown increases, due to the spreading of the vehicle loads along the axis of the structure.

Table 4. Equivalent Line Loads for HS-20 142.4 kN Single Axle Permit Load

Depth of Cover (meter)	Equivalent Line Load (kN/m)
0.5	75.0
1.0	48.8
1.5	37.5
2	32.5
3	27.7
4	21.5
5	18.4
7	14.2
9	11.3

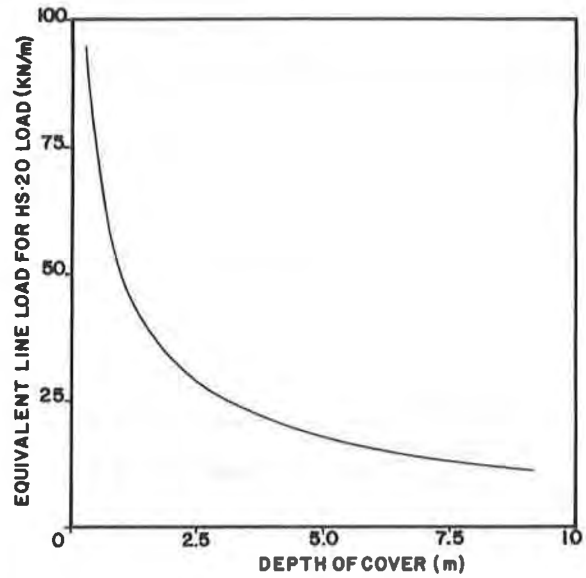


Figure 3. Variation of Line Load With Cover Depth

BENDING MOMENTS

The magnitudes and distributions of bending moments in the box culvert were determined with and without the overlays of geofabric under both backfill and traffic loads as shown in Figs. 4 and 5. The modulus-area (EA) of the fabric used in the analyses was varied through several orders of magnitude and the variations of maximum crown and haunch bending moments with EA are shown in Fig. 6. The bending moments determined with the fabric of EA = 14600 kN/m are listed in Table 5 with those obtained without the fabric. The reductions in bending moment due to the use of geofabric are in the range of 11 to 24%.

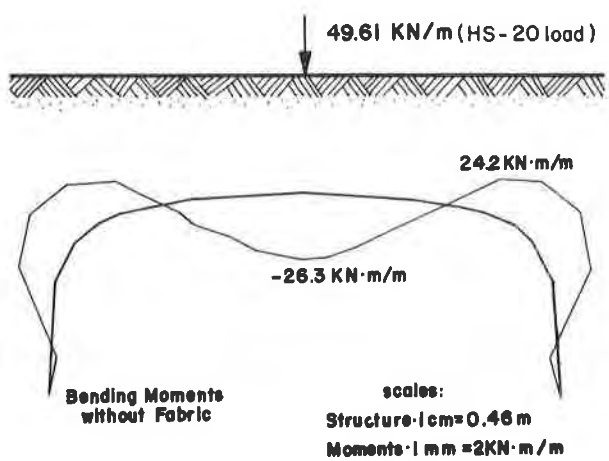


Figure 4. Bending Moment Distribution.

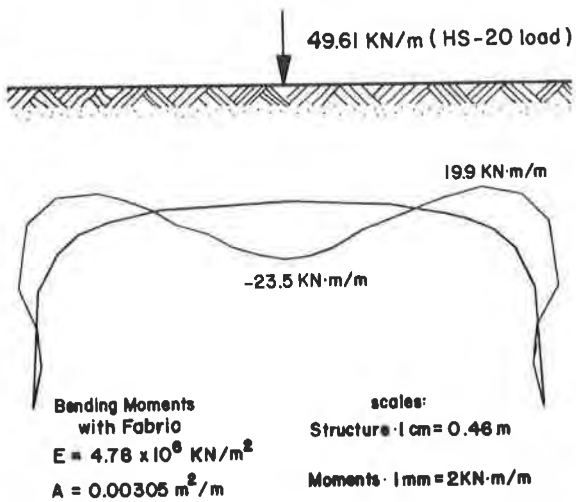


Figure 5. Bending Moment Distribution

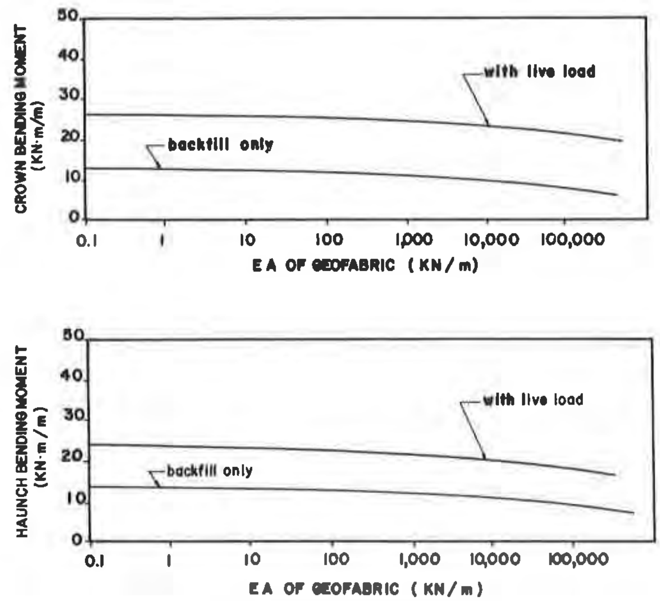


Figure 6. Variation of Bending Moment with Fabric Properties.

Table 5. Bending Moments

Location	Moments Due to Backfill only (kNm/m)		Moments Due to Backfill & Live Load (kNm/m)	
	w/o Fabric	w/Fabric	w/o Fabric	w/Fabric
Crown	12.8 (100%)	9.7 (76%)	26.3 (100%)	23.5 (89%)
Haunch	13.8 (100%)	10.6 (77%)	24.2 (100%)	19.9 (82%)

AXIAL FORCES

The distributions of axial force calculated for the geofabric with EA = 14600 kn/m under backfill and traffic loads are shown in Fig. 7 with those calculated without the fabric. The variations of maximum crown and haunch axial forces with EA of the fabric are shown in Fig. 7. Although the axial forces increased with the use of geofabric, these were sufficiently small in all cases, that they had a negligible influence on the factor of safety with respect to development of a plastic hing (Fp). The increases in axial force due to the use of geofabric with an EA = 14600 kn/m, are listed in Table 6. These increases are in the range of 13 to 83%.

Table 6. Axial Forces

Location	Axial Forces Due to Backfill only (kN/m)		Axial Forces Due to Backfill & Live Load (kN/m)	
	w/o Fabric	w/Fabric	w/o Fabric	w/Fabric
Crown	46.7 (100%)	57.0 (122%)	46.0 (100%)	84.6 (183%)
Haunch	65.4 (100%)	73.8 (113%)	94.6 (100%)	114.1 (121%)

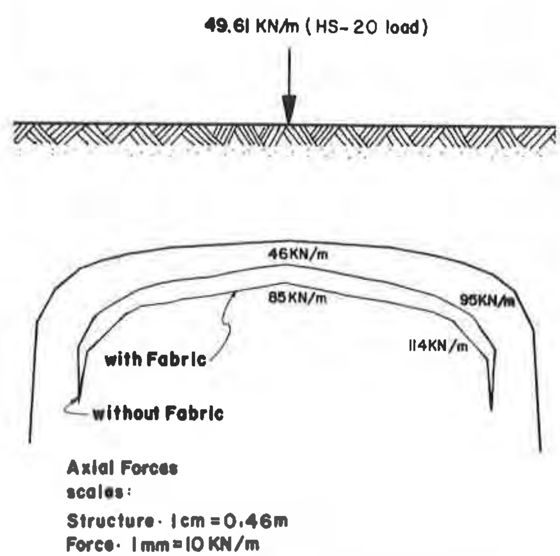


Figure 7. Axial Force Distribution.

FACTORS OF SAFETY

The factor of safety (F_p) against development of a plastic hinge may be calculated using the following equation, which considers the effects of both bending moments and axial forces in the culvert:

$$F_p = 0.5\frac{P_p}{P}\left\{\sqrt{\left(\frac{M}{M_p}\right)^2\left(\frac{P_p}{P}\right)^2 + 4} - \left(\frac{M}{M_p}\frac{P_p}{P}\right)\right\} \qquad (1)$$

in which P = axial force (kn/m)
P_p = plastic axial capacity (kN/m)

M = bending moment (kNm/m)
M_p = plastic moment capacity (kNm/m).

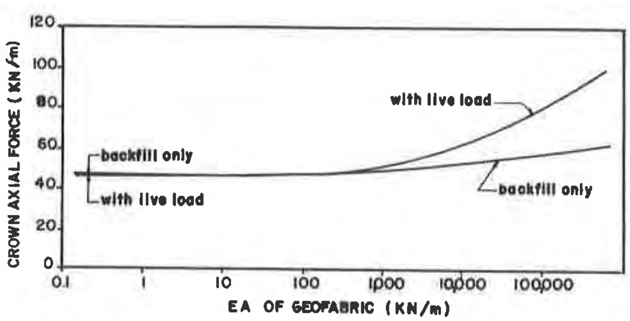
Using the bending moments and the axial forces listed in Tables 5 and 6, values of F_p were calculated for the crown and haunch sections of the culvert. These values are listed in Table 7 for the geofabric with an EA of 14600 kN/m. The increases in factor of safety due to the use of geofabric are in the range of 11 to 2C%.

Table 7. Factors of Safety Against Formation of Plastic Hinges

Location	For Backfill Loads		For Backfill and Live Loads	
	w/o Fabric	w/Fabric	w/o Fabric	w/Fabric
Crown	4.69	5.98	2.34	2.59
Haunch	4.01	4.99	2.34	2.81

CROWN DEFLECTIONS

Crown deflections due to backfilling are shown in Fig. 9 for both with and without geofabric. The calculated deflections for backfilling and live loads are listed in Table 8. The reductions in crown deflection due to the use of geofabric are in the range of 26 to 31%.



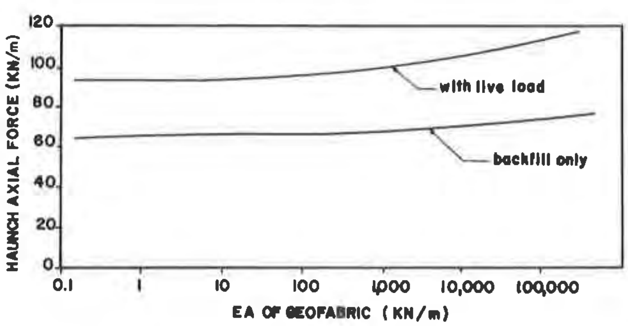


Figure 8. Variation of Axial Force with Fabric Properties

Table 8. Summary of Crown Deflections

	Deflections Due to Backfilling	Deflections Due to Backfilling and Live Loads
Without Fabric	14.5 mm (100%)	26.9 mm (100%)
With Fabric	10.0 (69%)	20.0 mm (74%)

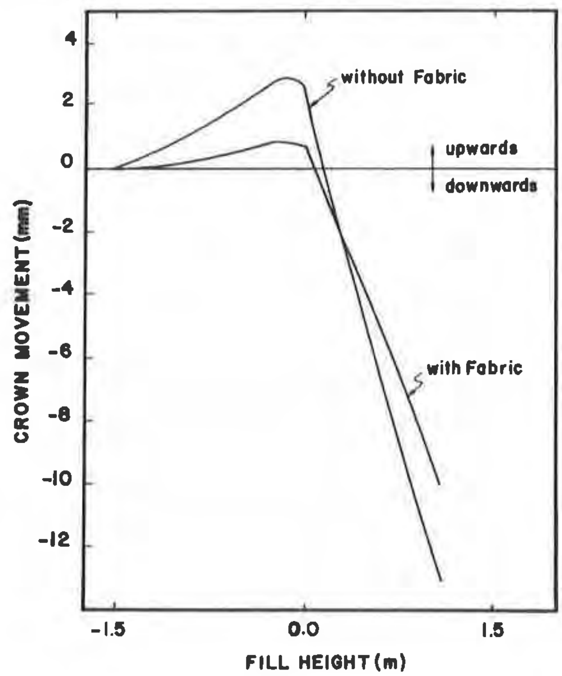


Figure 9. Crown Deflections During Backfilling.

CONCULSIONS

This study of the behavior of an aluminum corrugated box culvert under backfill and live loads clearly indicates the potential use of overlays of geofabric to increase the factors of safety and reduce the crown deflections. The factors of safety against development of plastic hinges increased by about 25% and the crown deflections decreased by about 30% for the cases analyzed in this study. This shows that the box culvert can successfully perform with thinner structural sections when geofabric overlays are used. This flexible culvert-soil-geofabric interactive design would lead to a greater economy in the use of flexible metal culverts for bridges.

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Fabric Reinforcement of Embankments and Cuttings

Renforcement des remblais et des déblais à l'aide de textiles

A method is described for enhancing the stability of cutting and embankment slopes employing fabric reinforcement. A feature of the method is that the load-extension properties of the fabric are linked in the design equations ensuring that the factor of safety in terms of serviceability is automatically satisfied.

The equations described have been applied to an example of an embankment constructed using a cohesive fill and also to the design of a motorway cutting reinstatement which recently failed in England. It was demonstrated that the repair of the cutting using reinforced soil reduced the costs by about 40 per cent when compared to conventional repair techniques.

L'auteur décrit une méthod d'amélioration de la stabilité des talus de déblai et de remblai par renforcement avec des textiles. Un aspect de cette méthod est que les caractéristiques d'extension du textile sous charge sont reliées entre elles dans les équations de dimensionnement, garantissant que le coéfficient de sécurité en termes de durée de service est automatiquement satisfait.

Les équations décrites ont été appliquées à un exemple de remblai en matériau cohérent et au dimensionnement de la réfection d'un déblai d'autoroute qui a récemment subi une rupture en Angleterre. On a démontré que la réparation du déblai avec de la terre armée réduit les coûts d'environ 40 % par comparaison avec les méthodes de réparation classiques.

INTRODUCTION

A major cost factor in the repair of slip failures in cuttings and embankments is associated with the haulage distances involved in the removal and importation of fill. In this paper a procedure is described which involved the re-use of the foundered soil by employing fabric layers to act as reinforcement. Such an approach can offer considerable savings when haulage distances are significant. Moreover, the calculation procedures described may also be utilised for designing fabric-reinforced embankments with steeper side slopes than would be possible for their unreinforced counterparts. Thus greater economies can be achieved by using locally available soils and by reducing the amount of fill and land required for constructing the embankment.

The method of calculation takes account of both adherence and tensile resistance of the fabric. The tensile resistance is based on a criterion of specified deformation rather than on the ultimate load characteristics of the fabric. An estimate of the deformations induced by construction plant can also be made.

The application of the method to the repair of a failed section of motorway cutting in England is briefly described.

Theoretical considerations

A simple bilineal slip plane is assumed to represent the failure surface (Fig 1). It is usually possible to make a reasonable representation of an actual failure surface by this approach. Two classes of problem are considered:

(i) Both failure surfaces emerge on the slope, representing most cutting situations.

(ii) The upper failure surface emerges on a horizontal plateau, as frequently occurs with embankment failures.

In a paper currently in press it was shown that resistance (R_T) against failure was given by:

$$R_T = \frac{1}{F}\left[\int\left(\gamma\, z \cos^2\beta\, (\tan\beta - \tan\theta)^2 - ru/\cos^2\theta\right\} \tan\phi^1 + c^1)\, dL + \Sigma\, T_z \sin\theta \tan\phi^1\right] \quad \ldots\ldots (1)$$

and that the total disturbance force (D_T) was given by:

$$D_T = \int \gamma\, z \sin\theta \cos\theta \left[1 + K \cos^2\beta \left\{\tan^2\beta - 1 + \frac{\tan\beta}{\tan\theta}(1 - \tan^2\theta)\right\}\right] dL - \Sigma\, T_z \cos\theta \quad \ldots\ldots (2)$$

The expressions under the integral signs $[\int]$ are taken along the full length of slip surface while the expressions under the summation sign (Σ) relates to the contributions made by the fabric layers in either tension or adherence. Clearly the lesser of these two conditions should be employed in the design. (The definitions of the symbols are provided in Appendix 1.)

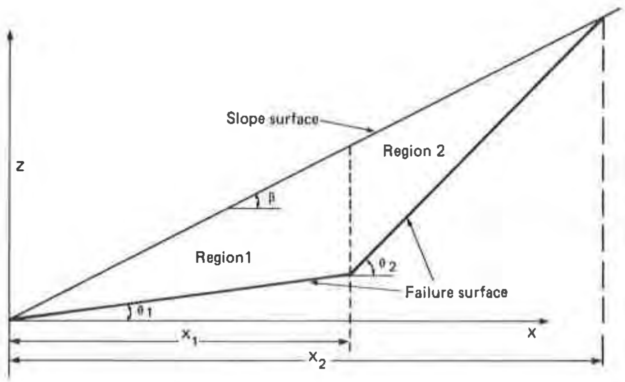

Fig. 1a Geometry of problem when both failure surfaces emerge on slope face

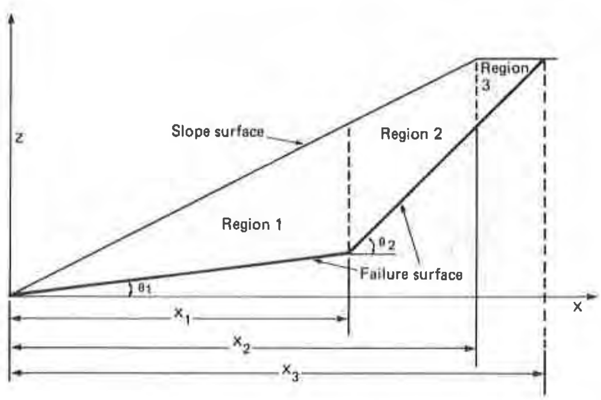

Fig. 1b Geometry of problem when one failure surface emerges on plateau above slope

To avoid excessive deformation of a fabric reinforced slope, it may often be necessary to limit the mobilised tensions in the fabric to only a small proportion of its ultimate strength. This requirement can lead to cumbersome design problems as the tensions developed cannot usually be assessed until after an initial design has been obtained and several further attempts may be necessary before a satisfactory solution is achieved. An alternative approach, which overcomes these difficulties, introduces the deformation criteria directly into the design formula, thus ensuring that the tension mobilised will be consistent with the deformation requirement. A further advantage is that the solution for both tension and adherence is obtained as a single computation.

It has been demonstrated (1) that the load-deformation characteristics of fabrics are often of hyperbolic form. The simplest hyperbolic expression relating load (T) to extension (e) is of the following form:

$$T = \frac{e}{me + c} \qquad \text{..... (3a)}$$

$$e = \frac{cT}{(1 - mT)} \qquad \text{..... (3b)}$$

These expressions imply that a linear relation will be obtained if e/T is plotted against e.

The analysis has been carried out on the basis of average values and no account has been taken of time-dependant creep strains. The horizontal stress (σ_x) acting on an element of soil adjacent to a slip surface (Fig 2) is given by:

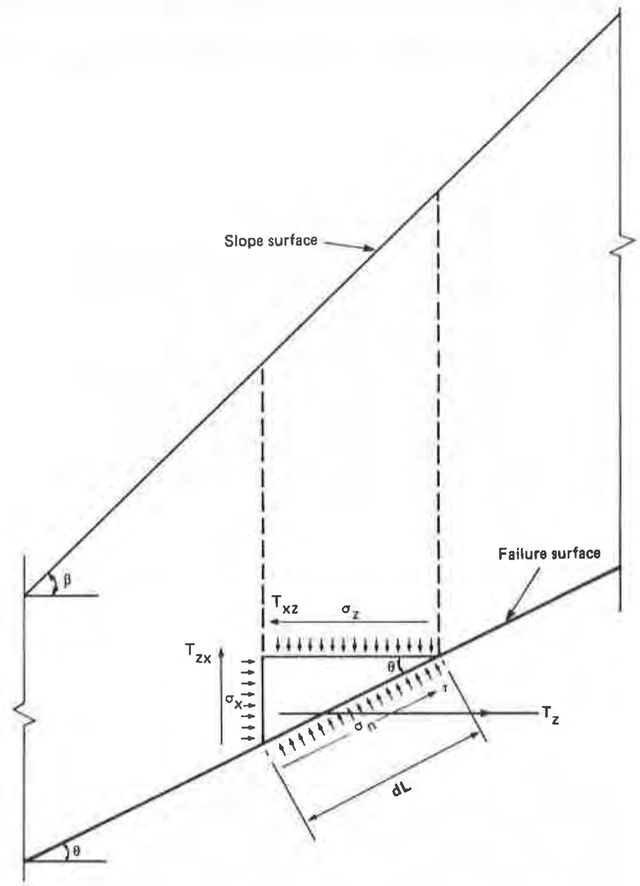

Fig. 2 Forces on element of reinforced soil immediately above failure surface

$$\sigma_x = K \gamma Z \cos^2\beta \qquad \text{...... (4)}$$

where K for active earth pressure conditions is equal to:

$$K = \frac{K_a + r_u (1 - K_a)}{(\cos^2\beta - K_a \sin^2\beta)} \qquad \text{...... (5)}$$

On the basis of a vertical spacing for the fabric layers of S_v, the maximum force developed in the fabric is given by:

$$T_M = S_v K \gamma Z \cos^2\beta \qquad \text{...... (6)}$$

The tension distribution in the fabric layer will probably vary from a maximum value near the potential failure surface to zero at the edge furthest from the slope. Assuming a linear variation of tension then the average value ($\bar{T}$) will be about half of that given by Equation (6). Substituting $\bar{T}$ in Equation (3a) enables the average vertical spacing (S_V) to be determined consistent with the specified extension (e).

$$\text{ie}\quad S_v = \frac{2e}{(me + c)} \times \frac{1}{K.\gamma.\bar{Z}.\ \cos^2\beta} \qquad \ldots\ldots (7)$$

Integration of Equations (1) and (2) employing the above deformation criteria produces the equations, in terms of effective stress, shown in Table 1. The coefficients associated with these equations are defined in Table 2.

TABLE 1

Equations employed in assessing stability

Sector	Resistance Equation	
	Contribution from soil	Contribution from fabric
1	$R_{1S} = \left[\frac{A_1\ X_1^{\ 2}}{2}(M_\beta - M_1) + C_1{}^1X_1\right]/\cos\theta_1$	$R_{1F} = \frac{B_1N_1}{2}\left[\frac{(N_1+1)(M_\beta-M_1)}{M_1} S_{v1} + M\beta L_1\right]$
2	$R_{2S} = \frac{(X_2-X_1)}{\cos\theta_2}\left[\frac{A_2}{2}\{X_2\ (M_\beta - M_2) + X_1\ (M_\beta + M_2 - 2M_1)\} + C_2{}^1\right]$	$R_{2F} = \frac{B_2N_2}{2}\left[\frac{(N_2+1)(M_\beta-M_2)}{M_2} S_{v2} + 2X_1\ (M_\beta - M_1) + M_\beta L_2\right]$
3	$R_{3S} = \frac{(X_3-X_2)}{\cos\theta_2}\left[A_3\ \{M_\beta X_2 + X_1(M_2 - M_1) - \frac{M_2}{2}(X_3+X_2)\} + C_3{}^1\right]$	$R_{3F} = \frac{B_3N_3}{2}\left[2\{X_2(M_\beta-M_X) + X_1(M_2-M_1)\} - \frac{(N_3 + 1)}{2} S_{v3}\right]$
Sector	**Disturbance Equation**	
	Contribution from soil	**Contribution from fabric**
1	$D_{1S} = \frac{E_1\ X_1^{\ 2}\ (M_\beta - M_1)}{2\ \cos\theta_1}$	$D_{1F} = -\frac{G_1N_1}{2}\left[\frac{(N_1+1)(M_\beta-M_1)}{M_1} S_{v1} + M_\beta L_1\right]$
2	$D_{2S} = \frac{E_2\ (X_2-X_1)}{2\ \cos\theta_2}\left[M_\beta(X_2+X_1) + X_1(M_2-2M_1) - M_2X_2\right]$	$D_{2F} = -\frac{G_2N_2}{2}\left[\frac{(N_2+1)(M_\beta-M_2)}{M_2} S_{v2} + 2X_1(M_\beta-M_1) + M_\beta L_2\right]$
3	$D_{3S} = \frac{E_3\ (X_3-X_2)}{\cos\theta_2}\left[M_\beta X_2 + X_1(M_2-M_1) - \frac{M_2}{2}(X_3+X_2)\right]$	$D_{3F} = -\frac{G_3N_3}{2}\left[2\{X_2(M_\beta-M_2) + X_1\ (M_2-M_1)\} - \frac{(N_3+1)}{M_2} S_{v3}\right]$

$$\text{Factor of safety (F)} = \frac{\sum_{i=1}^{i=3} (R_{is} + R_{iF} + F.D_{iF})}{\sum_{i=1}^{i=3} D_{is}}$$

TABLE 2

Coefficients employed in stability equations

Sector	Resistance Equation	
	Contribution from soil	Contribution from fabric
1	$A_1 = \gamma_1.\tan\phi_1^1.\cos^2\theta_1\left[1 + K_1\cos^2\beta(\tan\beta - \tan\theta_1)^2 - r_{u1}/\cos^2\theta_1\right]$	$B_1 = 2\mu_1L_1\gamma_1\ \sin\theta_1\ \tan\phi_1^1\ (1 + K_1\ \sin^2\beta - r_{u1}/\sin\theta_1\ \tan\phi_1^1)$
2	$A_2 = \gamma_2.\tan\phi_2^1.\cos^2\theta_2\left[1 + K_2\cos^2\theta(\tan\beta - \tan\theta_2)^2 - r_{u2}/\cos^2\theta_2\right]$	$B_2 = 2\mu_2L_2\gamma_2\ \sin\theta_2\ \tan\phi_2^1\ (1 + K_2\ \sin^2\beta - r_u/\sin\theta_2\ \tan\theta_2^1$
3	$A_3 = \gamma_3.\tan\phi_3^1.\cos^2\theta_2\left[1 - K_3\ \tan^2\theta_2 - r_{u3}/\cos^2\theta_2\right]$	$B_3 = 2\mu_3L_3\gamma_3\ \sin\theta_2.\tan\phi_3^1\ (1 - r_u/\sin\theta_2\ \tan\phi_3^1)$
Sector	**Disturbance Equation**	
	Contribution from soil	**Contribution from fabric**
1	$E_1 = \gamma_1.\sin\theta_1\ \cos\theta_1\left[1 + K_1\cos^2\beta\ \{\tan^2\beta-1 + \frac{\tan\beta}{\tan\theta_1}(1-\tan^2\theta_1)\}\right]$	$G_1 = 2\mu_1L_1\gamma_1\ \cos\theta_1\ (1 + K_1\ \sin^2\beta - r_u/\cos\theta_1)$
2	$E_2 = \gamma_2.\sin\theta_2\ \cos\theta_2\left[1 + K_2\cos^2\beta\ \{\tan^2\beta-1 + \frac{\tan\beta}{\tan\theta_2}(1-\tan^2\theta_2)\}\right]$	$G_2 = 2\mu_2L_2\gamma_2\ \cos\theta_2\ (1 + K_2\ \sin^2\beta - r_u/\cos\theta_2)$
3	$E_3 = \gamma_3\ \sin\theta_2\ \cos\theta_2\left[1 - K_3\right]$	$G_3 = 2\mu_3L_3\gamma_3\ \cos\theta_2\ (1 - r_u/\cos\theta_2)$

Influence of compaction on deformation

Construction plant is a further source of loading which can induce extension of the fabric. The mechanism involved may be as shown in Fig 3 where it is assumed that an effective bond exists between the fabric and soil particles. Compaction stresses induce local extension which is not fully recoverable because other particles interpose. Although compaction vibrations may result in some loss of contact stress, overall a net "locked-in" tension will result which prestrains the fabric and prevents further extension until the load induced by the fill creates tensions in excess of the "locked-in" values. Moreover, the pretension increases effective stress and improves the strength of the fill. The following analysis based on elasticity considerations permits fabric extension resulting from plant activity to be estimated.

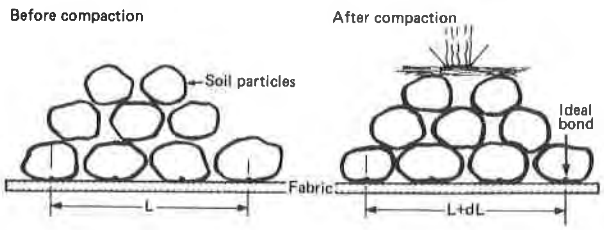

Fig. 3a Influence of compaction on pre-strain of soil

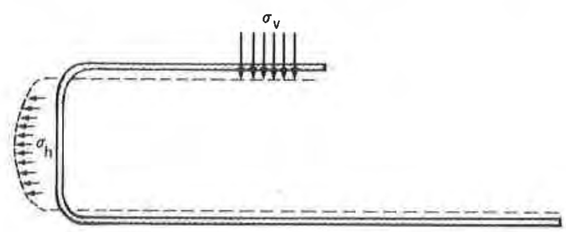

Fig. 3b Compaction stress inducing deformation at face of slope

Considering the force applied by the compaction roller as a surface line load acting normal to the slope face then the horizontal stress at a reflected boundary (Fig 3b) is obtained as follows (assuming Poisson's ratio for the soil is 0.5):

$$\sigma_h = \left[\frac{P}{\pi}\,\frac{x^3}{R^3}\right]_{X_B}^{X_A} \quad \ldots\ldots (8)$$

The value of P has to be increased to allow for the effect of centrifugal force if vibrating plant is employed. A factor of two is normally used to account for this effect. Thus the force (F_C) developed at the slope face for a compacted layer of thickness t is obtained as follows:

$$F_C = \frac{P}{\pi}\int_0^t \left(\frac{X_A^{\,3}}{(X_A^{\,2} + Z^2)^{3/2}} - \frac{X_B^{\,3}}{(X_B^{\,2} + Z^2)^{3/2}}\right)\frac{dZ}{Z} \quad \ldots\ldots (9a)$$

ie
$$F_C = \frac{P}{\pi}\left[\frac{X_A}{(X_A^{\,2} + t^2)^{1/2}} - \ln\left\{\frac{(X_A + (X_A^{\,2} + t^2)^{1/2}}{t}\right\} - \frac{X_B}{(X_B^{\,2} + t^2)^{1/2}} + \ln\left\{\frac{(X_B + (X_B^{\,2} + t^2)^{1/2}}{t}\right\}\right] . (9b)$$

Applying F_C in place of T in Equation (3b) permits an estimate of fabric extension at the slope face to be made. A similar analysis can be developed for regions remote from the slope face to obtain the following equation:

$$F_C = \frac{P}{\pi}\left[\ln\left\{\frac{(W + (W^2 + t^2)^{1/2}}{t}\right\} - \frac{W}{(W^2 + t^2)^{1/2}}\right] . (9c)$$

An estimate of overall fabric extension induced by compaction plant can be obtained by applying the mean force determined from Equations (9b) and (9c) in Equation (3b).

Application of the technique to embankment and cutting slopes

The design calculations require data on the shear strength characteristics of the fill and the interface friction between soil and fabric. The former tests are best carried out in terms of effective stress using the triaxial apparatus. The latter values can be obtained from modified shear box tests. Load-extension tests also need to be carried out on the fabric and although several methods are available, McGown et al have pointed out the advantages of testing structural fabrics in a soil environment (2,3). An assessment of the likely pore pressure conditions will also be needed with cuttings, and embankments constructed from cohesive fill.

An example of the application of the technique to the design of a cohesive fill embankment is shown in Fig 4, which also lists the assumed properties of the soil and fabric. The safety factor for the slip surface shown (Fig 4a) was determined as 0.75. A subsequent analysis employing fabric reinforcement permitting up to one per cent extension produced the arrangement shown in Fig 4b. The factor of safety on the previous slip surface had been increased to 1.3. The permitted extension of the fabric would have produced deformation at the face of the slope of about 4 cm, however, assuming a 10 kN/m roller was used to compact the fill in 0.25 m thick layers, 25 per cent of the deformation would have been taken up by the compaction plant. The analysis has clearly produced a reasonably satisfactory design with regard to the original slip surface but it would now be necessary to check the stability of potentially deeper-seated failure surfaces behind the reinforced section.

Following a recent failure of a motorway cutting in London clay in Berkshire, England, reinstatement was undertaken by re-using the original soil reinforced by layers of fabric (Netlon CE131). Thic technique was adopted because of the high haulage costs which would have been incurred by the conventional method of removing the failed soils and replacing them with good quality material.

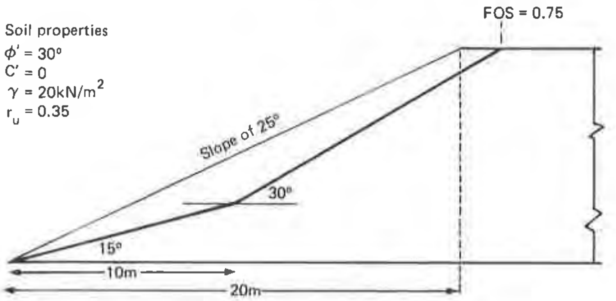

Fig. 4a Cohesive fill embankment unreinforced

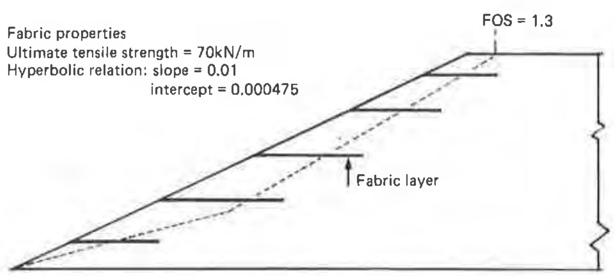

Fig. 4b Cohesive fill embankment reinforced by high strength fabric

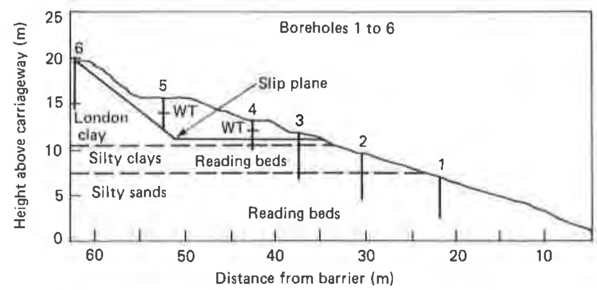

Fig. 5 Location of boreholes formed before remedial measures

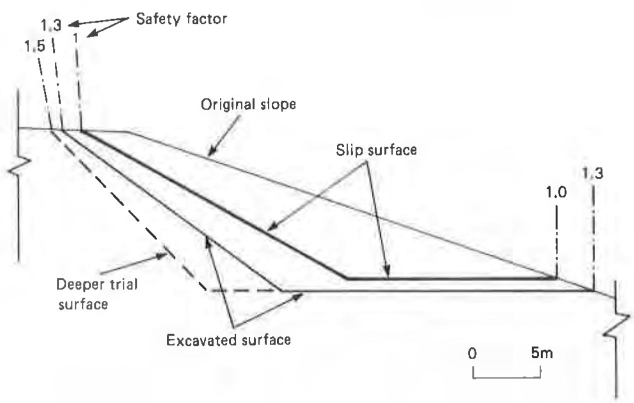

Fig. 6 Simplified profiles assumed for puposes of stability analysis

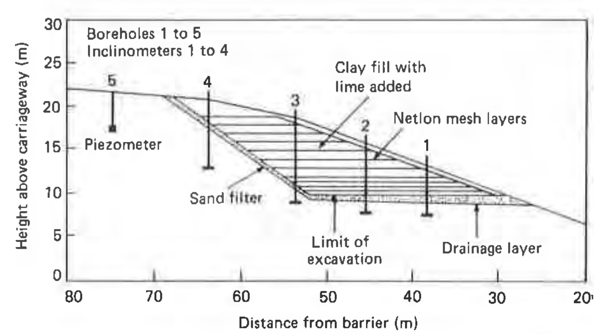

Fig. 7 Location of boreholes and inclinometers after remedial measures

A cross-section of the failed cutting slope is shown in Fig 5. Triaxial tests produced a ϕ^1 value for the soil of about 25° and a value for r_u of 0.3 was obtained from standpipe piezometers. The results of the various analyses and the associated slip surfaces are presented in Table 3 and Fig 6 respectively. These indicate that the possibility of failure of the original slip surface has been removed as the minimum factor of safety was 1.3, even without the benefit of drainage measures and lime treatment.

The use of about one per cent by weight of quicklime at the scheme proved particularly advantageous as the construction plant was able to work effectively on the reinstated clay, even after wet weather, and further increased the safety factor (Table 3).

The cross-section of the reinstated cutting is shown in Fig 7. The vertical spacing of the reinforcement was 0.5 m in the bottom region which was increased to 1 m at higher levels.

The cost of the repair was estimated to be about 40 per cent less than that of the conventional method. These savings were related to the high haulage costs at the scheme and conditions would not always be as favourable.

Inclinometers installed in the slope for more than a year have shown no indication of movement.

TABLE 3

Factors of safety for various potential failure surfaces and conditions

Failure profile	Treatment	Sector	ϕ^1 degree	r_u	Safety factor
Original slip surface	None	1 2 3	25	0.3	1.0
	Fabric rein-statement	1 2 3	25	0.3	>10 (Adherence 1.6 (Tension at 4 per cent strain)
Excavated zone	None	1 2 3	28 25 25	0.3	1.3
	Drainage measures	1 2 3	28 25 25	0.2	1.5
	Drainage measures and lime treatment	1 2 3	35	0.2	2.0
Deeper trial zone	None	1 2 3	28 25 25	0.3	1.5
	Drainage measures	1 2 3	28 25 25	0.2	1.8
	Drainage measures and lime treatment	1 2 3	35	0.2	2.3

Conclusion

A description is provided of an analytical procedure for the design of cuttings and enbankments reinforced with fabric. The equations described are linked to deformation criteria so that conditions of serviceability are automatically satisfied.

An example of the application of the method to the design of a cohesive fill embankment is given showing how such an embankment may be constructed with steeper side slopes than would normally be possible and producing savings both in quantity of fill and land take. The repair of a failed section of motorway cutting in England using the method is also briefly described whereby the foundered soil was re-used in conjunction with fabric mesh reinforcement. It was estimated that the cost of the repair was reduced by 40 per cent when compared to the conventional approach of removing the failed soil and replacing with good quality granular fill.

Acknowledgements

The work described in this paper forms part of the programme of the Transport and Road Research Laboratory and the paper is published by permission of the Director.

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APPENDIX 1

List of symbols

A_i = coefft associated with soil resistance in sector i (Table 2)

B_i = coefft associated with fabric resistance in sector i (Table 2)

c = intercept of load-extension relation for fabric on hyperbolic plot

c^1 = cohesion intercept in terms of effective stress

D_T = total disturbing force

D_{is} = contribution to disturbance force offered by soil in sector i

D_{if} = contribution to disturbance force offered by fabric in sector i

e = extension of fabric in load-extension test

E_i = coefficient associated with disturbance force offered by soil in sector i (Table 2)

f = subscript relating to fabric

F = factor of safety

F_c = force applied horizontally be compaction plant

G_i = coefft associated with distrubance force offered by fabric in sector i (Table 2)

i = subscript relating to sector 1, 2 or 3

K_a = active earth pressure coefft

K_i = earth pressure coefft in sector i

L_i = length of fabric beyond slip surface in sector i

M = slope of load-extension relation for fabric on hyperbolic plot

M_β = tan (β)

M_1 = $\tan\theta_1$

M_2 = $\tan\theta_2$

N_i = number of fabric layers in sector i

P = line load per m applied by compaction roller

R = $\sqrt{X^2 + Z^2}$ in Equation (8)

R_T = total resisting force

R_{is} = contribution to resisting force offered by soil in sector i

R_{if} = contribution to resisting force offered by fabric in sector i

r_u = pore pressure ratio

t = thickness of compacted layer

T = load in fabric during load-extension test

T_Z = resistance at depth Z offered by fabric

W = half-width of compaction roller

X_A = distance from slope face to closest edge of compaction roller

X_B = distance from slope face to further edge of compaction roller

X_1 = length from origin to end of sector 1

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X_2 = length from origin to end of sector 2

X_3 = length from origin to end of sector 3

Z = depth to point under consideration

$\bar{Z}$ = average depth in sector being considered

β = angle of slope

γ = unit weight of soil

ϕ^1 = internal friction angle of soil in terms of effective stress

θ_i = angle of slip lane in sector 1, or 2

μ_i = interface coefficient of friction between soil and fabric in sector i

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Rescue Operation at Karlino, Poland

L'action de secours à Karlino, Pologne

On Tuesday, Dec.9th 1980 occured enormous eruption of oil and gas by Karlino, Poland. Karlino became symbol of international solidarity in bringing assistance in order to subjugate the element. This article describes the application of geotextiles in construction of earth reinforcements at Karlino. Owing to them it was possible to attain quickness of works, necessary in rescue operation. On Dec.17th 1980 application of geotextiles has been proposed. On Jan.10th 1981 the action came to the end, although even optimists expected the end of action not earlier then the last decade of March 1981. I feel obliged to express my gratitude for uninterested and instant help of companies: I.C.I., Hoechst, Du Pont de Nemours, Fibertex.

Mardi, le 9 decembre 1980, pres de Karlino, en Pologne, a eu lieu une enorme eruption de petrole et de gaz. Karlino est devenu un magnifique symbole de la solidarite internationale dans l'action visant a dominer les forces de la nature. L'article analyse l'utilisation des geotextiles dans la construction des renforts en terre a Karlino, grace auxquels on a obtenu la cadence de travaux appropriee a une action de secour. L'utilisation des geotextiles a ete proposee le 17 decembre 80. Le 10 janvier 1981 tout a deja ete fini. Et pourtant les optimistes prevoyaient la fin de l'action de secour seulement vers la fin du mois de mars 1981. Je remercie de tout mon coeur les societes I.C.I., Hoechst, Du Pont de Nemours, Fibertex de leur aide desinteressee et rapide.

INTRODUCTION

It was up till now the biggest fire of oil well in Europe. The pressure of erupting oil of the burning well exceeded 550 Atm. Without absolute certainty of stoping the eruption,it was not possible to start to extinguish the fire. In case of sole extinguishing of the fire, flowing oil could catastrophically contaminate the environment. It could poison rivers Radew and Parsęta as well as the coast of Baltic in vicinity of Kołobrzeg,well known health resort. The distance to the sea coast was only 30 km. Both rivers are considered to have water of the highest purity, which is rather rare in Poland. Before starting to extinguish the fire, it was necessary to build strong earth reinforcements. Between 9th and 17th Dec.1980 quickly built embankments long about 4 km started to dissolve in constant rain. Built of local ground,without proper compacting, without proper slopes, embankments presented serious problem so, that it was considered necessary to dismantle them and build them anew of proper material. This had to be brought by trucks. Earth works were to be built with means of classical methods of compacting ground. Similarly the earth dam built of plastic clay, dissolved in continuous rainfall under it's own weight. Until this moment there were completed only 15% of earth works on the dam. After this mishap professional companies intended to build the dam anew, using Larsen Piles and reinforced-concrete constructions.

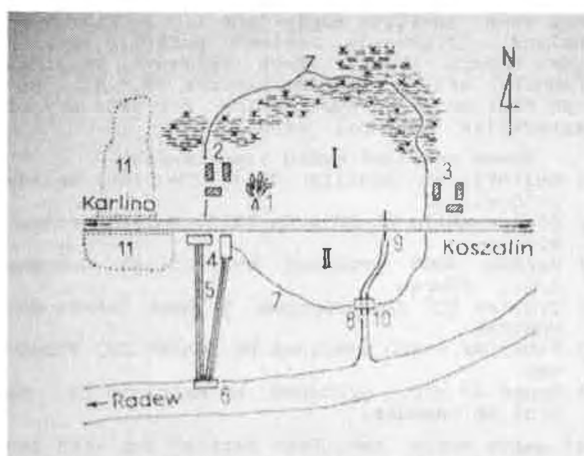


Fig.1 Situation sketch of the site of the rescue action at Karlino.
1 - source of fire, 2,3-evacuated farms, 4 - fire brigade's reservoirs, 5 - three pipelines Ø 500 to each reservoir, 6 - pumping station on pontoons, 7 - protecting embankments, 8 - dam, 9 - culvert Ø 200 under the highway, 10 - two culverts under the dam 2x200, 11 - forests, 12 - protecting embankment on the highway, I - first reservoir for oil and contaminated water, II - second reservoir for oil and contaminated water.

The period of rescue action would be in such a case prolonged to - at least - three months. It was also necessary to make typical designs, which would additionally prolong the time of action.

Author of this report applied as volunteer to the Staff of Rescue Action on Dec.17, 1980. His proposal to use geotextiles for construction of earth reinforcements was met with very critical attitude of managers of professional companies called to solve problems of these works. Owing to confidence of the management of Staff of Rescue Action, proposal of applying unknown method of construction of earth reinforcements with use of geotextiles has been accepted and executed. Earth reservoirs had to contain theoretically two days eruption of oil, after the fire has been extinguished. Beside that they had to contain enormous quantities of water and foam producing materials. Water was used continually to cool the ground around the well and the end of pipe, from which erupted burning geyser of oil. It was proposed to use 20 water guns. Each of them had to throw abt. 1600 ltr of water per minute.Additionally there were 3 guns with capacity of 2400 ltr per minute. Guns were situated in two rings. In the distance of 15 m from the well were positioned 10 guns and in the distance of 30 m - 13 guns. In one minute fell on the fire 39.2 m^3 water (39 200 litres) which is the quantity needed by city of 200 000 inhabitants.

GEOTEXTILES USED IN KARLINO

After receiving authorization by Staff of Action, several producing geotextiles companies in Western Europe were addressed by telex. At once came positive reply from I.C.I. Fibres in England. Afterwards arrived poistive replies from Messrs Hoechst (West Germany), Fibertex (Denmark) and Du Pont de Nemours (U.S.A.). Beside that we filed application for delivery of geotextiles of local production.

These were following assortments:

a) melioration textile WD-2,I/Sm 5106, Polish production
b) road textile WD-EB,I/Sm 5715, Polish production
c) Terram 4000 produced by English Company I.C.I. Fibres
d) Trevira 500 g/m produced by West German Co. Hoechst
e) Fibertex S-400 produced by Danish Co. Fibertex
f) Typar 27 g/m, produced by American Co. Du Pont de Nemours.

All earth works have been carried out with use of over 30 thousand m^2 of geotextiles according to the following list:

Polish WD-2	- 17 610 m^2
Polish WD-EB	- 600 m^2
English Terram 4000	- 5 300 m^2
German Trevira 500	- 2 000 m^2
Danish Fibertex S-400	- 1 800 m^2
American Typar	- 3 000 m^2
total	30 310 m^2

It is to be stressed, that geotextiles from abroad were offered free of charge by mentioned Companies.

EARTH WORKS

Scope of earth works was enormous. They were to be carried out in shortest possible time, because each day of fire caused enormous losses. There were to be built two reservoirs out of which I had the capacity of abt. 200 thousand m^3 (200 000 000 litres) and II had the capacity of 160 thousand m^3 (fig. 1). All earth reinforcements without exception were built from local ground (swamp, peat, plastic clay, quicksand a.s.o.). In one place just behind the farm (object 2 on fig. 1), there was used animal dung to build the embankment of the length of 25 m, because that was just at hand. It was possible only with use of geotextiles. Contrary to initial intentions, instead of dismantling banks made till Dec.17th, which were dissolving in continuous rainfall, it was decided to reinforce them by covering them with uninterrupted sequence of geotextiles. Shaping of banks had been omitted because it made no sense and there was no time and possibility to do so (fig. 2).

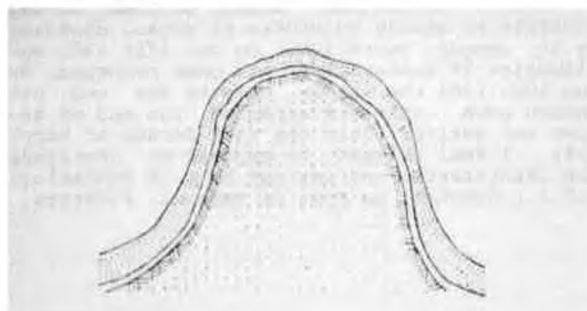


Fig.2 Reinforcing of existing embankments by spreading over them geotextiles and covering them with original ground.

Phot.1 Constructed embankment of reservoir I, in background from east, to the right side from south along the highway from Karlino to Koszalin.

The embankments covered with geotextiles were afterwards heightened with next layer of local ground to the height indicated by surveyors. All works were executed in greater part by soldiers in exceptionally hard ground and water conditions. It is illustrated by a photograph (phot. 1). Covered with geotextiles banks, guarded against immediate precipitation, became kind of core of entity of enlarged earth construction on the circumference of both reservoirs. The height of embankments in reservoir I oscillated from 1.0 to 1.5 m only in it's corner on the east side (fig. 1) the height of the bank exceeded 2.0 m. The slope of embankment built along the highway, on the inside of the reservoir I was 4.0 m high. In reservoir II, on the east side, the height of embankments oscillated between 0.8 and 2.0 m and on the south side, along the line of the dam, the height of earth reinforcements exceeded 8.0 m. Low, existing banks were covered in such way that along the top was uncoiled the reel of fabric and afterwards it was pulled to the sides, covering all lateral cross-section of the embankment (fig. 2). Needed beadths of geotextiles were attained by sewing particular strips by means of French machine type SAC-UP THIMONNIER. They passed perfectly the exam under difficult atmospheric and field conditions. They were serviced by transportable agregates producing alternating current 220 V. The force of agregates being 1.4 kM. For sewing were used Polish threads, polyamid, with symbol dtex 940 x 2 x 2 S.

THE DAM

It's construction involved special difficulties. The dam was the nevralgic point of total action as far as environment is involved. It was situated on south side of reservoir II. It created barrier to natural flew of ground and precipitation waters to river Radew. It had to guard the river against sewage from contaminated terrain. First trial of building the dam out of the local ground proved unsuccessful. In the place where it was situated there was mainly plastic clay and quicksand. Constant rains caused disintegration of executed works. On the bottom of local water flow were put two pipes Ø 200 mm and long 16 m. On the east side the works were somewhat advanced. However in certain moment earth masses flew under their own weight southward (fig. 3).

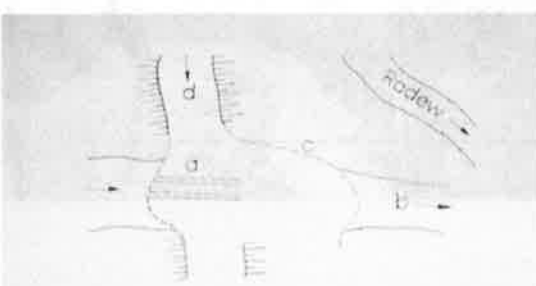


Fig.3 View from above of the south side dam, closing the reservoir II.
a) pipes Ø 200
b) to the river Radwa
c) the limit of the flow of the dam
d) direction of works.

Works were stopped. Called specialists advised the use of Larsen Piles and reinforced concrete. It threatened with very serious delay of action of extinguishing fire of the oil-well. Again was chosen the conception of using geotextiles - at my own risk. In order to reinforce lower layers of the dam, there was used Polish Patent Nr 111 085, being the property of Higher Engineering School in Zielona Góra, titled: "Mat to reinforce grounds". Patent solution was put in execution in two ways:

- in first way there were constructed special mats in shape of broad carpets made of geotextiles offered by Western Companies (fig. 4).

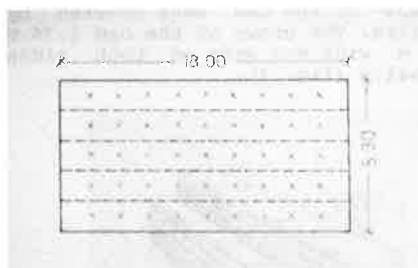


Fig.4 View from above of the mat to reinforce grounds, made according to Patent Nr 111 085 of different geotextiles and glass fibre fabricks.

- in the second, there were constructed special stripes long 50 m made of Polish geotextiles WD-2, with which were bound particular layers of the dam according to my idea (fig. 5 and fig. 6).

Fig.5 Binding of lower layers of the dam with special strips made of fabric WD-2 according to Patent Nr 111085.

Fig.6 Special strip made according to Patent Nr 111085, used for lower bindings of the most loaded layers of dam.

To make mats on the basis of afore mentioned Patent, there were also used glass textiles ST 41/I-90 and ST 44/I-107. Mats were sown by means of machines type SAC-UP THIMONNIER. The remaining surface of mats was pricked out by copper or alluminium wires. Mats for reinforcement of ground were done on a vase of geotextiles Terram 4000, Fibertex S-400, Trevira 500. They were used for layer reinforcement of the two lowest levels of the dam. Mats used in a form of strips made on a base of Polish geotextilematerial WD-2 were used for binding of at least 3 of the lowest layers of the dam. The remaining layers of the dam were covered by single geotextiles. The crown of the dam 1.76 m high and 5.00 m wide was made on both sides with vertical walls (fig. 7).

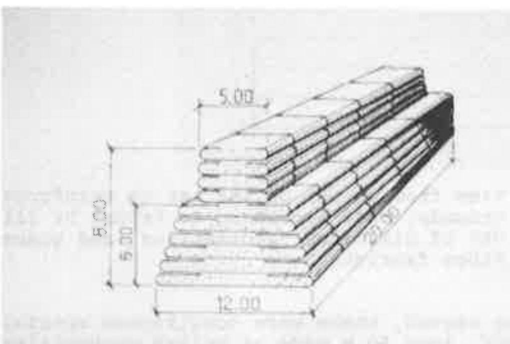
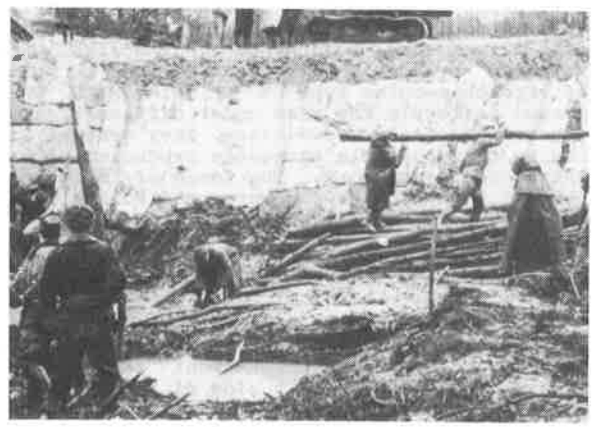


Fig.7 Dimensions of the dam.

Most serious problems were not only with the particularly unfavourable ground-water conditions but as well as constant rain-snow and rain falls. There were a few freezing days. Air temperature varied from -7° to $+4^{\circ}C$. But even during minus temperature the ground works with use of geotextiles were constantly continued, especially by constructing the dam (phot. 2-5)

Phot.2 Earth dam during construction, with aid of geotextiles.



Phot.6 Grimness and beauty of element's force.

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The Reinforcement of Granular Materials with Continuous Fibers
Le renforcement des matériaux granulaires avec des fils continus

RESUME

Les géotextiles sont conçus d'ordinaire comme des nappes que l'on déroule sur le sol, mais les avantages fondamentaux de l'association des fibres de polymère et des sols (résistance, ductilité, continuité, capacité de drainage et de filtration) ne sont pas limités à l'emploi des nappes et peuvent être mis à profit en faisant appel aux différents produits textiles faits à partir de fibres de polymère. Le plus courant de ces produits est le fil. La communication décrit le travail de recherche et de développement effectué sur l'association tridimensionnelle de sols granulaires et de fils continus, avec des informations sur les techniques de mélange, les essais en vraie grandeur de production et de comportement du matériau ainsi produit, les essais de laboratoire sur ses propriétés mécaniques et les mécanismes fondamentaux intervenant dans son comportement.

ABSTRACT

Geotextiles are usually viewed as rolls of fabric laid on the ground, but the fundamental advantages of associating polymer fibers and soils (resistance, ductility, continuity, drainage and filtration capacity) are not limited to the use of fabrics and may be put to use by resorting to the different textile products made from polymer fibers. The most common of these products is the thread. The paper describes research and development work made on the three-dimensional association of granular soils and continuous threads, with information on the techniques used for the mixing, experiments on full scale production and behaviour of the resulting material, laboratory tests on its mechanical properties and fundamental mechanisms involved in its behaviour.

INTRODUCTION

Depuis plus de dix ans, le développement des géotextiles illustre les avantages de l'introduction des fibres synthétiques dans les sols dans tous les domaines de la géotechnique. Ces avantages sont à la fois mécaniques et hydrauliques. Sur le plan mécanique le géotextile est un élément de continuité qui modifie le transfert des contraintes et la répartition des déformations, d'où une profonde modification du comportement du sol aussi bien sous l'effet des charges statiques que répétées. Sur le plan hydraulique le fait d'éviter les discontinuités dans les déformations est également un avantage important ; de plus les milieux fibreux ont la particularité de pouvoir présenter des porosités très élevées associées à des diamètres de filtration faibles, avec par conséquent des propriétés très intéressantes pour le drainage et la filtration.

Le développement des géotextiles s'est fait jusqu'à présent surtout sous la forme de nappes, c'est-à-dire de grande surfaces de faible épaisseur, auxquelles on peut ajouter quelques applications particulières comme les drains verticaux ou certaines tentatives d'armatures textiles, que l'on peut considérer, par rapport à la masse du sol, comme des éléments à une dimension. A trois dimensions on a réalisé des massifs multicouches constitués de nappes de géotextiles intercalées entre des couches de sol. On n'a cependant pas encore utilisé de composite sol-fibres à trois dimensions dans lequel l'association du milieu granulaire et des éléments fibreux est suffisamment intime pour constituer un matériau que l'on puisse considérer comme homogène, bien que le sujet ait déjà été abordé au niveau de la recherche de laboratoire il y a plusieurs années (1). On peut pourtant penser que les ressources potentielles évoquées brièvement ci-dessus du mariage sol-fibres puissent donner à un tel matériau des propriétés intéressantes. Cette communication présente les résultats de recherches destinées à étudier la faisabilité d'un tel matériau.

PRINCIPE DU PROCEDE

Le principe du mélange d'un matériau granulaire et de fils continus consiste à projeter un ou plusieurs fils, par voie pneumatique ou hydraulique, sur le matériau granulaire en mouvement, par exemple à l'extrémité d'une bande transporteuse, à la sortie d'un tuyau de remblai hydraulique, ou plus généralement à la sortie de tout système de transport ou d'épandage. On obtient alors un mélange tridimensionnel désordonné de fils et de particules solides ayant des propriétés mécaniques et hydrauliques intéressantes, qui sont décrites plus loin.

Dans ce qu'il a de plus essentiel le principe du procédé est donc d'associer à un matériau discontinu à frottement interne des éléments continus souples, dans une disposition géométrique désordonnée et tridimensionnelle. Il se distingue donc nettement :

- des composites où les éléments continus sont noyés dans une masse à laquelle ils sont liés par adhérence ;
- des association sol-armatures où ces dernières sont disposées de façon régulière et rectiligne, dans un sens déterminé et avec des espacements grands par rapport aux dimensions des particules du sol ;
- des mélanges utilisant des fibres coupées.

Les différences avec les procédés ci-dessus, qui apparaissent au niveau des composants et de leur mode d'association, se traduisent aussi au niveau du comportement, ainsi qu'on le verra plus loin.

ETUDES DE FAISABILITE

1 - Premières fabrications

L'idée de réaliser un mélange à trois dimensions de fibres et de matériaux granulaires est une conséquence du développement des géotextiles au début des années 70 et en constitue en quelque sorte une généralisation. Vers 1973-74 quelques tentatives tout à fait rudimentaires de mélange de fibres courtes et de sable ont été faits par l'auteur, qui ont été reprises à partir de 1975 dans le cadre d'une recherche conduite par le Laboratoire Central des Ponts et Chaussées à Paris avec la collaboration du Centre de Recherches de Genève de l'Institut Battelle. Cette recherche avait pour objet d'explorer quelques idées nouvelles dans le domaine des géotextiles et s'est assez vite orientée vers la question du mélange milieu granulaire-fibres synthétiques. C'est dans le cadre de cette recherche que l'on a abouti à l'idée, réalisée d'abord en laboratoire à Genève en 1978 puis au Centre d'Expérimentations Routières de Rouen en 1979, de projeter des fils continus sur le matériau en mouvement.

Les premiers essais de laboratoire ont été faits par projection pneumatique de fil sur du sable entraîné dans un courant d'eau ; les premiers essais à plus grande échelle, portant sur quelques mètres cubes, ont été réalisés par projection du fil à l'eau sur du sable déversé par une bande transporteuse (Figures 1 et 2).

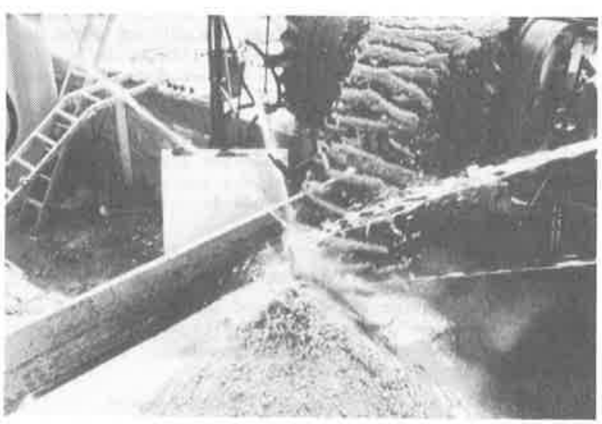

Figure 1 - Premiers essais en vraie grandeur en 1979. Les fils sont projetés par des jets d'eau.

First full size tests in 1979. The threads are ejected by water jets.

Figure 2 - Volume de sable traité à faible dosage (environ 0,5 pour mille). Fils tirés de la masse.

Sand mass with a low percentage of thread (approximately 0,5 ‰). Threads pulled from the mix.

2 - Premiers essais

A ce stade, des essais assez simples et surtout qualitatifs ont été faits pour avoir une idée du comportement du mélange sol-fil et en estimer l'intérêt éventuel.

En laboratoire on a fait des essais de compression simple et des essais de poinçonnement qui se sont révélés très encourageants (voir résultats plus loin).

Les essais de comportement ont été les suivants :

a) Traficabilité : sur une zone de sable non traité et non compacté un camion ne peut circuler car l'essieu moteur s'enfonce ; sur une zone de sable traité à un dosage pondéral inférieur à 1 pour mille la circulation se fait aisément sans création d'ornière importante.

b) Portance : une charge de plusieurs tonnes appliquée sur un tas conique de sable traité également à un dosage inférieur à 1 pour mille supporte cette charge avec des déformations très faibles (Figure 3).

Figure 3 - Charge exercée sur un tas traité à faible dosage. La pente reste stable.

Load on a mass treated with a low percentage. The slope remains stable.

c) Sollicitations dynamiques : essais à la "Dynaplaque". La dynaplaque (Figure 4) est un appareil d'essai comportant une plaque rigide de 60 cm de diamètre sur laquelle on exerce, par la chute d'une masse tombant sur la plaque par l'intermédiaire de ressorts, une impulsion correspondant au passage d'un jumelage de pneumatiques de camion circulant à 60 km/h, soit une force maximale de 6,5 T avec une durée d'impulsion de 3/100 de seconde environ. Le paramètre mesuré est le rebond de la masse qui est fonction, pour un milieu élastique, de son module. Sur un sable non traité le coefficient de restitution mesuré (rapport de la hauteur de rebond à la hauteur de chute) est d'environ 50 %. Sur le même sable traité au même dosage de fil que précédemment le coefficient de restitution est de l'ordre de 25 %.

Ce résultat apparaît a priori comme un peu contradictoire avec ce qui avait été constaté au cours de l'essai de traficabilité car la traficabilité d'un sol est d'autant meilleure que son module et son coefficient de restitution (qui varient dans le même sens pour un sol normal) sont élevés. Cette constatation montre que le comportement du mélange sable-fil est qualitativement différent de celui d'un sable naturel car :

- il supporte des efforts statiques plus élevés sans avoir un module augmenté en proportion ;

- il absorbe davantage d'énergie sous l'effet d'une sollicitation dynamique sans manifester de déformation apparente plus grande.

Figure 4 - *Test à la dynaplaque d'une masse traitée.*

Test with the "Dynaplaque" of a treated sand mass.

d) Résistance à l'érosion : une masse de 1 m³ environ a été soumise pendant une vingtaine de minutes à un jet d'eau puissant dirigé successivement dans différentes directions. L'effet de cette action érosive apparaît sur la figure 5 : le sable superficiel du mélange a été entraîné par l'eau mais au fur et à mesure de son départ la densité superficielle de fils a augmenté et a constitué finalement une couche de surface assurant une protection efficace contre l'érosion. Il est clair qu'un tel résultat ne peut être obtenu que si les éléments textiles incorporés dans la masse sont des filaments continus ; ils restent en effet ancrés dans le sable même s'ils sont dégagés de la masse grnaulaire sur une assez grande longueur, alors que des fibres courtes seraient libérées puis entraînées par l'eau.

Figure 5 - *Masse de sable traité ayant subi une attaque de 20 minutes au jet d'eau.*

Treated sand mass after a 20 minutes water jet attack.

Bien qu'essentiellement qualitatives ces observations montraient un intérêt suffisant du procédé pour appeler d'une part la confirmation de la faisabilité technique et économique et d'autre part préciser de façon chiffrée le comportement du mélange sable-fil.

3 - Deuxième étape des études de fabrication

Il s'agissait de répondre aux questions suivantes :

- peut-on produire un débit horaire suffisant pour des réalisations de chantier ?

- peut-on produire selon une géométrie contrôlée, puisque le matériau peut être difficilement repris ?

- la fiabilité du système de projection de fil est-elle satisfaisante et quel est son niveau de consommation d'énergie ?

- le coût du procédé permet-il d'envisager de nombreuses applications ?

La réponse aux trois premières questions a été apportée par la fabrication de mélange sable-fil avec un débit de 30 tonnes/heure, en adaptant les équipements nécessaires sur la machine d'épandage de matériau utilisée au Centre d'Expérimentations Routières de Rouen (Ponts et Chaussées) pour le remplissage des fosses d'essai destinées aux différentes études de ce centre.

Une vue de ce matériel est présentée sur la figure 6. La sable est amené par une bande transporteuse et est déversé dans une trémie ayant un mouvement alternatif de balayage transversal entre les deux bords de la fosse d'essai, pendant que la machine avance lentement le long de la fosse. Des fils sont projetés à la base de la trémie sur le sable en chute libre, au moyen d'une dizaine de buses d'éjection projetant chacune plusieurs fils.

On a pu réaliser ainsi sans grande difficulté un débit de 30 tonnes/heure, des débits plus élevés pouvant être réalisés selon le même principe.

Figure 6 - Vue de la machine d'épandage des matériaux du Centre d'Expérimentations Routières de Rouen (Ponts et Chaussées.
View of the material laying equipment of the Road Experimental Center of Rouen (Ponts et Chaussées).

Le principal problème à résoudre pour obtenir un débit relativement important est de pouvoir projeter une quantité de fil suffisante. Si l'on se base en effet sur le dosage pondéral de 1 pour mille, il faut projeter 30 kg de fil à l'heure pour 30 T/h de matériau. Si l'on emploie par exemple un fil de 160 dtex, ce qui correspond à 16 g par kilomètre de fil, il faut projeter 1 875 kilomètres de fil à l'heure, soit environ 500 mètres par seconde. Si la vitesse de défilement du fil est de 10 mètres par seconde, il faut projeter simultanément 50 fils et donc disposer sur la machine de 50 bobines de fil. En fait 70 bobines avaient été montées sur la machine, représentant 350 kg de fil.

Ces essais ont apporté la preuve :

- qu'il n'y a pas d'obstacle majeur pour réaliser des quantités de plusieurs dizaines de tonnes/heure ;

- que la fiabilité du système d'alimentation en fil était tout à fait satisfaisante ;

- qu'il était possible de réaliser un massif d'une géométrie déterminée ;

- que la consommation d'énergie n'était pas un problème puisque la consommation du compresseur utilisé correspondait à 0,15 kwh environ par tonne traitée, dans les conditions des essais où le matériel n'est pas encore optimisé.

Le dernier point ci-dessus relatif à la consommation d'énergie a pour conséquence que l'élément essentiel du coût d'un tel traitement est le coût du fil ; le reste correspond à la manutention du matériau granulaire, que l'on retrouve dans les opérations de terrassement comportant un traitement, quel que soit celui-ci.

Le coût des fils synthétiques produits couramment par l'industrie textile dépend de nombreux éléments : polymère utilisé, performances du fil, mode de présentation, etc. Dans les conditions actuelles on peut prendre un ordre de grandeur de 15 FF par kilogramme. Ceci conduit à un coût du fil de 15 FF par tonne traitée, soit environ 25 FF par m^3, pour un dosage de 1 pour mille. Un tel ordre de grandeur permet au procédé d'être compétitif s'il permet, sur le plan technique, d'apporter des solutions nouvelles à des problèmes actuellement mal résolus.

Parmi les applications envisageables, on peut notamment citer, à titre d'exemples :

- réalisation de fondations sur terrain compressible ou hétérogène ;

- remblais en sable, notamment pour ouvrages immergés ;

- massifs antisismiques, amélioration d'ouvrage en terre en vue d'augmenter leur résistance aux séismes.

Bien d'autres possibilités existent, dont l'intérêt dépendra des performances du matériau. Celles-ci ont été l'objet de mesures de laboratoire, décrites dans ce qui suit.

4 - Deuxième série d'essais

Les essais de laboratoire réalisés sur le mélange sable-fil ont été les suivants :

- essais de poinçonnement ;
- essais de compression simple ;
- essais triaxiaux ;
- essais triaxiaux avec chargements répétés.

a) Essais de poinçonnement

Ces essais ont été effectués suivant la procédure de l'essai CBR, sur des matériaux secs. Pour le sable seul l'essai a donné la valeur de 37 (avec surcharge). Avec 1 pour mille de fil la valeur obtenue est de 65, et atteint 100 pour un dosage de 3,5 pour mille.

b) Essais de compression simple

Des éprouvettes de 100 mm de diamètre et 200 mm de haut ont été fabriquées directement dans des moules avec un dispositif de fabrication spécial, étant donné que le carottage normal d'une éprouvette dans un massif n'est pas possible en raison de la présence des fils. Les résistances en compression simple mesurées ont été trouvées approximativement proportionnelles au dosage en fil, avec une valeur de 300 à 400 KPa pour 1 pour mille en poids.

c) Essais triaxiaux

Les essais ont été réalisés sur trois sables différents, sur des éprouvettes à l'état lâche et des éprouvettes à l'état compacté, avec des dosages compris entre 1,4 et 2 pour mille et des éprouvettes de sable seul, sans fil. Les essais ont été consolidés drainés, avec des valeurs de σ_3 comprises entre 50 et 500 KPa.

A l'état compacté on obtient un maximum puis une décroissance du déviateur, dans les éprouvettes avec fils comme dans les éprouvettes sans fil. Une différence du processus de rupture est cependant que la déformation axiale au maximum du déviateur de contrainte pour les éprouvettes avec fils est beaucoup plus élevé que pour les éprouvettes sans fils, atteignant 6 à 8 % contre 2 à 4 %.

Du point de vue de la résistance mécanique la présence du fil se traduit par l'apparition d'une cohésion de 150 à 250 KPa pour les dosages de 1,4 à 2 pour mille, pour les éprouvettes compactes. Pour les éprouvettes lâches la cohésion mesurée est plus faible, inférieure à 100 KPa.

d) Essais triaxiaux à chargements répétés

Ces essais ont été faits pour s'assurer que même avec un sable concassé la répétition des charges n'amenait pas une fatigue prématurée due au cisaillement des fils.

Les conditions d'essai étaient les suivantes :

- teneur en eau = 12 % ;
- poids volumique sec = 18 KN/m³ ;
- étreinte latérale 50 KPa ;
- rapport $\dfrac{(\sigma_1-\sigma_3)\text{ dynamique}}{(\sigma_1-\sigma_3)\text{ de rupture en statique}}$ = 0,4 - 0,46 - 0,5 - 0,64 ;
- nombre de cycles = 10^5;
- fréquence 0,5 Hz jusqu'à 200 cycles
  1 Hz " 2 000 cycles
  5 Hz " 10^5 " ;
- dosage en fil voisin de 2 pour mille.

On a constaté une déformation axiale cumulée augmentant linéairement en fonction du logarithme du nombre de cycles avec une déformation axiale finale comprise entre 2,5 et 6 % (sauf dans le cas où le déviateur atteignait 0,64 fois le déviateur de rupture en statique, où il y a eu rupture en cours d'essai).

Ce comportement en fatigue est donc normal, sans abrasion particulière des fils. Deux éprouvettes avec fils ont été rompues statiquement après avoir suvi 10^5 cycles ; leur résistance a été trouvée de 55 % plus élevée que les éprouvettes équivalentes non soumises aux chargements répétés, par suite de leur consolidation.

Une autre constatation faite au cours de ces essais concerne le module de déformation du matériau. Les différents modules mesurés sont dans l'ensemble un peu plus élevés lorsqu'il y a présence de fil, mais pas dans une forte proportion. Les modules tangents à l'origine de chaque cycle ne sont pas, en moyenne, plus élevés avec fil.

En conclusion de ces essais, on constate que l'adjonction de fils continus souples à un matériau granulaire a pour effet de lui conférer une cohésion importante en augmentant sa capacité de déformation à la rupture, mais sans accroître de façon notable le module du matériau dans le domaine des petites déformations. Il est à souligner que cette dernière conclusion n'est peut-être pas à généraliser car il se peut qu'elle soit liée aux conditions particulières de fabrication des éprouvettes d'essai, qui impliquent une forte tortuosité du fil. Il est possible qu'une mise en oeuvre en vraie grandeur conduisant à une géométrie différente des fils ait pour résultat un comportement différent dans le domaine des petites déformations. Il est vraisemblable également que l'emploi de fils à très haut module modifierait ce comportement mais l'aspect économique pourrait alors se trouver posé en d'autres termes au niveau des applications.

CONSIDERATIONS GENERALES SUR LE COMPORTEMENT D'UN MELANGE MATERIAU GRANULAIRE-FILS CONTINUS

1 - Avantages essentiels du mode de liaison par fils

Par rapport aux liants travaillant par adhérence, l'efficacité d'une liaison par fils ne dépend pas de la granulométrie du matériau granulaire, alors qu'une granulométrie contenant aussi peu de vides que possible est nécessaire pour bien utiliser un liant travaillant par adhérence.

Ce mode de liaison permet par conséquent de traiter des matériaux naturels de granulométrie creuse, qui ne seraient pas susceptibles d'un traitement classique sans correction granulométrique.

Par ailleurs la liaison par fils d'un matériau très creux permet d'obtenir un matériau résistant et cependant à forte perméabilité. On peut en effet considérer que la perméabilité n'est pas affectée par la présence des fils puisque ceux-ci n'occupent qu'une fraction négligeable du volume des vides (de l'ordre de 1 % pour un dosage de 2 pour mille).

Du point de vue hydraulique la présence d'un réseau de fils au sein du matériau constitue, à condition que le fil utilisé soit assez fin et le mélange assez régulier, un filtre tridimensionnel incorporé dans la masse, qui améliore la résistance à l'érosion interne du matériau granulaire initial. Il faut en effet bien noter que, même pour des dosages pondéraux aussi faibles que 1 pour mille, la longueur de fil par unité de volume traité est très importante si l'on utilise des fils de titres courants dans l'industrie textile (Figure 7). C'est ainsi par exemple que pour un matériau de densité sèche 1,7 traité à 1 pour mille par du fil de 100 décitex, la longueur de fil est de 17 cm par cm³ ; pour des dosages et des titres différents cette longueur est proportionnelle au dosage et inversement proportionnelle au titre.

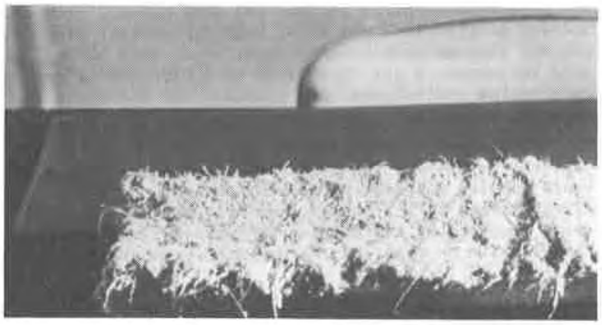

Figure 7 - Eprouvette de sable contenant 2 pour mille de fil, ayant été congelée puis rompue par écrasement diamétral.

Sample of sand containing 0.2 per cent of thread, after freezing and splitting by diametral compression.

Du point de vue mécanique, en plus de la possibilité de traiter un matériau de granulométrie creuse, le procédé présente l'avantage unique de pouvoir réaliser un matériau résistant non rigide, par conséquent non fragile et pouvant s'adapter à des déformations, problème souvent posé par les ouvrages de génie civil.

Il présente enfin la caractéristique de ne pas exiger de temps de prise.

2 - Mécanismes de la liaison par fils continus

Les mécanismes par lesquels des fils continus souples incorporés à un milieu granulaire modifient son comportement mécanique sont différents de ceux qui interviennent dans un composite à matrice continue et de ceux qui sont responsables de la tenue d'un système du type de la Terre Armée.

Une première observation est que pour un matériau où les fibres ne sont liées que par frottement et non pas par adhérence, les longueurs d'ancrage que l'on peut calculer pour des états de contrainte courants, en supposant qu'il y a un bon contact entre les fibres et le sol, sont de l'ordre de grandeur de la dizaine de centimètres ou du mètre. Il en résulte donc que des fibres coupées de quelques centimètres, comme celles que produit couramment l'industrie textile, ne peuvent pas être très efficaces, alors qu'elles peuvent l'être dans un composite où les fibres sont noyées dans une matrice continue à laquelle elles sont liées par adhérence.

Une deuxième observation est que l'hypothèse d'un "bon contact" entre les fibres et le sol granulaire faite pour calculer la longueur d'ancrage n'est pas vraisemblable si l'on tient compte de la flexibilité et des dimensions transversales d'un fil textile et du fait que le fil ou les fibres n'occupent qu'un volume de l'ordre de 1 % du volume des vides du milieu granulaire. On ne peut donc pas, dans ce cas, faire l'hypothèse, comme dans la Terre Armée, que la contrainte normale moyenne régnant au sein du milieu s'applique à la surface de l'armature et calculer la contrainte tangentielle possible à partir du coefficient de frottement. On peut au contraire considérer, à la limite, que les fibres textiles serpentent à l'intérieur du réseau des vides du milieu granulaire sans être serrées au sein de ce milieu.

En fait cette hypothèse est probablement elle-même excessive, mais on doit supposer que le fil n'est serré au sein du milieu que de points en points et que par conséquent la contrainte moyenne qui règne au sein du milieu n'est transmise que partiellement au fil et que l'effort tangentiel disponible correspondant n'est lui-même que partiel.

On pourrait alors à ce stade de l'analyse douter de l'efficacité du procédé, même avec des fils continus dont la longueur d'ancrage disponible n'est pas limitée. En fait le procédé est efficace car deux autres mécanismes interviennent dans son fonctionnement.

Le premier mécanisme est lié à la tortuosité du fil. Chaque fil présente de nombreuses sinuosités ; lorsqu'un fil est mis en tension des forces de frottement se développent dans les zones incurvées et introduisent une résistance qui croît en fonction de la tension appliquée sur le fil, à condition que quelques efforts localisés, au moins minimes, s'opposent à son glissement. Ce mécanisme lié à la courbure est celui que l'on retrouve utilisé par exemple dans la marine pour retenir un navire par une corde avec la force d'un seul homme, en enroulant la corde autour d'une borne d'amarrage.

Le second mécanisme est lié à l'entrecroisement des fils. Si la tension exercée sur une boucle tend à déplacer le fil transversalement par rapport à lui-même, il rencontre immédiatement d'autres fils qui s'opposent à ce déplacement et auxquels se transmettent des efforts, mettant ainsi en jeu de nouvelles zones du matériau.

Les deux mécanismes décrits brièvement ci-dessus ne sont présents ni dans les composites à matrice continue ni dans la Terre Armée. Le matériau décrit dans cette communication est donc bien entièrement distinct de ces deux conceptions.

DEVELOPPEMENT

La recherche sur les possibilités de ce matériau se poursuit dans les Laboratoires des Ponts et Chaussées avec la collaboration d'un producteur de fils synthétiques, d'un bureau d'ingénieurs-conseils et d'une entreprise de travaux publics. Le procédé a été l'objet de demandes de brevets en 1979 et 1980 et a reçu l'appellation de TEXSOL.

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Stability of Slopes Constructed with Polyester Reinforcing Fabric, Test Section at Almere—Holland, '79

Stabilité des talus renforcés au moyen des tissés d'armature polyester, Remblai d'Almere—Pays-Bas, '79

To determine whether polyester fabrics are capable of taking up prolonged heavy loads after slight deformation - installed horizontally between an embankment and a soft soil - can contribute materially towards the stability of embankments, four test sections, with the "Almere" section concluding the trials, were laid out in The Netherlands and in the Federal Republic of Germany in the period 1975-1980.
The results of visual observations, measurements and calculations of the Almere experiment showed that the contribution of these fabrics is indeed significant. Failure of the fabric-reinforced embankment occurred at a layer thickness of 3.50 m, whereby an effective load of over 100 kN/m was mobilised. The embankment in the unreinforced section failed at a height of 1.75 m. The thickness of the soft subsoil of this section was 3.50 m. In view of these results a proposal has been made on basis of the required material properties of reinforcing fabrics and the test methods relating thereto.

Pour vérifier si des tissus polyester, incorporés à l'horizontale entre un remblai et un sol mou est susceptibles, après une légère déformation, d'absorber un effort permanent important, apportent une contribution valable à la stabilité de ce remblai, on a ménagé, entre 1975 et 1980, aux Pays-Bas et en RFA, quatre sections expérimentales avec de telles armatures tissées, dont la dernière à Almere aux Pays-Bas, était le plus importante et décisive.
Le résultat des observations faites à l'oeil nu, et des mesures et évaluations effectuées pour cette dernière section ont montré que la contribution apportée est effectivement appréciable. Le remblai de sable muni d'une armature synthétique s'est effondré à 3,50 m. d'épaisseur, la force active mobilisée par le tissu s'élevant alors à plus de 100 kN/m. La partie sans armature du remblai a cédé à 1,75 m de hauteur. La couche compressible à cet endroit avait 3,50 m. d'épaisseur.
A partir de cette expérience, des propositions ont été émises pour la définition des exigences posées aux armatures tissées polyester et les méthodes d'essai correspondantes.

1. INTRODUCTION

In the past decade synthetic woven and non-woven materials, made up of synthetic yarns or fibres, have found growing acceptance in civil engineering. The large variety of types, of uses and the prospects offered by these materials to civil engineering, prompted The Netherlands Road Construction Research Centre in 1976 to form the Working group G2 "Synthetic Woven and non-woven Road Construction Fabrics". With a view to the above "mer à boire" this Working group decided to split up rapidly into a number of sub-groups. These sub-groups were named "Road base structures", "Road Embankment" and the third sub-group, derived from them was named "Measuring Methods". The split-up founds its origin in the differences in standards prescribed for base structures and embankments and in the specific functions to be performed by geotextiles in these constructions, notably primarily of separation layer and filter medium(Road base structures), and of constructive element (Road Embankments). In addition, earth retaining structures were built outside The Netherlands, whereby due to the fabrics - each time installed horizontally in the fill itself - e.g. a road widening embankment with 2:1 slope and a height of 12 m was realised in the south of France.
In the present article the "Road Embankment" sub Working group reports, under the responsibility of the entire Working group, on its results, the possibilities offered by synthetic reinforcing fabrics to contribute towards maintaining the stability of embankments on a soft soil and of steep-slope earth-retaining structures, as is shown in figures 1a and 1b.

If the reinforcing fabric can contribute substantially to the stability of these embankments, this application would offer the following advantages:

- the sand sub-base of a road can be laid directly on the natural soil without the need for digging a stabilizing trench,
- a greater depth of fill can be deposited in one operation without loss of stability of the embankment,
- the designed height can be attained with less overall (toe-to-toe) width of the embankment.

Moreover, when used for earth-retaining structures:

- this solution is much more economical than the conventional earth-retaining walls,
- settlement hardly matters because of the vertical deformation possibilities of the earth-retaining structure reinforced in this way.

2. DEVELOPMENT BETWEEN 1975 AND 1980

In order to ascertain how and to what extent reinforcing fabrics can make a contribution to the stability of earth fills of soils of low bearing capacity and to formulate the requirements to be met by such fabrics four experiments were carried out in the field.

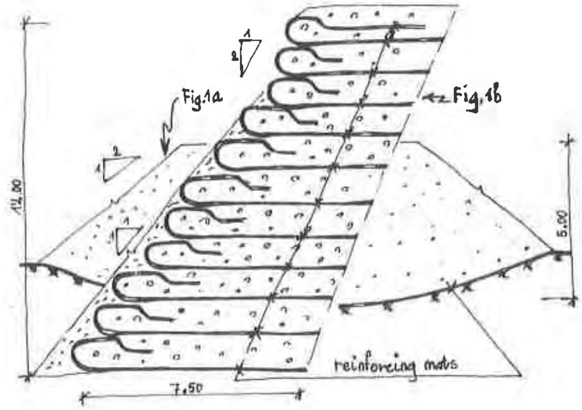

Fig. 1a and 1b.

2.1 In 1975 a comparitive investigation was conducted into the contribution made by synthetic woven fabrics to the stability of a road embankment laid on soft soil. This investigation was carried out by Koninklijke Wegenbouw Stevin and Enka near Zevenhoven, province of South Holland. Several times in succession two mounds of sand were deposited on the soft soil, one mound unreinforced and the other reinforced with synthetic fabric.
The purpose was to establish to what level a fill could be raised fast before failure occurred and to what an extent and in what places the fabric would be loaded. Calculations made by the Delft Soil Mechanics Laboratory (LGM) predicted that given the laminated soft clay-peat subsoil of 4 m thickness a circular slide would occur at a height of sandfill of 4 to 4.5m above the original ground level. This actually happened for the unreinforced sandfill, the adjacent soil being thrust up approx. 1 m. The mound raised on the reinforcing mat remained stable, although the adjacent soil was pushed up 30 cm. Measurements made on the fabric showed that it had absorbed a load of 20 kN/m at a maximum local elongation of 12%. Calculations made afterwards revealed that the factor of safety against sliding had increased by 5 to 10%.

2.2 In 1976, within the scope of the planned widening of the RW12 Motorway near Harmelen (province of Utrecht) the then Netherlands Motorway Egineering Laboratory set up a number of experimental sections, in which six synthetic non-woven and woven fabrics with different mechanical properties were laid directly on ground level.
In these sections hydraulic placing of a wet sandfill 2.50 m in height was planned. The subsoil consisted of a soft clay-peat layer of approx. 4 m thickness.
Hopes that the embankment could be raised in one stage instead of two 1.25 m thick layers were not fulfilled. Moreover, there was no significant difference between reinforced and unreinforced sections.
Afterwards calculations showed that the reinforcing materials should have been able to take up an effective load of approx. 75 kN/m at a maximum deformation of 4 to 5%. On average, this was ten times as high as the actual load take-up of the materials applied !
It was therefore concluded that the range of "woven and non-woven road construction fabrics" available then and at present are unsuitable for this application. Nevertheless, the experience gained in this experiment has proved to be of great importance in establishing the "Almere" experimental section and its predecessor the "Cuxhaven" experimental section.

2.3 The "Almere" experiment (see section 3.1) was preceded by a trial made in the north of the Federal Republic of Germany. At the end of 1978 an experimental section was established here after the RW12 model (see section 2.2) - but now with a reinforcing fabric - in connection with the construction of the A27 motorway near Cuxhaven. The experiment was carried out in co-operation with the Netherlands Road Construction Research Centre (SCW) and the Bundesanstalt für Strassenwesen (Federal Motorway Engineering Research Institute) in Cologne.
On a 5 m thick layer of soil consisting of clayey sand of poor permeability a sandfill with a height of 5 m and a slope of 1:1.5 had to be deposited fast by hydraulic filling. Calculations showed that with an adaptation of the excess pore water pressure of less than 40%, the factor of safety would become < 0.95.
Therefore the construction site management deemed it advisable to install a reinforcing fabric under the sandfill that would be capable of taking up a load of approx. 100 kN/m at low elongation during the consolidation period. The Working group G2 "Synthetic woven and non-woven fabrics used as mats in road construction" of SCW was allowed to study the load take-up of the fabric during and after filling with a specially developed measuring system using strain gauges fixed to the fabric.
Unfortunately the subsoil was found to be considerably more permeable than indicated by the results of the soil investigation. Moreover, the embankment was constructed with a slope of 1:2 so that the adaptation was invariably more than 50 to 60%. As a result the load take-up in the reinforcing fabric remained limited to 10 kN/m and no answers were provided to the questions as formulated in section 3.1.
A fortunate circumstance was that the method of measurement employed was successful; only 3 out of 40 strain gauges did not function !
Hence the opportunity offered mid-1979 by the Zuider Zee Works Department to establish an experimental section in connection with the construction of a sand dump near Almere was eagerly accepted.
Besides, the experimental section could be loaded to failure, so that a repetition of the "Cuxhaven" experiment would not occur.

3. EXPERIMENTAL SECTION "ALMERE"

3.1 Statement of Problem; Experiment Set-up

The purpose of the "Almere" experimental section was to provide an answer to the question whether a reinforcing fabric as used in Cuxhaven does indeed make a significant contribution to the stability of sandfills rapidly deposited on soft soil. At the same time it was to be investigated whether such a reinforcing fabric would be maximally loaded at the intersection with the critical sliding circle, which would be in accordance with expectations. To this end a reinforced and an unreinforced slope were studied by means of visual observations and measurements of pore water pressures in the subsoil and local strains in the reinforcing fabric. The measured data were introduced into mathematical models on the basis of the failure modes shown in figures 2a and 2b.

3.2 Experimental Section

3.2.1 A soil investigation on the spot showed that the subsoil consisted of a soft clay-peat layer of 3 to 4.5 m thickness with an undrained cohesion value of 10 kN/m² and with sufficient homogeneity (figures 3a and 3b).

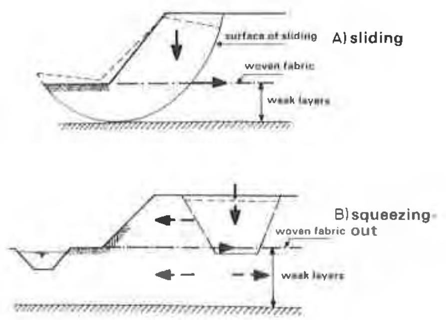

Fig. 2 Modes of Failure

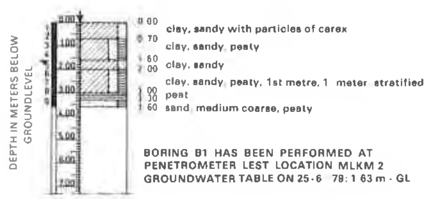

Fig. 3a Model of Boring Description

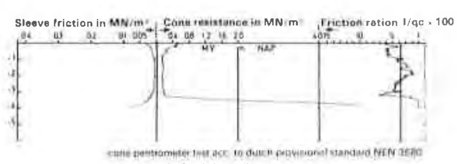

Fig. 3b Model of Cone Penetration Description

3.2.2 Both the reinforced section and the unreinforced reference section has a length of 60 m and a cross section as shown in figure 4.

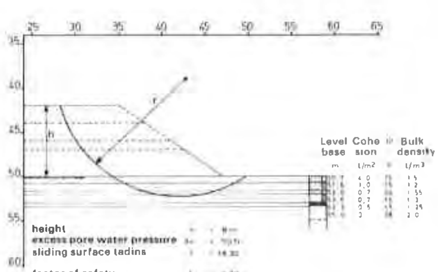

Fig. 4 Result of Stability Analysis with Ditch

In figure 4 it is also predicted that the safety factor (F) will become < 1 at a height of fill of > 3 m, assuming an excess pore water pressure (Δ u) of 50%.

3.2.3 The reinforcing fabric chosen was Stabilenka 200, which is a polyester reinforcing fabric characterized by a longitudinal stress-strain relation as indicated in figure 5.

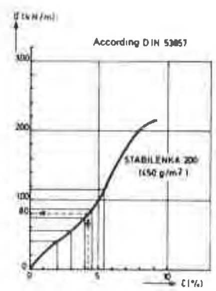

Fig. 5 Stress-strain Curves of Reinforcing Fabrics

This fabric was laid on ground level on lengths of 25 m in load-transmitting direction with 30 cm overlap (photo 1).

Photo 1

Arrangement of reinforced experimental section, north-south. Relieving trench on left, retaining bank at centre, reinforcing fabric on right (fabric continues under bank). In the background is blank experimental section I. A "neutral zone" separates it from the reinforced section.

Figure 6 shows the arrangement of the pore water pressure gauges and the strain gauges in the reinforced section. The pore water pressure gauges were arranged in the same way in the reference section (minus strain gauges).

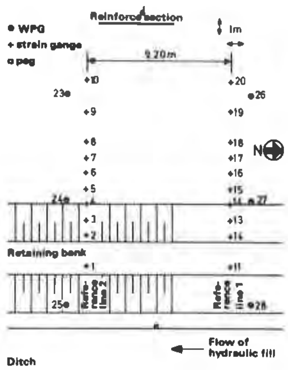

Fig. 6 Location of Strain Gauges and Water Pressure Gauges in Reinforced Section

3.3 Measured Results and Visual Observations

3.3.1 Hydraulic filling in the reinforced section started on Wednesday, October 3, 1979 at 09.00 a.m. 24 hours later the excess pore water pressure in reference had already attained its maximum value (adaptation <5%), as is shown in figures 7a and 7b. The level of fill was then approx. 2.75 m above ground level.

Nevertheless, the fill remained stable, although in 24 hours the retaining bank shifted 1 m towards the ditch. During this time tensile stresses up to 95 kN/m were measured in the fabric (see figure 7a).

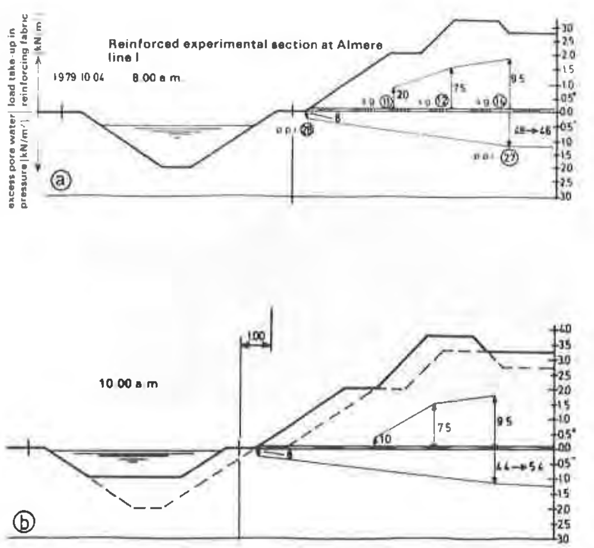

Fig. 7 Reinforced Section at Reference Line I

3.4 Measurements and Calculations

The stability calculations for the reinforced sections were based on the failure modes shown in figure 2. This means that the conditions have been considered for maintaining:

- equilibrium in case of circular sliding, and
- horizontal equilibrium.

3.4.1 Calculation of Equilibrium against Circular Sliding

On the basis of the geometry and measured values as indicated in figure 7b the Soil Engineering Bureau Fugro calculated the contribution to be made by the reinforcing fabric to prevent circular sliding (2). The result was that the fabric had to provide an effective counteracting force of 80 kN/m tangent to the sliding, which is in fairly good agreement with the actually measured load take-up in the fabric of approx. 100 kN/m in the horizontal plane (figure 8).

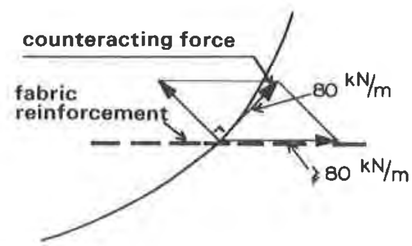

Fig. 8 Schemetic Presentation of Forces acting at the Intersection of Fabric and Surface of Sliding

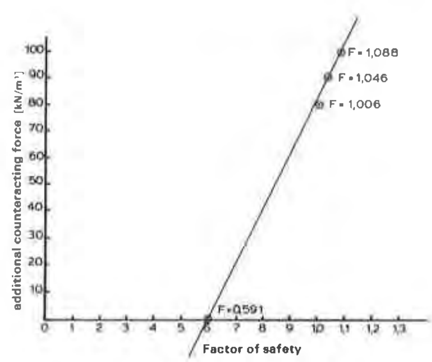

Fig. 9 Relation between Factor of Safety and Counteracting Force, based on an Embankment Height of 3.5 m

3.4.2 Calculation for maintaining horizontal Equilibrium

Here an attempt is made to explain why the embankment itself was still stable on October 4, 1979 at 08.00 a.m. despite an ongoing horizontal displacement of the retaining bank and simultaneous squeezing-out of the soft subsoil. Photo 2, taken at 09.45 a.m. on October 4, 1979, shows the situation at reference line I.

In the reinforcing fabric there was then a maximum tensile stress of 95 kN/m (figure 7a) opposite to the maximum difference between driving and counteracting forces in the embankment and subsoil on the hatched mass of soil as shown in figure 10.

Photo 2

Bottom of ditch heaves forcibly at reference line I, north-south, at 09.45 a.m.

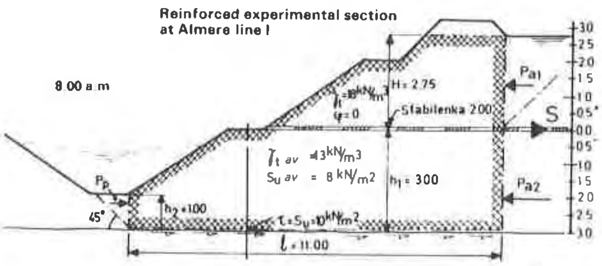

Fig. 10 Reinforced Section at Reference Line I

To maintain horizontal equilibrium (F > 1) it should then hold:

$P_{\Delta_1} + P_{\Delta_2} - P_p - T.1 < S = 90$ kN.

This is corroborated by the calculations made beforehand (7).

So in this case the calculation was based on total stresses (S_u - analysis), assuming that $\phi = 0$ (5), which is justified because at reference line I the fill was impassable (not fit to be walked on during hours) and the excess pore water pressures in the subsoil were at their maximum.

Unfortunately the total displacement of the retaining bank by about 1 m on October 4, 1979 at 10.00 a.m. cannot be explained from the actually measured elongations of the reinforcing fabric. In this section many strain gauges had already failed at an early stage on the inside of the retaining bank. A plausible explanation is that the measured maximum tensile stress of 95 kN/m at approx. 8 m distance from the toe of the retaining bank also existed approx. 10 m further inwards, which is in agreement with the failure mode according to figure 2b.

The reinforced section finally failed on October 4, 1979 at about 11.00 a.m., due to circular sliding (photo 3). When the fabric was dug out, it was found that the fabric had torn but that the anchor length on either side of the rupture had been sufficient.

3.4.3 The unreinforced section failed between 7 and 8 p.m. on October 8, 1979 at approx. 12 hours after the start of hydraulic filling, so much earlier than the reinforced section. At the time of failure the fill had a height of 1.75 m above ground level. This was in agreement with the measurements and calculations made according to the failure mode shown in figure 2b (7).

4. CONCLUSIONS

4.1 The application of a reinforcing fabric with a low elongation at break increases the stability of a slope.

4.2 The results of the calculation method employed show a good agreement with the observations.

4.3 Just as in the Cuxhaven experiment, the deformations of the reinforcing fabric could without difficulty be measured with the aid of strain gauges.

Photo 3

Failure of retaining bank due to sliding along a circular surface, north-south, at 11.00 a.m., at reference line I. Note the start of this surface of sliding at the crest of the retaining bank. The hydraulic placing pipeline had now suddenly come into view.

5. RELEVANT MATERIAL PROPERTIES FOR WOVEN POLYESTER REINFORCING FABRICS

So far woven polyester reinforcing fabrics in civil engineering have mainly been used for the applications depicted in figures 1a and 1b. With the aid of currently available mathematical models, it can be established that in the cross section and geometry according to figure 1a the fabrics must be capable of taking up effective loads of:

100 to 200 kN/m' (2), (3), (4), for a period of 2 or 5 years at a local initial elongation of 3 to 5%

and in the situation shown in figure 1b:

30 to 50 kN/m' (4), for a period of 50 years or more at a local initial elongation of 2 to 3%

In view of the above requirements for fabrics as constructive elements the following material properties and characteristics are of great importance:

5.1 Stress-strain relation and modulus of elasticity.
5.2 Behaviour under permanent load:
- creep,
- relaxation.

5.3 Chemical resistance in soil.

5.4 Coefficient of friction in cohesive and non-cohesive soil.

5.5 Vulnerability during storage and installation.

5.6 Weight and dimensions of reinforcing fabrics (5.5 and 5.6 concern typical execution aspects).

In specific cases the following properties may play a role:
- separating efficiency,
- permeability to water,
- impermeability to sand.

6. TEST METHODS

6.1 For the determination of the stress-strain relation of woven reinforcing fabrics there is only one single routine method, notably the gauge test. According to this method, narrow - 5 cm wide standard - strips are inserted in special fabric clamps and subjected to a breaking load in approx. 30 seconds. This method is similar to the existing methods and equipment used for the testing of (technical) fabrics (DIN 53857, AFNOR G 07-001, ASTM D 1682).

6.2 In principle, it is possible to determine creep and relaxation on the equipment referred to under 6.1, but long-term tests on tensile testers are not realistic. For an indication relaxation can be determined in only a few days' to one week's time, whereas creep tests may require 5 or even 10 years - depending on the basic material - before statistically justified predictions for an active period of 50 or more years can be made ! Therefore static creep tests should be made. In these tests the fabric specimens are loaded with weights to 25 or 50% of their breaking load after which the behaviour in time is observed in a representative environment. Afterwards, e.g. after a period of two years, such pre-stressed specimens can, moreover, still be subjected to breaking load by the method described under 6.1, to determine the residual breaking strength; in this way relevant information is obtained.

6.3 For the determination of the chemical resistance in soil the procedure described in 6.2 can be followed. The specimens are subjected for a protracted period of time to an acid or alkaline medium after which the residual tensile strength is determined. A pH range between 2 and 8 is considered to give sufficient information.

6.4 To determine the coefficient of friction in cohesive and non-cohesive soil a horizontal sliding test is most appropriate. The apparatus to be used should be 5 to 10 times as large as the conventional equipment (used to determine the horizontal sliding resistance of sand) so as to avoid edge effects.

6.5 The vulnerability during storage, e.g. resistance to UV light is determined in the same way as described under 6.2. Since reinforcing mats are not exposed to daylight for a period longer than six months, the duration of the test can be shortened accordingly.

6.6 As to the vulnerability during installation (damages caused by heavy traffic, compacting equipment, construction materials) there are certain limitations. If the top layer is thin, crushed sharp minerals may damage the fabric. To determine the risidual tensile strength or further elongation potential the fabric should be dug up and subjected to the test described in 6.1.

6.7 With regard to the vulnerability to other mechanical damages it should be noted that a local perforation or notch does not reduce the tearing strength of the relative loose woven reinforcing fabrics made from filament yarns.
Although there are a dozen measurements methods in use, it is more important to ascertain whether a "natural" tear stop will form in the fabric through reorientation of the yarn bundles. This can easily be established by pushing, for instance, a ball-point pen through the fabric.

6.8 For this kind of reinforcing fabrics the m²-weight or specific weight can be determined by the usual methods.

6.9 Properties such as permeability to water, impermeability to sand and separating capacity of geotextiles are routinely determined by the Delft Hydraulics Laboratory, Voorst, The Netherlands.

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Strength Properties Measurement for Practical Applications

Le mesure des caractéristiques de traction en vue des appplications pratiques

RESUME

Pour la qualification des géotextiles le Comité Français des Géotextiles a choisi l'essai de traction simple sur éprouvette large (500 x 100 mm). Cet essai est assez simple pour être largement utilisé, facilement normalisé et reproductible. Les dimensions de l'éprouvette sont choisies pour avoir une dimension minimale de 100 mm et un rapport largeur/hauteur de 5, ce qui est nécessaire pour ne pas sous-estimer systématiquement la résistance et l'allongement à la rupture, ce qui est observé aussi bien pour les tissés que pour les non-tissés lorsque ce rapport est plus faible, ainsi que le montrent de très nombreux essais. L'allongement à la rupture est corrigé pour tenir compte de la déformation transversale ; ainsi, la non-possibilité de réorganisation des fibres due dans la réalité à la présence du sol est, pour l'essentiel, prise en compte. La communication décrit aussi les améliorations apportées récemment à l'essai sur manchon cylindrique de Saint-Brieuc, qui est l'essai qui donne le maximum d'information sur le comportement d'un textile.

ABSTRACT

For qualification of geotextiles the French Committee of Geotextiles has chosen the wide sample (500 x 100 mm) simple traction test. This test is considered as simple enough to be widely used, easily standardized and reproducible. The size of the sample is chosen as to have 100 mm for the lowest dimension and a width/heigt ratio of 5, which is necessary to avoid a systematic under evaluation of strength and elongation at failure, which is observed both for wovens and non-wovens if this ratio has a smaller value, as shown by a large number of tests. The elongation at failure is corrected to take into account the transverse deformation ; in this way, the non-possibility of reorganization of fibers in the fabric due to the presence of soil in actual performance is, for the largest part, taken into account by this test. The paper describes also the recent improvements given to the Saint-Brieuc cylinder test, which is the test that gives the maximum of information on the behaviour of a fabric.

1 - OBJECTIF

Il s'agit de définir des conditions d'essai normalisées qui reproduisent les sollicitations les plus fréquemment appliquées aux géotextiles et dont les résultats soient directement utilisables par le plus grand nombre d'utilisateurs, c'est-à-dire qui conviennent à la fois aux applications empiriques et aux calculs d'ouvrages. On doit donc trouver un compromis entre la simplicité de l'essai et son aptitude à fournir des résultats interprétables sur le plan mécanique.

L'essai choisi sera utilisé pour la qualification des produits ; il n'exclut pas que des essais plus élaborés soient réalisés pour des besoins particuliers.

2 - CRITERES DE CHOIX D'UN ESSAI

a) Simplicité

Bien que le contrôle courant d'une livraison ne fasse intervenir que les caractéristiques d'identification, il faut que le contrôle direct des performances en traction soit possible pour l'utilisateur avec un délai et un coût acceptables. L'essai devra donc être assez simple pour qu'un nombre suffisant de laboratoires d'essai de matériaux soit en mesure de l'exécuter.

b) Mode de sollicitation

Les conditions de l'essai doivent se rapprocher suffisamment de la situation réelle que l'on rencontre le plus fréquemment dans la pratique qui est celle d'une nappe placée dans le sol et soumise à un effort de traction dans le sens production ou dans le sens travers. La caractéristique principale de cette situation est que la déformation transversale du géotextile est empêchée. L'essai de traction doit donc être conçu de telle sorte que la déformation transversale soit aussi faible que possible.

Un autre aspect de la situation envisagée est la pression exercée par le sol dans le sens de l'épaisseur du géotextile, qui peut influencer son comportement mécanique. Nous verrons que cet effet n'est, semble-t-il, que secondaire si la première condition (déformation transversale très faible) est satisfaite.

c) Reproductibilité

Cette condition n'est pas particulière aux essais de traction sur géotextiles ; elle met beaucoup en cause la simplicité de l'essai (principe et appareillage) pour qu'il y ait le moins possible de sources de dispersion dans la manipulation et que les appareils d'essai soient bien semblables d'un laboratoire à l'autre.

d) Dimensions de l'éprouvette

L'éprouvette doit être assez grande pour que les phénomènes dont elle est le siège lors de l'essai soient les mêmes que ceux qui se produisent en grandeur réelle.

Il semble qu'un large accord se fasse sur la valeur minimale de 100 mm pour la plus petite dimension d'une éprouvette soumise à un essai de traction.

3 - ESSAI CHOISI PAR LE COMITE FRANCAIS DES GEOTEXTILES

L'essai choisi en France est l'essai de traction simple sur éprouvette de 500 mm de largeur avec distance entre pinces (hauteur libre de l'éprouvette) de 100 mm. La vitesse de traction doit être comprise entre 10 et 100 % par minute. On entend par traction simple le fait que l'appareil d'essai exerce sur l'éprouvette un effort de traction dans une seule direction. Le principe de cet essai est aussi simple que possible, la déformation transversale de l'éprouvette y est très faible, sa conception donne les meilleures garanties de reproductibilité et il satisfait à la condition de dimension minimale de 100 mm.

Ce choix n'exclut pas d'avoir recours à d'autres modes de sollicitation lorsqu'une information spéciale est jugée nécessaire. A cet égard l'essai sur manchon cylindrique apporte des informations que la plupart des autres essais ne peuvent donner ; c'est la raison pour laquelle le dernier paragraphe de cette communication lui est consacré.

4 - RAISONS DU CHOIX DE L'ESSAI DE TRACTION SIMPLE SUR EPROUVETTE 500 mm x 100 mm

a) Choix d'un essai de traction simple

La première raison de ce choix est que l'on désire obtenir des valeurs numériques interprétables sur le plan mécanique pour pouvoir être utilisées dans des calculs. Il en résulte que les essais circulaires (éclatomètre et poinçonnement sur moule CBR) et les essais du type "grab test" ne conviennent pas.

Les essais circulaires ne peuvent pas donner d'information sur le comportement des matériaux dans leurs différentes directions ; de plus un allongement à la rupture ni un module ne peuvent en être déduits aisément. Le "grab test" ne peut donner aucune valeur numérique utilisable dans un calcul en raison de la très grande hétérogénéité des contraintes et des déformations dans l'éprouvette.

L'autre raison du choix de l'essai de traction simple est que les autres essais qui permettent d'obtenir des valeurs numériques utilisables sont trop complexes pour être économiques, rapidement réalisables et faciles à normaliser. Ce sont notamment l'essai sur manchon cylindrique, l'éclatomètre rectangulaire, l'essai en croix, l'essai biaxial avec mâchoires multiples et l'essai de traction plane avec pression de sol.

Il ne reste alors que l'essai de traction simple sur échantillon plan rectangulaire. La question qui reste posée est le choix des dimensions de l'éprouvette.

Remarque : La solution "réglettes à picots" (1) n'a pas été retenue en France car elle présente quelques inconvénients :

- difficulté d'application à certains matériaux,
- mise en oeuvre de l'essai un peu plus difficile,
- incertitude de l'effet des picots sur certains matériaux.

b) Choix des dimensions 500 mm x 100 mm

On a d'abord admis que la plus petite dimension d'une éprouvette doit être d'au moins 100 mm. Ce point étant acquis la question qui reste à trancher est celle du rapport largeur b/hauteur h.

On a alors fait des essais sur des tissés et des non-tissés avec différentes valeurs du rapport b/h (en prenant un éventail de valeurs de b et un éventail de valeurs de h) et on a examiné dans un premier temps la résistance maximale à la traction F (kN/m) et l'allongement à la rupture mesuré ε_1 ; on a également noté les déformations transversales ε_2 et la zone dans laquelle apparaît la rupture.

Dans un deuxième temps on a calculé pour ces essais l'allongement à la rupture corrigé ε_R (voir plus loin définition de ce paramètre) et on a comparé les résultats obtenus à ceux d'essais sur manchon cylindrique et d'essais avec pression de sol.

Le nombre des essais de traction réalisés dans le cadre de cette étude a été de plusieurs centaines, effectués pour l'essentiel par l'Institut Textile de France.

c) Résistance maximale à la traction F

Les résultats obtenus sur deux tissés, deux non-tissés aiguilletés (l'un en polyester, l'autre en polypropylène) et deux non-tissés thermosoudés (également en polyester et en polypropylène) sont reportés sur la figure 1. Les géotextiles testés proviennent de six producteurs différents.

On voit sur la figure 1 que F augmente en fonction de b/h pour tous les matériaux, tissés et non-tissés. La valeur de l'augmentation de F est donnée par le tableau A (Remarque : les masses surfaciques des matériaux étant différentes, il n'y a pas lieu de comparer entre elles les valeurs absolues des résistances obtenues).

On constate que F augmente en moyenne de 14,5 % lorsque l'éprouvette passe de la forme carrée (b/h = 1) à une forme beaucoup plus large (b/h = 5). Cette augmentation résulte en réalité de deux effets qui se cumulent car ces résultats ont trait à des éprouvettes où la hauteur est différente (la moyenne des valeurs de h étant plus faible pour les éprouvettes où la valeur de b/h est la plus élevée). Pour h constant = 100 mm, on constate (2e colonne du tableau A) : que : (i) l'augmentation de F due uniquement à la variation de forme est de 9,5 %, ce qui est encore important ; (ii) il y a en conséquence un effet d'échelle, à savoir une certaine augmentation (de l'ordre de 5 % en moyenne) de F lorsque h diminue de 100 mm à 50 mm ; (iii) l'augmentation de F en fonction de b/h lorsque h reste constant est de plus en plus faible lorsque b/h augmente, ce qui est nettement confirmé par les résultats de Rigo et Perfetti (2) sur un non-tissé aiguilleté (augmentation de 15 % quand b passe de 100 à 500 mm, pas de variation significative entre 500 et 800 mm).

La dernière colonne du tableau A indique qu'en moyenne l'augmentation de F quand b/h passe de 2 à 5 est encore de 8,3 %. On peut donc conclure qu'un rapport b/h de l'ordre de 5 est préférable à un rapport plus faible si l'on ne veut pas sous-estimer systématiquement le résistance des produits testés, y compris celle des tissés, contrairement à un point de vue assez fréquemment exprimé au sujet de ces derniers.

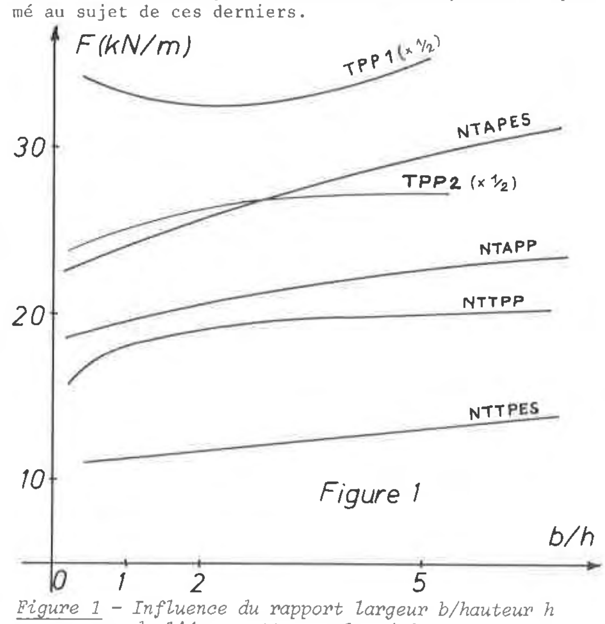


Figure 1 - Influence du rapport largeur b/hauteur h de l'éprouvette sur la résistance F.

	1	2	3
NTAPES	22,7	13,8	15,0
NTAPP	15,8	6,7	9,7
NTTPES	18,0	14,9	11,0
NTTPP	12,2	3,6	5,2
TPP 1	6,0	5,8	7,7
TPP 2	12,4	12,4	1
Moyenne	14,5	9,5	8,3

TABLEAU A - Augmentation en % de la résistance à la traction F de 6 géotextiles lorsque le rapport larbeur b/hauteur h de l'éprouvette varie.
1. b/h passe de 1 à 5 (b et h variables)
2. " " " (h = constante = 100 mm)
3. b/h passe de 2 à 5 (b et h variables).
NTAPES : non-tissé aiguilleté polyester
NTAPP : " " " polypropylène
NTTPES : " " thermosoudé polyester
NTTPP : " " " polypropylène
TPP : tissé polypropylène.

d) Allongement à la rupture mesuré ε_1

Les résultats sont présentés sur la figure 2. Cette figure montre que pour l'ensemble des matériaux ε_1 augmente lorsque b/h croît. Une partie de cette augmentation est également due à un effet d'échelle, ε_1 étant plus élevé lorsque h diminue, à forme d'éprouvette constante. A h constant l'augmentation relative de ε_1 quand b/h passe de 1 à 5 peut néanmoins être de 10 ou 20 % et atteint 40 % pour le non-tissé thermosoudé en polypropylène. On constate donc là encore qu'il est nécessaire de choisir un rapport b/h assez élevé si on ne veut pas obtenir une valeur trop faible de l'allongement à la rupture.

Sans entrer dans une analyse détaillée, on peut indiquer que l'augmentation conjointe de F et de ε_1 correspond à une rupture plus tardive de l'éprouvette par rapport à ce qui se produit dans le cas d'une éprouvette étroite où l'hétérogénéité des sollicitations entraîne une rupture prématurée.

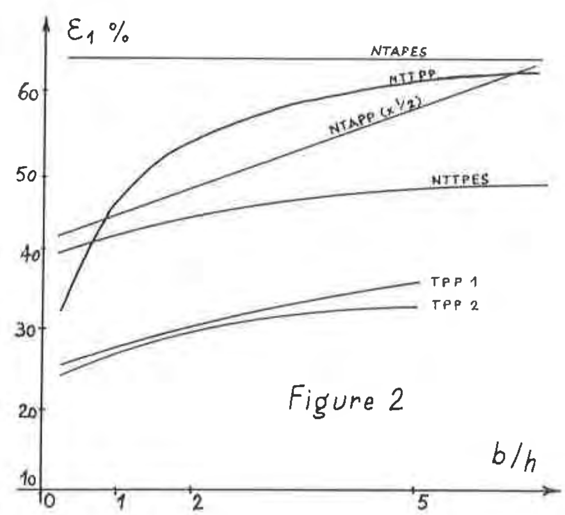


Figure 2 - Influence du rapport largeur b/hauteur h de l'éprouvette sur l'allongement à la rupture ε_1.

e) Déformations transversales et zone de rupture

La déformation transversale ε_2 au milieu de l'éprouvette a été mesurée à la rupture ; de plus, pour de nombreux essais, ε_2 a été notée au cours de l'essai. Les valeurs à la rupture sont données par le tableau B. ε_2 est d'autant plus faible, pour un matériau donne, que b/h est élevé.

Rapport b/h	0,25	1	2	2,5	5
NTAPES	68	52		22	12
NTAPP	78	55		26	15
NTTPES	50	20			6
NTTPP	20	15	12		4
TPP 1	19	15	9		2
TPP 2		6	5		2

TABLEAU B - Valeurs moyennes de la déformation transversale ε_2 en fonction du rapport b/h pour six géotextiles.

On peut noter (i) les valeurs élevées de ε_2 pour b/h = 0,25 pour les non-tissés aiguilletés (68 et 78 %) et pour un des thermosoudés (50 %) ; (ii) l'écart important entre les valeurs obtenues pour b/h = 2 ou 2,5 et b/h = 5 ; (iii) les valeurs obtenues pour le tissé N° 1.

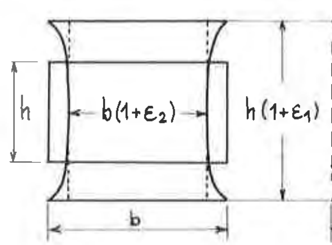
Au sujet de la zone de rupture, les essais ont montré que pour l'ensemble des matériaux la rupture n'apparaît dans la zone centrale de l'éprouvette que si b/h est assez grand, au moins de l'ordre de 2 et en général supérieur (observation notée aussi dans (2)). Lorsque b/h est faible ou moyen on a en effet une rupture au voisinage des mors ou une rupture prenant naissance sur un des côtés de l'éprouvette.

f) Allongement à la rupture corrigé ε_R et variations de surface de l'éprouvette en cours d'essai

L'application d'un effort de traction à une éprouvette textile conduit simultanément à un allongement des fibres et à une réorganisation du matériau. L'allongement des fibres tend à produire une augmentation de surface de l'éprouvette et la réorganisation tend généralement vers une réduction de cette surface. Le bilan des deux phénomènes peut être une augmentation ou une réduction de la surface.

La réorganisation du matériau dépend, dans un essai de traction simple, de la forme de l'éprouvette. Ceci est mis en évidence par la grande dépendance de ε_2 par rapport à b/h (tableau B).

L'objectif poursuivi est de choisir un essai où la déformation transversale soit la plus faible possible, c'est-à-dire un essai tel que la réduction de surface de l'éprouvette due à la réorganisation du matériau ne puisse pas se produire, autant que faire se peut. Il est donc intéressant d'examiner la variation de surface des éprouvettes soumises aux essais de traction en fonction de b/h.



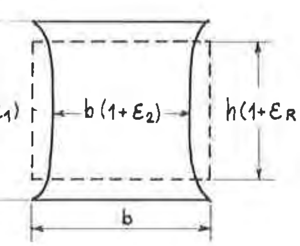


Figure 3

Variation de surface d'une éprouvette dans un essai de traction
S initiale A_o = hb
S finale approchée
$A = (h(1+\varepsilon_1)\cdot b(1+\varepsilon_2)$

Figure 4

Définition de l'allongement corrigé ε_R
$h(1+\varepsilon_1)\cdot b(1+\varepsilon_2)=h(1+\varepsilon_R)\cdot b$
$\varepsilon_R = \varepsilon_1 + \varepsilon_2 + \varepsilon_1\varepsilon_2$

La surface initiale de l'éprouvette est A_o = hb. La surface finale A peut être évaluée approximativement (en négligeant les coins déformés de l'éprouvette) par l'expression (figure 3) :

$$A = h\ (1+\varepsilon_1)\cdot b\ (1+\varepsilon_2)$$

et le rapport $a = A/A_o = (1+\varepsilon_1)(1+\varepsilon_2)$

(ε_1 et ε_2 sont comptés positivement pour un allongement et négativement dans le cas contraire).

Si l'éprouvette n'avait pas manifesté de déformation transversale (ε_2 = 0) et si sa surface à la rupture avait été la même, son allongement à la rupture ε_R aurait été, par définition, donné par la relation (figure 4) :

$$h\ (1+\varepsilon_1)\cdot b\ (1+\varepsilon_2) = h\ (1+\varepsilon_R)\cdot b$$

ε_R est un allongement à la rupture corrigé conventionnel, qui est l'allongement à la rupture qu'aurait eu l'éprouvette si sa déformation transversale ε_2 avait été nulle avec la même surface à la rupture. On peut écrire :

$$(1+\varepsilon_1)\ (1+\varepsilon_2) = 1 + \varepsilon_R = A/A_o = a.$$

On peut donc utiliser indifféremment le paramètre ε_R et le paramètre a donnant la variation de surface de l'éprouvette entre l'état initial et l'état de rupture.

Les variations de $a = 1+\varepsilon_R$ en fonction de b/h pour les six géotextiles étudiés sont données sur la figure 5. On constate que pour b/h faible a est très inférieur à 1 pour plusieurs produits et voisin de 1 pour d'autres ; lorsque b/h augmente a est croissant pour tous les matériaux et n'atteint une limite que pour les valeurs de b/h assez élevées, de l'ordre de 4 ou 5.

On conclut donc aussi de l'examen de ce paramètre qu'une valeur de b/h élevée est nécessaire pour que tous les produits soient mis à l'épreuve dans des conditions qui mettent en jeu toute leur capacité de résistance et qui reproduisent au mieux la non-possibilité de réorganisation de la structure du matériau qu'impose dans la réalité la présence du sol.

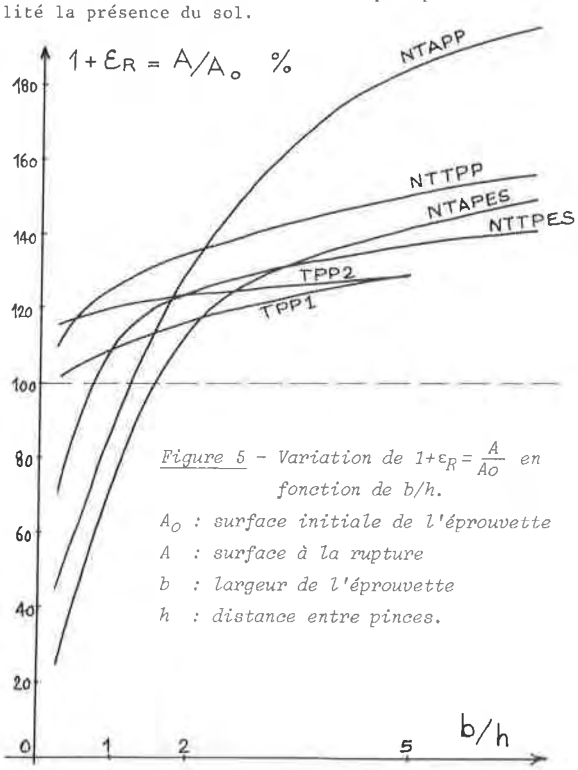


Figure 5 - Variation de $1+\varepsilon_R = \frac{A}{Ao}$ en fonction de b/h.

A_o : surface initiale de l'éprouvette
A : surface à la rupture
b : largeur de l'éprouvette
h : distance entre pinces.

5 - INTERPRETATION DES ESSAIS DE TRACTION

Un mode opératoire étant choisi, il faut définir les paramètres que l'on utilise pour exprimer les résultats à partir de la courbe effort-déformation obtenue. Pour une expression chiffrée et synthétique des résultats le Comité Français des Géotextiles a choisi les valeurs de la résistance à la rupture et de l'allongement maximal à la rupture exprimé par l'allongement corrigé ε_R. Ces choix sont commentés ci-après.

a) Choix des caractéristiques à la rupture

Les valeurs à la rupture sont choisies d'abord par souci de simplicité : leur signification physique essentielle est en effet immédiatement accessible sans ambiguité à tous les utilisateurs. Elles sont choisies par ailleurs pour permettre le calcul de la sécurité à la rupture, calcul souvent fait dans les ouvrages en terre.

La seule connaissance des valeurs à la rupture peut cependant être insuffisante dans certains cas ; c'est ainsi que pour des calculs en petites déformations la connaissance d'un module est nécessaire. Pour certains matériaux l'approximation linéaire entre l'origine et le point de rupture fournit une valeur du module tout à fait acceptable, mais pour d'autres elle introduit une erreur importante. Dans ce dernier cas il faut utiliser la courbe effort-déformation.

En revanche l'introduction de la notion de module pour l'évaluation courante des géotextiles présenterait des difficultés et des risques. On peut en effet choisir plusieurs modules. Le plus simple est le module apparent de chaque nappe calculé par unité de largeur et exprimé en kN/m. Il dépend de la masse surfacique et n'est donc pas une caractéristique intrinsèque d'un type de produit ou de fabrication, à l'opposé des modules des autres matériaux de construction. Des confusions sont donc à craindre dès que l'on voudra faire des comparaisons.

Une définition du module qui éviterait cet inconvénient serait le module apparent par unité de masse surfacique ; cette notion est cependant moins simple et pourrait aussi être source de confusion.

Une troisième possibilité serait d'exprimer le module par une contrainte (force par unité de section) comme pour les autres matériaux de construction. Le calcul de ce module ferait alors intervenir l'épaisseur ; il conduirait donc à des valeurs, fonction de la porosité et de l'état de compression. L'introduction de ce module-là ne paraît pas souhaitable.

La notion de module pour les géotextiles n'est donc pas facile à appréhender et il est souhaitable d'en limiter l'emploi au niveau des bureaux d'études spécialisés sans en introduire l'usage parmi les utilisateurs courants. C'est la forte porosité et la compressibilité des géotextiles qui est la cause de ce que la notion habituelle de module utilisée pour les autres matériaux leur est difficilement applicable.

b) Choix de l'allongement à la rupture corrigé ε_R

Ce choix apporte une meilleure cohérence des résultats sur le plan pratique ; de plus le lien entre ε_R et la variation de surface de l'éprouvette lui donne une signification intéressante.

Sur le plan pratique les trois essais qui répondent aux conditions définies au début de cette communication et pour lesquels nous disposons de résultats chiffrés précis sont l'essai sur manchon cylindrique de Saint-Brieuc (3), l'essai avec pression de sol de l'Université de Strathclyde (4) et l'essai de traction simple 500x100 mm.

Dans l'essai de Saint-Brieuc ε_2 est faible et légèrement positif. Les valeurs de ε_1 à la rupture obtenues dans cet essai sont systématiquement plus faibles que celles de l'essai de traction simple (Tableau C).

	1	2	3
NTAPES	64	52	50
NTAPP	120	85	79
NTTPP	63	52	50

TABLEAU C - Comparaison des allongements à la rupture.
1. ε_1 (mesuré) essai de traction directe b/h = 5
2. ε_R (corrigé) " " " " " "
3. ε_1 (mesuré) essai sur manchon cylindrique (St-Brieuc).

En revanche ils se rapprochent beaucoup des valeurs de ε_R. Dans le tableau C les allongements moyens à la rupture Saint-Brieuc pour les deux non-tissés de polypropylène se réfèrent à des produits du même type que pour l'essai de traction directe, mais de différentes masses surfaciques ; pour le non-tissé de polyester il s'agit du même produit ; le nombre d'essais Saint-Brieuc pour chacun des trois matériaux est respectivement de 29, 29 et 15.

On constate sur la tableau C que les valeurs de Saint-Brieuc sont proches de ε_R par défaut, ce qui est logique, alors que les valeurs brutes de ε_1 résultant de l'essai de traction directe sont beaucoup plus élevées. L'emploi de ε_R permet donc de rapprocher les résultats d'essais différents.

La comparaison avec les données tirées de l'essai avec pression de sol de l'Université de Strathclyde peut être faite pour un non-tissé (Bidim U24) pour lequel des résultats sont donnés dans la figure 22 de la référence (4) et pour lequel des essais avec b/h = 2 et b/h = 5 ont été faits à l'Institut Textile de France (Tableau D).

Essai Univ. Strathclyde		Essai Inst. Text. France	
Module sécant à ε = 20 %		Module sécant rupture	
	kN/m	kN/m	
Pression du sol = 0 b = 200 h = 100	22,5	25	Module non corrigé b = 200 h = 100
		27,9	Module non corrigé b = 500 h = 100
Pression du sol = 100 kN/m² b = 200 h = 100	36,5	39,5	Module corrigé (d'après ε_R) b = 500 h = 100

Tableau D - Comparaison des modules (Bidim U24)

Le Bidim U24 est choisi car l'effet de la pression du sol est maximum pour les aiguilletés et car la linéarité du produit permet de comparer le module à 20 % d'allongement et le module à la rupture.

On constate que le module calculé à partir de l'essai avec b/h = 5 en tenant compte de l'allongement à la rupture corrigé ε_R est voisin du module donné par l'essai avec pression de sol. Cela correspond au fait que l'effet de la pression du sol réside essentiellement dans la non-possibilité de réorganisation du matériau. La convergence des résultats obtenus justifie encore l'emploi de ε_R.

Enfin $1+\varepsilon_R$ = a représente la variation de surface de l'éprouvette entre le début et la fin de l'essai ; or cette variation de surface est elle-même une donnée physique intéressante : elle indique la plus ou moins grande aptitude du matériau à s'adapter à une certaine géométrie, en relation avec l'effort correspondant. A cet égard les essais de traction avec b/h faible présentent l'intérêt d'indiquer si le matériau peut diminuer de surface sous certaines sollicitations ; or l'aptitude à diminuer de surface est utile chaque fois que l'on souhaite recouvrir une surface irrégulière. On peut considérer que le meilleur géotextile est celui qui a des valeurs de a très inférieures à 1 pour b/h faible et qui tendent vers une limite pas trop élevée lorsque b/h devient grand. Il semble que ce critère d'appréciation permette à la fois de classer les matériaux en fonction de la nature du polymère de base et du mode de fabrication.

6 - DETERMINATION DES CARACTERISTIQUES DE RESISTANCE ET DE DEFORMABILITE DES TEXTILES PAR DILATATION D'UN MANCHON CYLINDRIQUE - ESSAI DE SAINT-BRIEUC

L'essai de dilatation d'un manchon cylindrique est relativement bien adapté à l'étude des textiles non-tissée. Nécessitant un appareillage spécifique, il ne peut être considéré comme un essai de routine, mais plutôt comme un moyen d'étude des caractéristiques générales de ces produits.

Le principe de l'essai est basé sur la théorie des cylindres à paroi mince (3). Un manchon cylindrique du textile analysé (Ø intérieur d_{io} = 10 cm, longueur utile H = 20 cm) confectionné par couture, est fixé aux extrémités sur les deux embases de la cellule d'essai. Le manchon est protégé sur la face interne par une membrane en caoutchouc. L'essai est réalisé en maintenant la pression extérieure (p_e) constante et égale à la pression atmosphérique et en injectant à l'intérieur du manchon de l'eau sous pression (p_i) par paliers successifs d'incrément constant maintenus chacun deux minutes. Pendant l'essai, on note les variations du volume intérieur ΔV correspondantes. La hauteur H du manchon est maintenue constante, donc l'allongement ε_z est nul et l'on note l'effort F de réaction sur l'embase supérieure.

Un avantage fondamental de cet essai est, par cette mesure de l'effort de réaction F sur l'embase supérieure, de permettre de déterminer l'ensemble des contraintes dans l'éprouvette soumise à l'essai et de calculer ainsi un coefficient de Poisson bidimensionnel pour un textile. Un autre avantage de l'appareillage est de permettre d'appliquer une pression normale au textile par le montage d'une membrane extérieure à l'éprouvette et en exerçant une pression supplémentaire dans la cellule. Compte tenu du fait que la déformation ε_z est empêchée, la déformation du manchon est cylindrique et l'on se trouve ainsi en condition de déformation plane.

Déformation verticale

En fait, il s'avère qu'en cours d'essai, le manchon ne reste pas cylindrique. Il se dilate en forme de tonneau. La hauteur entre les deux embases reste constante, mais du fait de la déformation du manchon, il n'en est pas de même de la longueur d'une génératrice. La déformation du textile ε_z selon une génératrice n'est pas nulle.

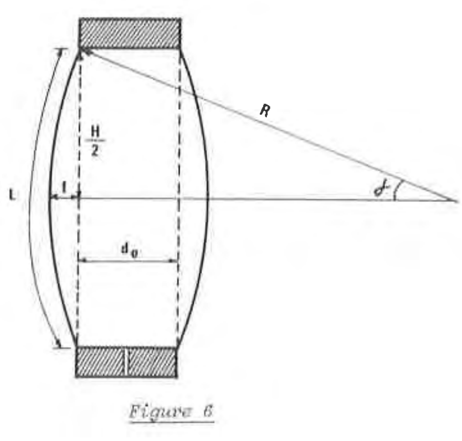


Figure 6

Si on considère (Fig. 6) un plan vertical passant par l'axe du manchon et la génératrice prenant la forme d'un arc de cercle et si :

H = longueur initiale de la génératrice
L = " de la génératrice à un moment donné
d_o = diamètre intérieur initial du manchon
d = $d_o + 2f$ = diamètre extérieur du manchon à un moment donné
f = flèche
R = rayon du cercle de déformation,

on a les relations suivantes :

$$R = \frac{f^2 + (H/2)^2}{2f} \qquad \alpha = \text{Arc tg} \frac{H}{2(R - \frac{d-d_o}{2})} .$$

La relation mathématique liant ε_z à ε_θ se traduit par la courbe 1 de la fig. 7 (ε_θ : déformation circonférentielle).

En résumé, selon la déformation du textile, nous avons les relations suivantes :

déformation en cylindre droit	déformation en tonneau
$d = \sqrt{d_o^2 + \frac{4\,\Delta V}{\pi H}}$	$d = \sqrt{d_o^2 + \frac{6\,\Delta V}{\pi H}}$

$$\varepsilon\theta = \frac{d}{d_o} - 1$$

$\varepsilon_z = 0$	$\varepsilon_z = \frac{L-H}{H} = \frac{2R\alpha}{H} - 1$

Modifications envisagées

Dans le but de diminuer l'importance de ε_z, nous avons modifié notre cellule d'essais de manière à tester des manchons cylindriques de même diamètre (10 cm) mais de longueurs utiles différentes : 20 cm, 40 cm et 60 cm, et comparer les valeurs des caractéristiques de résistance mécanique et de déformabilité. Le principe de l'essai reste absolument le même.

A titre d'exemple, les résultats obtenus sur un NTAPES montrent que les caractéristiques mesurées sont du même ordre quelle que soit la longueur utile du manchon, mais fait apparaître une très nette diminution de ε_z en fonction de la longueur de la génératrice.

	σ_θ	ε_θ	ε_z
H = 20	11,9	0,40	0,03
H = 42,2 cm	12,0	0,41	0,007
H = 60,2 cm	11,7	0,35	0,0025

Sur la figure 7 nous avons tracé les courbes donnant la relation existant entre ε_z et ε_θ pour les trois longueurs de manchons 20 cm, 40 cm et 60 cm.

Ces courbes montrent bien que quelle que soit la valeur de ε_θ, ε_z devient négligeable dès que la longueur des manchons dépasse 40 cm.

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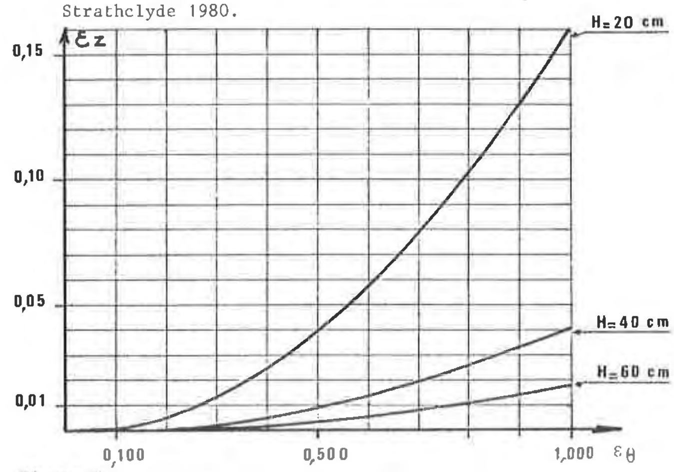

Figure 7 - Essai sur manchon cylindrique. Influence de la hauteur du manchon sur l'allongement axial ε_z.

Figure 8 - Essai sur manchon cylindrique de Saint-Brieuc. Manchon de 200 mm de hauteur. Vue d'une éprouvette de non-tissé au moment de la rupture.

Figure 9 - Essai sur manchon cylindrique de Saint-Brieuc. Manchon de 600 mm de hauteur en cours d'essai.

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A Wide Strip Tensile Test of Geotextiles

Essai de traction des géotextiles sur eprouvette large

A strip tensile test program studied the effects of test variables on laboratory measurement of tensile loads and strain. Considered were specimen size and aspect ratio, rate of strain, test methods, and sample variability. The specimen size is not very significant for ultimate load measurements. Moduli of geotextiles, however, varied significantly with specimen size and aspect ratio for non-woven fabrics. Effects of specimen width were insignificant for woven geotextiles.

On the basis of these data, a wide strip tensile test with specimen 200 mm (8 inches) wide and 100 mm (4 inches) gauge length is recommended for routine measurement of tensile properties. At this specimen size, the tensile load-strain plots of the six geotextiles tested were not significantly different from those obtained from plane-strain tests. Strain rates of 1-1/4 to 12 percent per minute did not show significant difference in the results and 10 percent is recommended. In view of the sample variability of geotextiles, a precision of ± 10% at 95% probability is recommended to provide acceptable accuracy.

L'influence des variables d'essais sur les mesures en laboratoire des efforts de tension et des propriétés à la déformation a ete etudiee. Les paramètres suivants ont ete considérés: dimension de l'éprouvette et rapport longueur sur largeur, taux de déformation, méthode employée ainsi que variabilité de l'échantillon. La taille de l'éprouvette est negligeable dans le cas des mesures à la charge de rupture. Par contre, dans le cas de géotextiles non tissés, les modules des géotextiles ont changé d'une manière significative avec la taille de l'éprouvette et le rapport longueur sur largeur. L'influence de la largeur de l'éprouvette a été negligeable pour les géotextiles tissés. Suivant ces résultats, on recommende une éprouvette large de 200 mm et longue de 100 mm. Avec ces dimensions, pour les six géotextiles utilisés, les courbes de l'effort de tension en fonction de la déformation n'apparaissent pas très différentes des courbes obtenues dans le cas d'experience en deux dimensions. Des Taux de déformation de 1-1/4 a 12% par minute n'ont pas montré de difference significative dans les résultats; et on recommende d'utiliser 10%. Une précision de ± 10% a 95% de probabilité est recommendée.

INTRODUCTION

The satisfactory field performances of geotextiles in civil engineering applications such as earth reinforcement depend to a large extent on the tensile loads and strain properties of the geotextiles. It is agreed unequivocally that the standard laboratory tensile tests used for textile fabrics do not represent the field conditions normally imposed on geotextiles. In many applications the geotextiles are stressed in plane-strain conditions. Therefore, the standard tensile tests used for textile fabrics are generally considered as "inappropriate and misleading" for geotextiles (1). Various new test methods including a hydraulic trough test with zero transverse strain (1), biaxial tensile tests (2), cylindrical sleeve tests (3), and others have been devised and suggested for laboratory evaluation of geotextiles. These test methods have the capability of duplicating the in-situ load and strain conditions of geotextiles; however, all are believed to be too complex for routine laboratory use.

This paper presents the results of a laboratory investigation to develop a wide strip tensile test appropriate for routine laboratory measurement of tensile stress and strain properties of geotextiles (4). A total of 383 tensile test runs were performed on six typical geotextiles to include test variables of specimen size, width to length ratio, rate of strain, plane-strain and uniaxial loading, and sample variability. The plane-strain condition was simulated by installing light wooden brackets set with pins to restrain the test specimen from contracting in the transverse direction (5).

TEST EQUIPMENTS AND PROCEDURE

The equipments used in this experimental work consisted of standard tensile testing machines, automatic load and elongation recorder and special wide jaws. The special jaws consisted of 229 mm (9 inches) wide by 57 mm (2-1/4 inches) high serrated faces. Specimens were trimmed to the final dimensions to a tolerance of ± 3 mm (1/8 inch). Each specimen was weighed to evaluate sample variabilities. The specimens were installed in the special jaws, mounted in the tensile test machine, and loaded at a constant rate of strain at laboratory temperature.

SAMPLE SELECTION

The type of geotextile construction has a great influence on the tensile load and strain characteristics. Therefore, several geotextile constructions were selected for testing as shown in Table 1.

Table 1. Geotextiles Investigated

Number	Fiber-Polymer	Construction
NW-1	Continuous Polyester	Nonwoven Resin bonded
NW-3	Continuous Polypropylene	Nonwoven Heat bonded
NW-5	Continuous Polypropylene	Nonwoven Needlepunched
NW-6	Staple Polypropylene	Nonwoven Needlepunched
W-4	Monofilament Polypropylene	Woven
C-1	Slit Film Polypropylene	Woven-needled nap

EXPERIMENTAL RESULTS

The testing program is listed in Table 2. Test variables were specimen size and aspect ratio (defined as specimen width to gauge length), rate of strain, test method, and sample variability.

Table 2. Strip Tensile Test Series

Series Number	Test Method	Width x Gauge Length (mm)	Strain Rate (% per minute)	Fabrics Tested	Specimens Each Test
1	Strip	200 x 25 MD	5	6	5
		200 x 50 MD	5	6	5
		200 x 100 MD	5	6	5
		200 x 200 MD	5	6	5
2	Strip	25 x 100 MD	5	6	5
		50 x 100 MD	5	6	5
		100 x 100 MD	5	6	5
		150 x 100 MD	5	6	5
3	Strip	200 x 100 MD	1-1/4	6	3+
		200 x 100 MD	12-1/2	6	5
4	Plane-Strain	200 x 100 MD	5	6	5

Notes: MD = Machine Direction
+Since the strain rate was very slow, only three specimens were tested for each fabric.
The laboratory room temperature and humidity were 68°F to 82°F and 20% to 58%.

Effect of Specimen Size and Aspect Ratio

Test Series 1 and 2 were performed to investigate the effect of aspect ratio on the tensile test results. In Test Series 1, specimen gauge length, defined as the initial length of fabric between the test jaws, was varied from 25 mm (1 inch) to 200 mm (8 inches) and the specimen width was constant at 200 mm (8 inches). In Test Series 2, the specimen width was varied from 25 mm (1 inch) to 200 mm (8 inches) and the specimen gauge length was invariant at 100 mm (4 inches). The results of the Series 1 and 2 tests are presented in Figures 1 to 10.

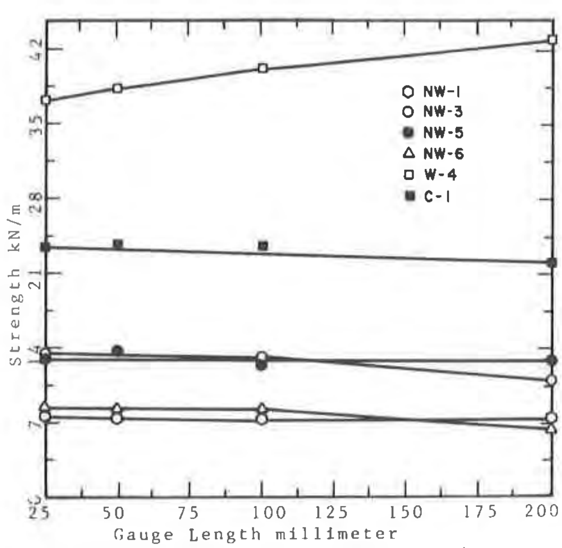

FIGURE 1: Strength vs. Gauge Length
Specimen Width 200 mm (8 inches)

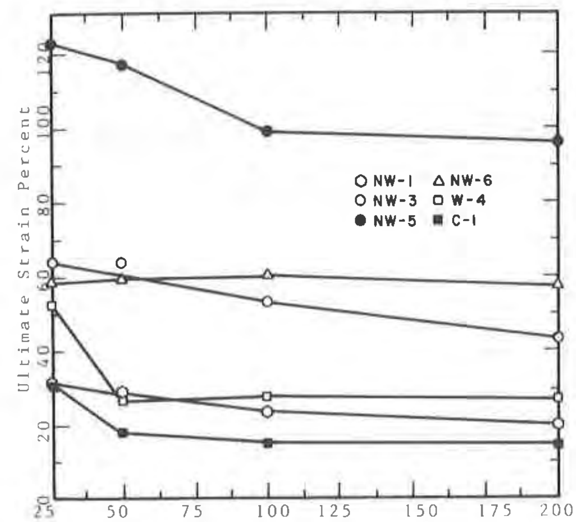

FIGURE 2: Ultimate Strain vs. Gauge length
Specimen Width 200 mm (8 inches)

Figures 1 and 2 show the ultimate loads (strength) and strains plotted as functions of specimen gauge lengths with specimen width constant at 200 mm (8 inches). The strength remained practically constant for all geotextiles, except for W-4 which showed about 14% increase with increase in gauge length. The ultimate strains generally decreased with increase in length. This decrease was most pronounced for very short specimens and for woven fabrics.

Figures 3 and 4 present loads at 10% strain and strains at 40% strength as functions of gauge lengths. The loads on the woven geotextiles increased by about 50% with increase in the gauge length up to 50 mm (2 inches) and then remained practically constant with further increases.

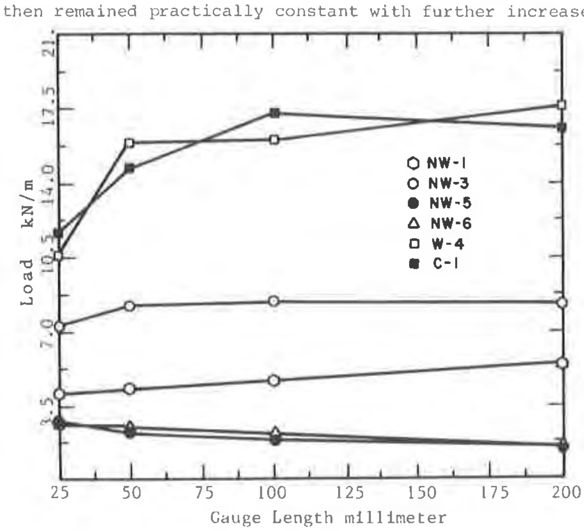

FIGURE .3: Load vs. Gauge Length.
Specimen width 200 mm (8 inches)
Strain = 10 percent

The loads for the nonwoven geotextiles showed less effects. The strains decreased with increase in specimen gauge length to 100 mm (4 inches) for the bonded nonwoven and woven geotextiles, however, for the needlepunched nonwovens, they increased.

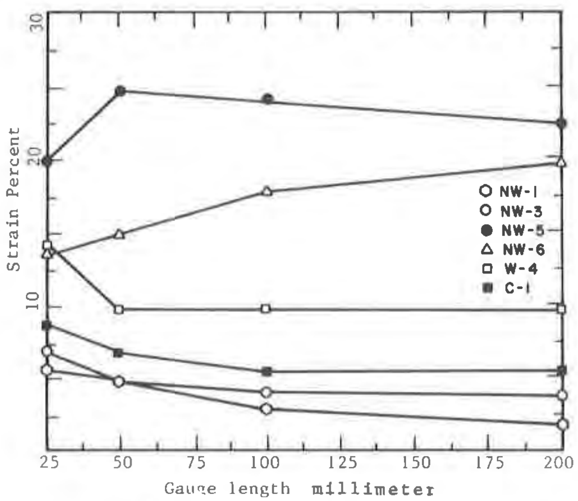

FIGURE .4: Strain vs. Gauge Length
Specimen width 200 mm (8 inches)
Load = 40% Strength

Figure 5 shows that ultimate loads (strengths) of geotextiles remain almost constant with increase in specimen width for the 100 mm (4 inches) wide specimens. However, Figure 6 shows that ultimate strain may be significantly affected depending on geotextile structure.

Figures 7 and 8 show plots of stresses at 10% strain and strains at 40% strength respectively as functions of

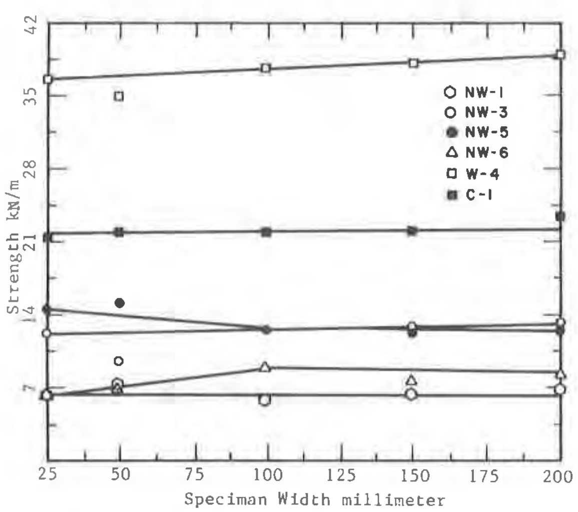

FIGURE .5: Strength vs. Specimen Width.
Gauge length 100 mm (4 inches)

specimen width. The stresses of the nonwovens increased by 5% to 100% with increase in specimen width up to 100 mm (4 inches). For wider specimens, the stresses remained practically constant. For the wovens, the initial stresses remained practically constant at all specimen widths. The strains of the needlepunched nonwovens de-

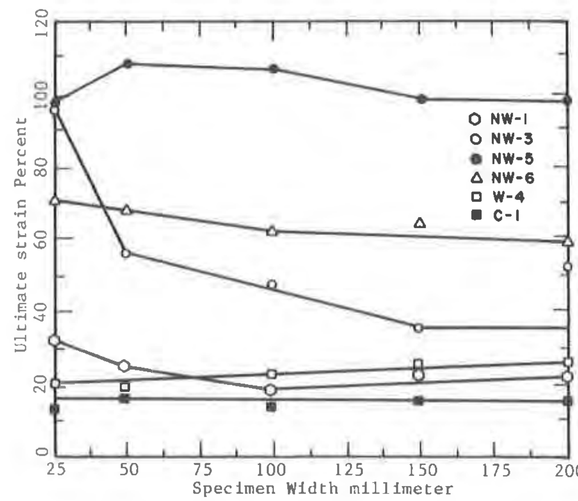

FIGURE .6: Ultimate strain vs. Specimen Width.
Gauge length 100 mm (4 inches)

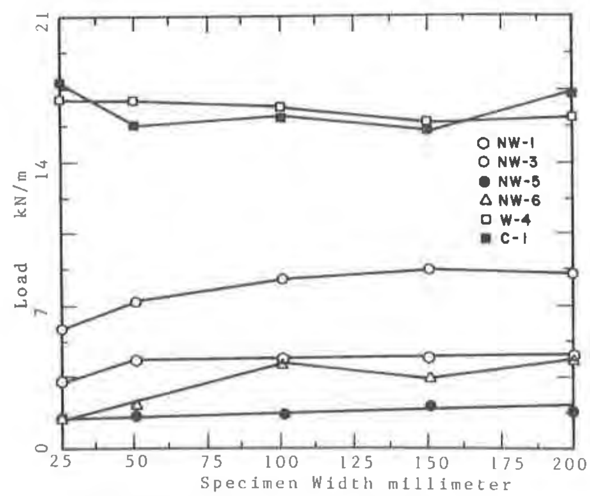

FIGURE 7: Load vs. Specimen Width
Strain = 10%, Gauge length = 200 mm (8 inches)

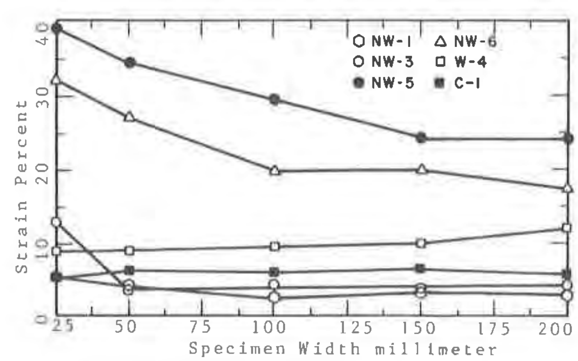

FIGURE 8: Strain vs. Specimen Width
Load = 40% Strength, Gauge length = 200 mm (8 inches)

creased with increases in specimen widths to 100 mm (4 inches). For the wovens and bonded nonwovens, however, the specimen width did not appear to have a significant effect.

The results of Series 1 and 2 tests are combined in Figures 9 and 10. The trends are similar for all geotextiles; however, for woven geotextiles, the loads and strains are most influenced by aspect ratios greater than 4.0, whereas, the nonwoven fabrics are most influenced by aspect ratios less than 2.0.

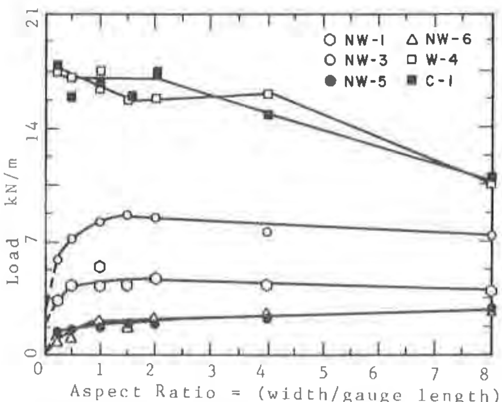

FIGURE 9: Load vs. Aspect Ratio
Strain = 10%

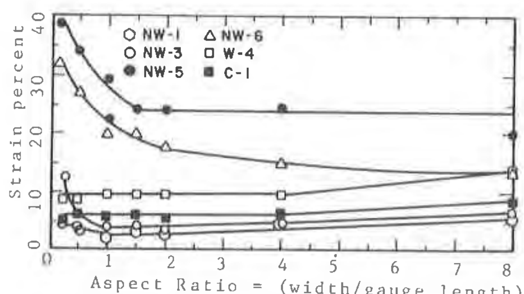

FIGURE 10: Strain vs. Aspect Ratio
Load = 40% strength

Effect of Strain Rate

The results of Series 3 tests in which strain rates from 1-1/4 to 12-1/2 percent per minute were applied to specimens of size 200 mm (8 inches) wide and 100 mm (4 inches) gauge length are presented in Table 3. The ultimate tensile loads do not show any consistent trend with variation in strain rates. Further, plots of the test results showed that strain rates from 1-1/4 to 12-1/2 percent per minute for practical purposes show the same tensile load and strain behavior. These plots are not included due to space limitations.

Effect of Test Method (Plane-Strain Versus Strip Tensile Tests)

For Test Series 4, plane-strain condition was simulated on specimens of size 200 mm (8 inches) wide and 100 mm (4 inches) long by installing light wooden brackets set with pins similar to the ones devised by Sissons (5) to restrain the specimens from lateral contraction (necking)

Table 3. Ultimate Load and Strain at Different Strain Rates

	12-1/2% per minute		5% per minute		1-1/4% per minute	
Fabric	Load kN/m	Strain %	Load kN/m	Strain %	Load kN/m	Strain %
NW-1	8.05	23	7.70	22	7.88	23
NW-3	13.65	49	13.30	53	13.30	62
NW-5	0.032*	97	0.030*	91	0.036*	115
NW-6	8.23	56	8.23	60	8.58	65
W-4	36.93	22	40.25	28	38.85	29
C-1	23.10	14	23.63	15	22.05	17

*Normalized to gm/m^2 1 lbs/inch = 0.175 kN/m

Figures 11 to 13 show plots of the tensile load-strain curves for the geotextiles NW-1, NW-3, and NW-6. It is apparent from these curves that the two test methods (strip tensile and Sissons plane-strain) give almost identical results for the specimen size 200 mm (8 inches) wide and 100 mm (4 inches) long. This was also true for the other geotextiles tested.

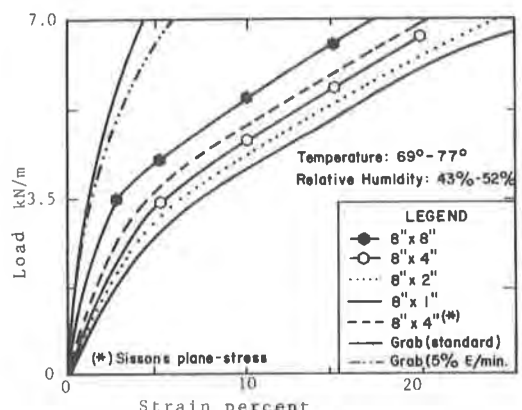

FIGURE 11: Load -Strain Relationships of NW-1 Geotextile for Different Specimen Sizes and Test Methods.

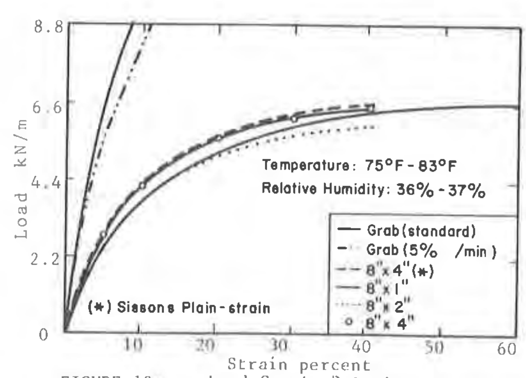

FIGURE 12: Load-Strain Relationship of NW-3 Geotextile for Different Specimen Sizes and Test Methods.

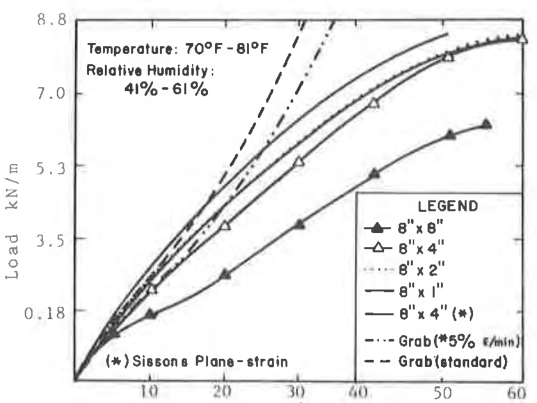

FIGURE 13: Load-Strain Relationships of NW-6 Geotextile for Different Specimen Sizes and Test Methods.

Effect of Sample Variability

A minimum of five specimens were tested for each test variable. All the results presented in previous sections are averages of five specimens. Table 4 presents the maximum coefficients of variation calculated from Series 1 to 5 test results and the minimum number of specimens required to obtain three different levels of precisions on the basis of Student's t-distribution (6).

Table 4. Test Specimens Required for Different Precisions and Probability Levels

Fabric	Max. Coeff. of Variation	± 5% Precision 95% Probability	± 5% Precision 90% Probability	± 10% Precision 95% Probability*
NW-1	.20	64	45	16
NW-3	.12	22	16	6
NW-5	.13	28	20	7
NW-6	.13	27	20	7
W-4	.07	8	5	2
C-1	.09	12	8	3

*Using Student's t-distribution (6), $n = 0.038\ \nu^2$ for ± 10% precision at 95% probability level; where n = number of test repetitions, and ν = coefficient of variation.

DISCUSSION OF RESULTS

Results in Figures 1 and 5 show the effects of specimen width and gauge length on tensile strength are not significant compared to the effects on ultimate strains illustrated in Figures 2 and 6.

Both the loads at 10% strain (Figures 3 and 7) and strains at 40% of strength (Figures 4 and 8) are influenced by the specimen width and gauge length. The amount of variation differs from one geotextile type to another. For woven geotextiles, gauge lengths less than 50 mm (2 inches) show large variations in these loads and strains. Specimen width, however, did not appear to have a significant influence for the wovens. For nonwoven geotextiles, gauge lengths less than 100 mm (4 inches) show large variations in loads and strains. From the plots of the combined effects of both specimen widths and gauge lengths in Figures 9 and 10, aspect ratios less than 4 for wovens and greater than 2 for nonwovens show insignificant variation in both loads and strains. Apparently a satisfactory specimen size for strip tensile tests of both woven and nonwoven geotextiles corresponds to an aspect ratio equal to 2 with a specimen width equal to at least 200 mm (8 inches). Tests performed at this specimen size approximate the plane-strain condition as simulated by the Sissons method (5) as shown in Figures 11, 12, and 13.

The strain rates of 0.5% to 1% per minute normally used in geotechnical laboratory tests are too slow to be economical for geotextiles because of their much higher strains at failure. For example fabric NW-5 would require at least two hours for each specimen at a strain rate of 1% per minute. Series 3 test results show tensile load-strain curves are not significantly different at strain rates of 1-1/4%, 5% and 12-1/2%. On the basis of this observation, a test rate of 10% per minute appears to be reasonable for routine laboratory tests.

The coefficient of variation varied from one geotextile type to another as shown in Table 4 by as much as 30%. Initial laboratory tests should include at least five specimens. Then on the basis of the coefficient variation calculated from these results, the number of specimens required for a desired level of probability can be calculated by using the Student's t-distribution (6). The number of test specimens required as shown in Table 4 to obtain results within ± 10% precision at 95% probability level is not excessively large and this provides reasonable accuracy for routine tensile tests of geotextiles.

CONCLUSIONS AND RECOMMENDATIONS

1. The ultimate strengths of geotextiles measured in strip tensile tests are not significantly affected by specimen size; however, the failure strains of the laboratory specimens are affected significantly by specimen width and gauge length.
2. Load-strain relationships measured for geotextiles may vary considerably with specimen width and gauge length. Woven geotextiles are most influenced by aspect ratios greater than 4.0 and specimen gauge lengths of less than 50 mm (2 inches). Nonwoven geotextiles are most influenced by aspect ratios less than 2.0 and specimen widths less than 100 mm (4 inches).
3. A wide strip tensile test utilizing specimens 200 mm (8 inches) wide and 100 mm (4 inches) gauge length is recommended for routine laboratory testing. At this specimen size, plane-strain loading conditions on geotextiles can be approximated without use of a restraining device to limit lateral contraction of specimens during strip tensile tests.
4. The laboratory measurements of tensile properties are not significantly different at strain rates 1-1/4 to 12-1/2 percent per minute. A strain rate of 10% per minute is recommended for routine strip tensile tests.
5. A precision within ± 10% at 95% probability is recommended for routine laboratory tensile testing by the wide strip tensile test.

ACKNOWLEDGEMENT

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Fatigue Study of Geotextiles

Etude en fatigue des géotextiles

S U M M A R Y

In some applications geotextiles are confronted with the problem of repeated loads.

Till now, calculators didn't dispose of datas concerning failure mecanism of that kind of material in such a way of stressing.

The purposes of this study are the determination of the endurance limite of "fatigue-traction" of geotextiles and the determination for loads level smaller than that limit by the use of rheological models.

R E S U M E

Certaines utilisations des géotextiles voient ceux-ci confrontés aux problèmes des charges répétées.

A ce jour, les auteurs de projets ne disposent pas de renseignements relatifs aux mécanismes de rupture de ces matériaux sous ce type de sollicitation.

L'étude proposée a pour objet la détermination de la limite d'endurance en "fatigue-traction" des géotextiles et la prédétermination des déformations pour des niveaux de charges inférieurs à cette limite. Pour cela un modèle rhéologique a été établi.

1- INTRODUCTION

Cette étude est le résultat d'une campagne menée tant sur le plan expérimental que théorique avec des géotextiles sollicités en "fatigue-traction" sous charges répétées.

L'observation du comportement sans charges statiques et dynamiques nous permet d'établir des modèles mathématiques sur la rhéologie de ces matériaux.

2- ESSAIS

2.1. Méthodologie

L'étude expérimentale consiste en la sollicitation en traction, à une fréquence de 0,6 Hertz (passage d'un camion à 10 km/h), d'éprouvettes type "Bande large" entre des niveaux de charge de :
10 % - palier inférieur
90, 80, 70 % ... palier supérieur de la charge de rupture statique.

L'essai de traction sur éprouvette type "Bande large" a été mis au point à l'Université de Liège en 1979 et depuis adopté par la RILEM [1] . Les éprouvettes sont rectangulaires (L = 0,8 m l = 0,1 m). La vitesse de déformation est de 50 % de la déformation par minute.

Sur un matériau testé l'enchaînement des opérations adopté est le suivant :

- Détermination de la charge de rupture statique.
- Elaboration du modèle rhéologique statique.
- Calcul des niveaux de palier supérieur 90, 80, 70 % de la charge de rupture statique ; calcul du niveau inférieur fixé conventionnellement à 10 %.
- Essai de fatigue. Dans une première phase entre le palier supérieur 90 % et inférieur 10 %, si rupture, nouvel essai entre 80 % et 10 % et ainsi de suite jusqu'à vérification du critère d'endurance (cf. 2.2.)

Lors des essais en fatigue la vitesse de déplacement est réglée automatiquement de manière à suivre un signal sinusoïdal selon la fréquence définie précédemment.

2.2. Critères de résistance à la fatigue

La limite d'endurance correspond au palier supérieur de charge pour lequel 5 éprouvettes résistent à 50000 cycles ; le palier inférieur étant fixé conventionnellement à 10 % de la charge de rupture statique.

2.3. Machine de fatigue

Le dynamomètre employé est de marque TINIUS OLSEN dont la charge maximale est de 500 kN. Il est équipé d'un dispositif qui contrôle soit les déformations, soit les charges. Tous les essais ont été réalisés à charge contrôlée (signal sinusoïdal).

2.4. Matériaux testés

Les matériaux ont été sélectionnés en relation avec leur présence sur le marché des géotextiles, et leurs paramètres de constitution (polymère) et de fabrication (tissé ; nontissé, - aiguilleté - thermosoudé).

2.4.1. Tissé de bandelettes de polypropylène - masse surfacique 0,2 kg/m2 - marque SCOTLAY.

2.4.2. Nontissé aiguilleté obtenu par filature directe ; polyester ; masse surfacique 0,21 kg/m2 - marque BIDIM.

2.4.3. Nontissé aiguilleté obtenu par filature directe ; polypropylène ; masse surfacique 0,2 kg/m2 - marque SODOCA.

2.4.4. Nontissé thermosoudé obtenu par filature directe ; polypropylène ; masse surfacique 0,2 kg/m2 - marque TYPAR.

3- SIMULATION MATHEMATIQUE

Le modèle rhéologique reproduisant les déformations du géotextile en fatigue (nombre et type de cycles) dépend de son comportement sous charge statique.

On peut donc écrire la relation :

"Modèle rhéologique fatigue = Modèle rhéologique statique + Effet différé fatigue".

3.1. Modèles rhéologiques en traction statique

Nous avons observé en cours d'expérimentation deux types de comportement.

3.1.1. Déformations sans effet de fluage[2]

On représente ce type de comportement au moyen d'un modèle rhéologique comportant deux ressorts en série (Figure 1), un des deux ressorts est mis en parallèle avec une crémaillère qui autorise uniquement les allongements.

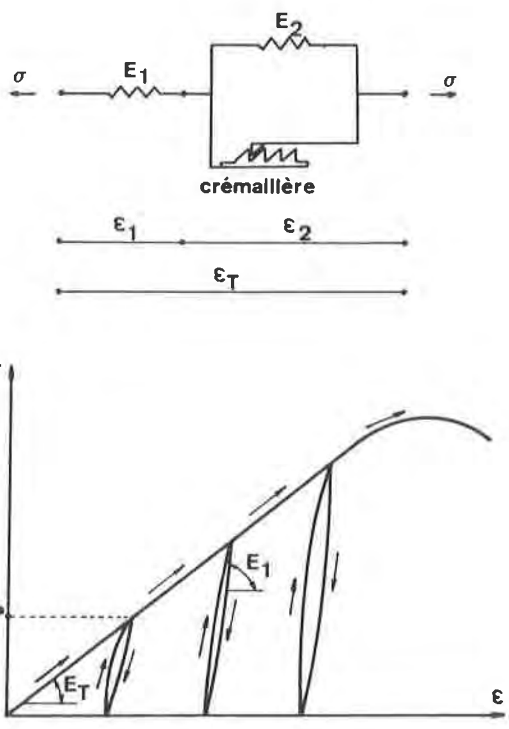

FIGURE 1 : Déformation sans effet de fluage (traction statique)

La mise en évidence de chacun des composants du modèle (Figure 1) est possible au moyen d'un essai type "Bande large" (cf. 2.1.).

Lors de la première mise en charge, les deux ressorts sont sollicités simultanément. Si pour un niveau de charge intermédiaire σ^* on opère une décharge (Figure 1) la crémaillère bloque le ressort E_2 au niveau de déformation maximum atteint sans et seul le ressort E_1 travaille.

Lors de la remise en charge du système et lorsque la contrainte σ^* est atteinte, le ressort E_2 entre de nouveau en fonctionnement. Le module résultant du système (E_T) se décomposant ainsi :

$$\frac{1}{E_T} = \frac{1}{E_1} - \frac{1}{E_2} \qquad (1)$$

Les modules E_T et E_1 sont définis expérimentalement et la relation (1) permet le calcul de E_2

$$E_2 = \frac{E_1 \times E_T}{E_1 - E_T} \qquad (2)$$

- Le ressort E_1 représente l'élasticité rémanante du géotextile en cours de déformation.
- Le système "ressort E_2 crémaillère" représente la destruction interne du matériau.

3.1.2. Déformations avec effet de fluage

Dans ce cas le modèle rhéologique est plus complexe (Figure 2) ; il consiste en la mise en série de plusieurs systèmes (1,2,3).

- Un ressort E_1
- Un ressort E_2 + crémaillère en parallèle
- Un ressort E_3 + dash pot η_3 en parallèle

Soit (Figure 2) $\varepsilon_T = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$ (3)

La déformation a lieu à vitesse constante on obtient :

$$\varepsilon_T = k \times t \qquad (4)$$

k = vitesse de déformation (m/s)
t = temps (seconde)

L'équation rhéologique donnant la valeur de σ en fonction de $\varepsilon = k \times t$ est :

$$\sigma = E_T k t - \frac{E_T^2 \times k \times \eta_3}{(E_T + E_3)^2}\left[\left(\frac{E_T + E_3}{\eta_3} t - 1\right) + e^{-\frac{E_T + E_3}{\eta_3} t}\right] \qquad (5)$$

Lorsque t tend vers zéro, la dérivée de cette fonction (Figure 3) tend vers :

$$\left(\frac{\partial \sigma}{\partial t}\right)_{t \to 0} = E_T k \qquad (7)$$

$$E_T = \frac{E_1 \times E_2}{E_1 + E_2} \qquad (6)$$

La valeur de E_T est donc définie.

Lorsqu'une décharge est amorcée, le ressort E_2 est bloqué par la crémaillère. Dans ce cas E_1, E_3, η_3 travaillent. On peut démontrer que, si σ^* est la valeur de la contrainte au moment de la décharge (Figure3)

$$\left(\frac{\partial \sigma}{\partial t}\right)_{\sigma \to \sigma^*} = E_1 k \text{ (décharge)} \qquad (8)$$

La formule (6) permet la détermination de E_2. Pour la détermination de E_3 l'équation (5) se simplifie (disparition du terme η_3) lorsque le temps prend une valeur importante, d'où E_3. Pour déterminer η_3 on ajuste statistiquement le modèle en choisissant la plage de temps où les effets du fluage sont les plus sensibles.

- Le ressort E_1 représente l'élasticité rémanente du géotextile
- Le système "ressort E_2 + crémaillère représente la déformation irréversible donc la destruction interne du matériau
- Le système "ressort E_3 + dash pot" η_3 rend compte de l'effet de fluage. Les déformations relatives à ce système sont réversibles mais différées.

Après décharge intermédiaire (σ^*) accompagnée d'un certain temps de repos, il ne subsiste alors qu'une déformation rémanente égale à ε_2 contenue dans le système 2 (ressort + crémaillère).

3.2. Modèle rhéologique en "traction fatigue"

Nous nous proposons au travers de ce modèle, d'exprimer la valeur de l'allongement du géotextile en fonction du nombre de cycles subis (fonction du temps).

Sur la base de l'expérience acquise, nous pouvons proposer pour tous les géotextiles le modèle de la figure 2.

- Les systèmes 1 (E_1) et 2 (E_2 + crémaillère) conservent la même signification que pour la traction statique.
- Le système 3 (E_3 + η_3) représente la déformation différée sans charges répétées.

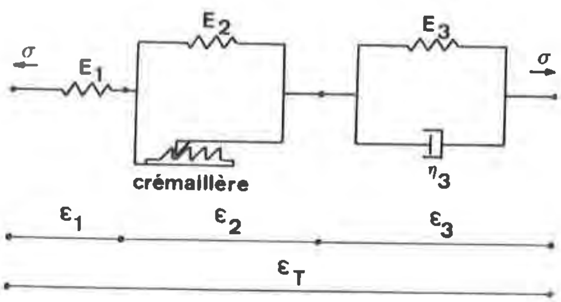

FIGURE 2 : Modèle rhéologique d'un géotextile avec effet de fluage (traction statique)

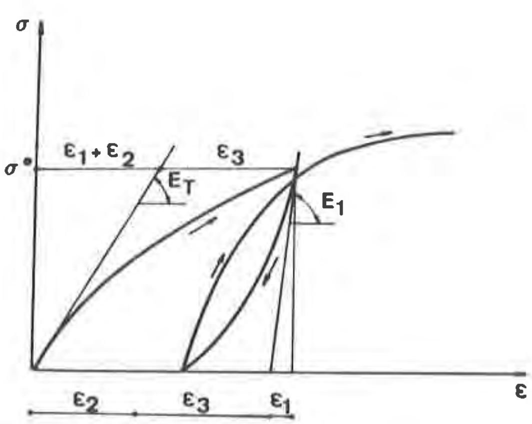

FIGURE 3 : Mise en évidence des constituants d'un modèle visco-plastique de géotextile

On constate par expérience que tous les géotextiles sont régis en "traction fatigue" par une même loi de comportement, ce qui n'est pas le cas en traction statique.

Appliquons aux bornes du modèle rhéologique Figure 2 un sollicitation :

$$\sigma = \sigma_M \times A \sin \omega t \qquad (9)$$

après calcul, on obtient la relation :

$$\varepsilon_{(t)} = \frac{\sigma_M}{E_T} + A\left(\frac{1}{E_T} + \frac{1}{E_1}\right) + \frac{\sigma_M}{E_3}\left(1 - e^{-\frac{E_3}{\eta_3}t}\right) + \frac{A}{E_1}\sin \omega t + \frac{A}{E_3^2 + \omega^2\eta_3^2}\left(E_3 \sin \omega t - \omega\eta_3 \cos \omega t + \omega\eta_3 e^{-\frac{E_3}{\eta_3}t}\right) \qquad (10)$$

où
$\varepsilon_{(t)}$ - Allongement relatif total au temps t(-)
σ_M - Contraite au niveau moyen (kN/m)
A - Amplitude de la contrainte (kN/m)
E_T - Module d'élasticité global à l'origine (kN/m)
E_1 - Modùle du système 1 (kN/m)
E_3 - Module de fluage (kN/m)
η_3 - Coefficient d'amortissement de fluage ($\frac{kN \times sec}{m}$)
ω - Pulsation (rad/sec)
t - Temps (sec)

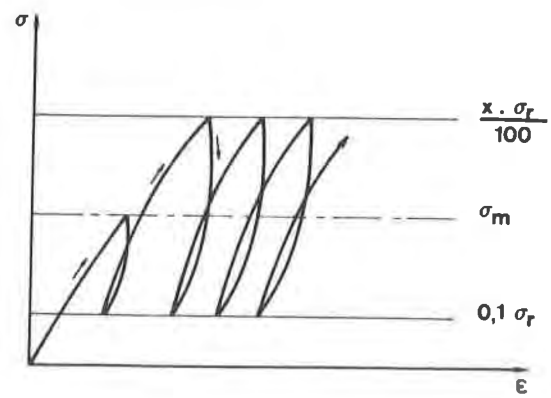

FIGURE 4 : Essais de fatigue entre x % et 10 % de la charge de rupture statique σ_R

La détermination de E_1 et E_2 s'opère comme précédemment. Pour ce qui est de E_3 et de η_3, on observe d'abord que lorsque t prend des valeurs importantes l'effet visqueux devient négligeable. De plus en calculant la déformation au niveau du chargement moyen $\sigma = \sigma_M$, $\sin \omega t = 0$. Enfin le dernier terme de la relation (10) peut être négligé devant les autres.

Il reste :

$$\varepsilon_T = \frac{\sigma_M}{E_T} + A\left(\frac{1}{E_T} + \frac{1}{E_1}\right) + \frac{\sigma_M}{E_3} \qquad (11)$$

Tous les termes sont connus sauf E_3, on peut après détermination de E_3 ajuster η_3 dans la relation (10) de manière à faire coincider les déformations calculées et observées au cours des premiers cycles de chargement.

4- RESULTATS OBTENUS

4.1. Essais statiques

TABLEAU 1 Essais de traction statique

GEOTEXTILES		σ_R (kN/m)	ε_R (-)	E_T (kN/m)	E_1 (kN/m)	E_2 (kN/m)	E_3 (kN/m)	η_3 $\frac{(kN.sec)}{m}$
BIDIM	SP	23,4	0,68	46,2	641	49,8	19,0	3782
	ST	18,6	0,66	31,3	486	33,5	12,3	4566
SODOCA	SP	12,4	1,55	15,6	144	17,6	0,1	2000
	ST	14,0	2,01	17,9	178	19,9	5,0	1425
TYPAR	SP	14,5	0,28	741	1101	2266	42,0	956
	ST	14,0	0,21	483	844	1128	47,5	960
SCOTLAY	SP	20,1	0,38	67,0	570	76,0	-	-
	ST	8,9	0,23	34,4	226	40,6	-	-

SP - Sens Production
ST - Sens Travers

4.2. Essais de fatigue

TABLEAU 2 Essais de traction fatigue

GEOTEXTILES		σ_{and} (kN/m)	$\ell im.and$ (% σ_R)	ε_{50000} (-)	E_3 (kN/m)	η_3 $\frac{(kN \times sec)}{m}$	$\varepsilon_{rupt.}$ (-)
BIDIM	SP	16,4	70	0,54	62,7	65 600	0,70
	ST	13,0	70	0,69	66,6	42 800	0,69
SODOCA	SP	6,2	50	1,65	2,9	50 000	1,65
	ST	9,8	70	1,73	4,9	45 000	2,22
TYPAR	SP	10,2	70	0,66	7,8	117 000	0,55
	ST	8,4	60	0,32	12,7	142 000	0,39
SCOTLAY	SP	10	50	0,29	163,0	485 000	0,33
	ST	7,1	80	0,34	140,0	420 000	0,37

SP - Sens Production
ST - Sens Travers

Rappelons que :

la limite d'endurance est exprimée en pourcentage de la résistance à la rupture (cf. 22) en "traction statique" (Tableau 1) en tenant compte de l'orientation des efforts appliqués (SP-ST)

La contrainte d'endurance σ_{and} correspond à la contrainte de rupture σ_R par la limite d'endurance.

ε_{50000} est la déformation moyenne observée au 50000ème cycle d'un essai de fatigue "réussi" (cf.22)

E_3 et η_3 correspondent au cas de l'essai de fatigue.

E_1 et E_2 du modèle rhéologique en fatigue correspondent aux E_1, E_2 obtenus au moyen d'essai statique (Tableau 1).

$\varepsilon_{rupt.}$ est relative aux allongements correspondant à la rupture en fatigue dans le cas où l'on se trouve au-delà de la limite d'endurance.

4.3. Commentaires sur les résultats obtenus

Nous n'aborderons pas dans ce paragraphe, une quelconque comparaison entre les divers matériaux testés, mais uniquement une analyse de leur comportement en soulignant les paramètres qui nous semblent les plus importants, et qui jouent un rôle primordial dans la connaissance des phénomènes de fatigue. Une interprétation plus détaillée fera l'objet d'une prochaine publication.

4.3.1. Traction statique

La connaissance des valeurs E_1 et E_2 obtenues par des essais en traction statique est nécessaire pour aborder le modèle rhéologique en fatigue.

Nous avons observé que la plupart des matériaux testés répondent à un modèle avec effet de fluage (Figure 2) mis à part pour le SCOTLAY où E_3 et η_3 ne sont pas définis.

A l'exception du TYPAR, on remarque que le module E_T est très différent de E_1 . Cette constatation peut être visualisée sur les figures 5-6-7 et 8.

E_T est évalué sur la base de l'allongement $\varepsilon_1+\varepsilon_2$ alors que E_1 l'est sur la base de ε_1

Pour tous les matériaux, sauf pour TYPAR ε_2 est très différent de ε_1 et par conséquent des modules E_T et E_1 très différents.

Le module E_2 est en relation à l'aptitude du géotextile à encaisser une "destruction interne irrécupérable". Si E_2 augmente la destruction interne diminue. On mettra en rapport les valeurs des déformations rémanentes ε_2 aux figures 5,6,7 et 8 et les valeurs des modules E_1 au Tableau 1.

Il est à remarquer que le point de contact de l'hystérésis avec l'axe des déformations ε (Fig. 5,6, 7,8) ne correspond pas à la restitution intégrale de l'effet visqueux emmaganisé dans le géotextile.

Le cycle déterminé expérimentalement est réalisé à une vitesse de déformation constante de 0,05 m par minute, ce qui est trop rapide pour espérer une restitution instantanée et complète de la déformation en fluage.

La module E_3 associé avec le dash-pot η_3 témoigne d'un effet de fluage plus ou moins marqué. Le dash-pot η_3 est un frein aux déformations du géotextile. Si η_3 est important (Figure 5 et 6), on se rapproche d'un comportement élastique (Figure 1). Le tissé (Figure 8) SCOTLAY peut être considéré comme élastique jusqu'aux valeurs proches de sa rupture.

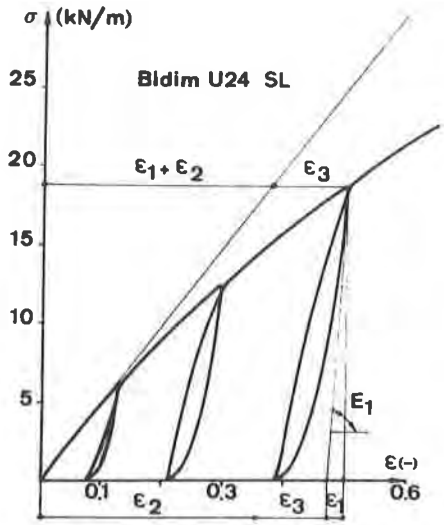

FIGURE 5 : Nontissé aiguilleté polyester

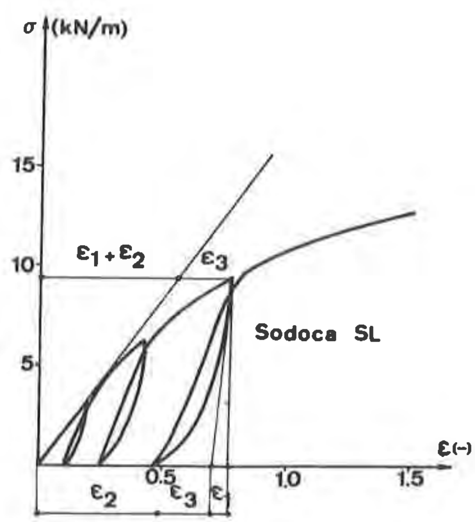

FIGURE 6 : Nontissé aiguilleté polypropylène

4.3.2. Traction fatigue

La limite d'endurance dépend de l'orientation de l'effort appliqué. Pour des paliers supérieurs plus importants que la limite d'endurance, on constate des ruptures d'éprouvettes après un nombre de cycles très variable (rapport de 1 à 50). Pour BIDIM, SODOCA, SCOTLAY, la rupture se produit pour des valeurs d'allongement voisines de celles obtenues au cours d'essai statique. Il semble donc que pour ces matériaux l'allongement peut être considéré comme une caractéristique intrinsèque. Pour TYPAR le comportement en fatigue est différent et l'observation précédente est mise en défaut ; on assiste vraisemblablement à un phénomène de redistribution interne des contraintes.

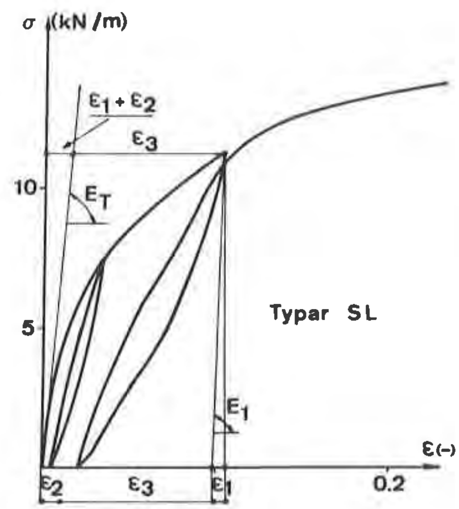

FIGURE 7 : Nontissé thermosoudé polypropylène

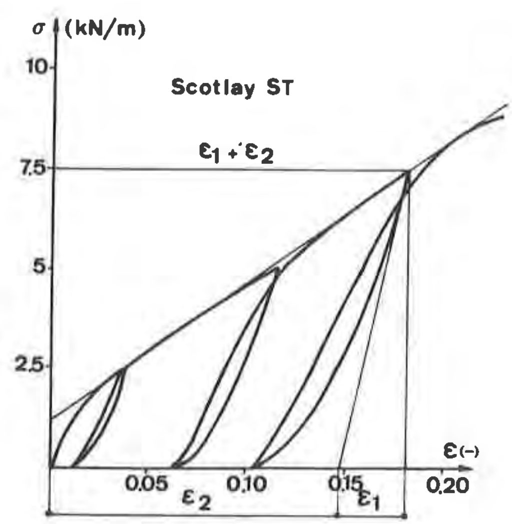

FIGURE 8 : Tissé bandelette polypropylène

Les valeurs de E_1 , E_2 , E_3 et η_3 permettent de prédire en deçà de la limite d'endurance les déformations du géotextile en fonction du nombre de cycles subis.

Les modules E_1 et E_2 ont été définis au cours d'essais en traction statique. Dans le cas de la fatigue, le module E_3 fixe le niveau des déformations en fin d'essai réussi ($\varepsilon_{50\,000}$). On observe que E_3 est d'autant plus grand que $\varepsilon_{50\,000}$ est faible. E_3 dépend également de E_1 et E_2

Le dash pot η_3 marque la progressivité des déformations vers la valeur ε_{50000} . Si η_3 est faible $\varepsilon(t)$ atteint rapidement ε_{50000}

6- CONCLUSIONS

A ce stade des travaux conduits à l'université de Liège, les auteurs peuvent conclure de l'existence d'une limite d'endurance des géotextiles en fatigue traction.

Au moyen des formules mathématiques proposées, une évaluation des déformations des géotextiles sollicités en "traction fatigue" est devenue possible. La rupture par fatigue survient par le dépassement d'un allongement limite caractéristique du matériau. On peut donc évaluer un coefficient de sécurité.

Les modèles rhéologiques proposés montrent que les géotextiles sous fatigue sont régis par une même loi de comportement, ce qui n'est pas le cas en traction statique.

Les auteurs proposent dans cette étude une première approche du problème, mais de nombreuses questions restent posées :
fréquence, niveau inférieur du palier de charge, corrélation avec la texture du géotextile ...

L'exploitation pratique des résultats de cette étude permet d'entrevoir une approche sérieuse du calcul de dimensionnement des géotextiles dans les voies de circulation. Pour cela des essais de simulation s'imposent en laboratoire. Une première tentative a déjà été réalisée dans ce sens.[3]

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Light Resistance of Textile Fibers
Résistance à la lumière des fibres textiles

The light fastness of geotextiles is only of minor importance. Nevertheless, they could be exposed to sunlight for some days or weeks and, as a consequence, loose a considerable part of their resistance. A general prediction of their performance due to light exposure is, however, not possible as too many factors are simultaneously involved (sun, air pollution, temperature, rain etc.). By means of a mathematical model (Weibull function) an attempt has been made to obtain from laboratory tests certain indications as to the practical behaviour if some simplifying assumptions are made. In this way, eight different exposure methods were compared experimentally with the exterior weathering using eight textile fibers. It is shown that the laboratory methods, when applied during a determined time, correspond to periods of exterior weatherings ranging from 1 to more than 100 days, depending on the type of fibre. No test method shows any particular advantage.

La résistance à la lumière des géotextiles ne joue qu'un rôle secondaire. Cependant il est possible que les géotextiles soient exposés au soleil plusieurs jours ou semaines et que leur résistance diminue sensiblement durant cette période. Une prévision générale sur le comportement mécanique après exposition n'est pas possible, car il y a trop de facteurs qui agissent simultanément (soleil, pollution de l'air, température, pluie, etc.). Dans cette étude nous avons essayé sur la base d'un modèle mathématique (fonction de Weibull) d'obtenir certaines indications sur le comportement pratique à partir d'essais de laboratoire, tout en faisant certaines hypothèses simplificatrices. Nous avons comparé 8 méthodes d'exposition avec l'exposition extérieure et ceci pour 8 fibres textiles. Il en ressort que les expositions artificielles appliquées durant un temps donné correspondent à une exposition extérieure de 1 à 100 jours selon le type de fibres. Aucune des méthodes étudiées présente des avantages particuliers.

1 INTRODUCTION

La résistance des matériaux textiles à l'influence du rayonnement électromagnétique joue un rôle dans presque tous les cas. Des dégâts se manifestèrent d'abord aux rideaux, plus tard aux stores, ceintures, tapis, revêtements d'automobiles, protections contre avalanches, etc. Ces dégâts sont surtout dus à la portion du spectre de longueurs d'ondes courtes, spécialement au rayonnement UV. C'est pour cette raison que les producteurs de fibres ont essayé de leur côté, en partie avec succès, de stabiliser les produits, alors que les instituts de contrôles ou de recherches se sont efforcés de trouver une méthode de courte durée qui permet de définir avec une précision suffisante le comportement des fibres à la lumière du soleil.

Dans le cas des géotextiles la résistance à la lumière peut jouer un rôle puisque ceux-ci peuvent être exposés au soleil pendant plusieurs jours, voir semaines sur les chantiers. L'utilisation de ces produits en technique agraire et en "paysagisme" exige une résistance accrue.

Un test rapide devant simuler le comportement pratique est toujours problématique: si en pratique le processus se déroule lentement sous l'influence des facteurs relativement faibles, le test rapide doit produire le même effet en un temps comprimé, ceci sans possibilité de récupération pour le matériau et sous l'influence accrue d'un facteur précis. En pratique, on a en plus toujours plusieurs facteurs intervenant simultanément. Ceux-ci peuvent avoir et ont en général une influence sur le facteur étudié. Le test de laboratoire ne permet toutefois d'étudier qu'un seul facteur, rarement deux ou trois simultanément.

Lors de l'exposition au climat extérieur les paramètres suivants jouent un rôle important dans le cas des textiles:

- température : Du fait du rayonnement, le textile peut avoir une température supérieure à celle de l'air ambiant. Cette température n'est peut-être pas assez élevée pour induire une dégradation thermique mais favorise probablement des dégradations hydrolytiques, oxidatives et photochimiques secondaires.
- humidité/précipitations: Ces facteurs peuvent avant tout provoquer des dégâts physiques (gel) et un gonflement. L'hydrolyse et la catalyse sont également favorisées par des précipitations polluées.
- vent : Pour les géotextiles il s'agit d'une sollicitation mécanique, renforcée en particulier par l'impact de fines particules.

- polluants de l'air: Ceux-ci peuvent attaquer les fibres textiles de façon extrême. Il faut avant tout évoquer:

  SO_2, SO_3, H_2S

  $(NO)_x$

  O_3

  composés organiques (les alkyls non saturés, les composés aromatiques catalysent la photo-oxydation).

- lumière : C'est la partie de longueur d'ondes courtes de la lumière solaire (UV) qui détruit les liaisons chimiques des fibres textiles. Ici il faut noter que la part des UV dépend fortement du lieu géographique et de l'altitude. De plus les variations saisonnières et journalières sont très importantes et changent d'une année à l'autre. Ces variations sont d'autant plus fortes que la longueur d'ondes de la lumière est plus petite. La littérature (1) décrit par exemple le cas de deux textiles exposés à la lumière en Arizona et en Floride, pour lesquels on a obtenu des résultats opposés.

On peut dire, presque avec certitude, que tous ces facteurs s'influencent réciproquement, c'est-à-dire que leur action cumulée sur le géotextile sera renforcée. Il paraît donc clair que l'on ne peut pas comparer des expositions au climat extérieur réalisées à des endroits et à des moments différents. Si cela n'est pas possible, il est peu probable que l'on puisse développer une exposition artificielle à la lumière ou au climat qui puisse décrire de manière générale le comportement d'un géotextile au soleil. On peut au mieux imaginer une relation entre un test de laboratoire et une exposition extérieure donnée à un moment donné.

2 CONSIDERATION THEORIQUES

Plusieurs modèles ont été proposés pour décrire les changements de propriété dûs à l'influence du climat. En général on utilise une courbe exponentielle (2), dont la forme la plus complète est la suivante:

$$y = b_1 \exp\left[-\left(\frac{t+b_2}{b_3}\right)^{b_4}\right] + b_5 \quad \text{(Fonction Weibull)} \qquad (1)$$

Les cinq paramètres décrivent la courbe de dégradation et on peut leur attribuer les significations physiques suivantes: b_1 se rapporte au maximum de la propriété au temps $t = -b_2$ ($b_1 + b_5$ = maximum); b_2 est le pré-ou post-vieillissement. Si b_2 est positif on a déjà en début d'observation un certain vieillissement ou une dégradation correspondant au temps b_2; b_3 représente le temps de décroissance (temps calculé à partir de $t = -b_2$) nécessaire pour que la propriété tombe à 37 % de sa valeur initiale. b_4 décrit la forme de la courbe: $b_4 \leq 1$ décroissance initiale rapide, $b_4 > 1$ décroissance initiale retardée (voir fig. 1). b_5 est la valeur asymptotique de la propriété (valeur pour $t = \infty$). En général $b_5 = 0$, c'est-à-dire que p.ex. la force de rupture sera nulle pour un temps d'exposition suffisamment long.

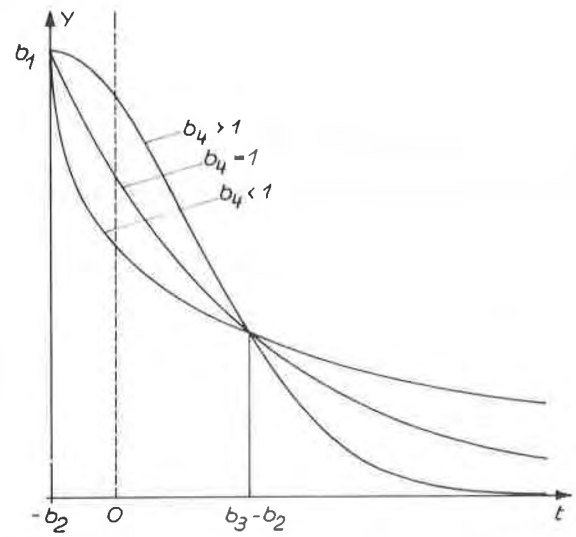

Fig. 1: Fonctions Weibull

Dans la littérature (2) on suppose que b_2 est souvent = 0, ce que nous n'avons toutefois pas pu confirmer.

Si donc notre modèle

$$y = b_1 \exp\left[-\left(\frac{t+b_2}{b_3}\right)^{b_4}\right] \qquad (2)$$

était correct, ceci aurait des conséquences sur la signification d'une exposition d'un genre quelconque pour l'exposition extérieure.

Si en effet

$$y = b_1 \exp\left[-\left(\frac{t+b_2}{b_3}\right)^{b_4}\right] \qquad (3)$$

représente une exposition artificielle et

$$z = a_1 \exp\left[-\left(\frac{T+a_2}{a_3}\right)^{a_4}\right] \qquad (4)$$

l'exposition extérieure, une relation linéaire n'est pas possible. Cela signifie qu'en général on ne peut pas passer de l'exposition artificielle à l'exposition extérieure par un simple facteur de proportionalité. En effet

$$\frac{y}{z} = A \exp\left[-\left(\frac{t+b_2}{b_3}\right)^{b_4} + \left(\frac{T+a_2}{a_3}\right)^{a_3}\right] \qquad (5)$$

En fait, il serait souhaitable que $\frac{t}{T}$ = constant pour $\frac{y}{z} = 1$. On peut admettre A égal à 1, puisque dans le cas présent il s'agit du même matériau et que des valeurs initiales identiques sont ainsi garanties.

Par conséquent

$$\left(\frac{t+b_2}{b_3}\right)^{b_4} = \left(\frac{T+a_4}{a_3}\right)^{a_4} \qquad (6)$$

donc aucunement une relation linéaire. Sauf si

$b_4 = a_4 = 1$, nous avons

$$\frac{t+b_2}{b_3} = \frac{T+a_2}{a_3} \quad (7)$$

et, comme pour t = T = 0 il s'agit du même matériau et donc des mêmes conditions,

$$\frac{b_2}{b_3} = \frac{a_2}{a_3} \quad \text{et ainsi} \quad (8)$$

$$T = t.\ \frac{a_3}{b_3} \quad (9)$$

donc une relation linéaire.

Les simplifications que nous avons faites ne sont pas permises, comme nous allons le montrer à partir de courbes mesurées.

3 METHODES DE CONTROLE

Malgré ce que nous venons de dire, on essaie toujours d'adapter des tests de laboratoire à l'exposition extérieure. La littérature décrit les possibilités les plus diverses. Cela nous a amené à réaliser un essai comparatif à grande échelle.

Parallèlement à un test extérieur réalisé à St-Gall (alt. 760 m) selon ISO 105 - BO3 nous avons utilisé les appareils suivants:

Exposition au Xenontest 450 selon ISO 105 - BO2: les échantillons sont placés circulairement à une distance d'env. 25 cm de la lampe à arc au xénon et tournent autour de celle-ci. Des filtres UV et IR sont placés entre la lampe et les échantillons. L'exposition se fait à une température de l'air de 35° et une humidité relative proche de 65 %.

Exposition à la lampe UV: les échantillons sont placés circulairement à une distance d'environ 28 cm d'une lampe au mercure à haute pression. Les échantillons sont immobiles et l'enceinte n'est pas régularisée en humidité et en température.

Exposition au Fadeomètre - AATCC Test Method 16A - 1977: les échantillons tournent autour d'une lampe à arc au charbon à une distance d'environ 25 cm. (Les charbons doivent être changés environ toutes les 24 heures).

Exposition climatique au Xenontest selon ISO 105 - BO 4: durant l'exposition les échantillons sont arrosés pendant une minute (eau déionisée) et séchés pendant 29 minutes (sans apports supplémentaires de chaleur et d'humidité).

Xenontest sans filtre KG 1: Xenontest normal comme décrit ci-dessus en supprimant 4 des 7 filtres KG 1 (IR), ainsi que Xenontest en supprimant 2, respectivement 3 filtres KG 1 et en remplaçant le cylindre extérieur en verre (filtre UV) par un cylindre en verre de quartz.

Xenontest humide: comme test normal, mais avec une humidité relative de 95 % dans l'enceinte d'essai.

Les temps d'exposition sont donnés dans le tableau suivant:

Tableau 1: Temps d'exposition

Méthode	Unités	Temps d'exposition
exposition extérieure	jours	0 21 42 90 180 360
	heures d'ensoleillement	0 60 105 180 400 1300
lampe UV	h	0 2 4 8 16 32
Fadeomètre	h	0 20 40 80 160 320
Xenontest	h	0 60 125 250 500
Xenontest à 90 % d'humidité relative	h	0 50 100 150
Xenontest, exposition climatique	h	0 60 125 250 500
Xenontest avec 3 filtres KG 1	h	0 60 125 250 500 1000
Xenontest avec 4 filtres KG 1 et verre de quartz	h	0 125 250
Xenontest avec 5 filtres KG 1 et verre de quartz	h	0 60 125 250

4 MESURES ET MATERIAUX

Pour contrôler la dégradation nous avons mesuré la force de rupture, l'allongement à la rupture, l'énergie de rupture et la viscosité après plusieurs temps d'exposition. La viscosité est une mesure de la longueur moyenne des molécules et donc de la "qualité" des fibres.

Dans un premier temps nous avons choisi 8 matériaux sous forme de filés. Dans une étude future nous exposerons environ une vingtaine d'autres matériaux de la même manière.

Tableau 2: Materiaux utilisés

1. PA 6 haute résistance, brillant, tex 94 f 136
2. PA 6.6 stabilisé à la lumière, tex 94 f 140
3. PP/PE 80 %/20 %, tex 140x1
4. PP/PE id. stabilisé UV
5. PES, brillant, tex 110 f 192
6. PA aromatique, tex 25x2
7. id. (autre type)
8. coton écru, tex 30x1

5 RESULTATS

Une première analyse des résultats permet de faire les constatations suivantes:

1. Les diminutions de la force de rupture et de l'allongement à la rupture sont à peu près parallèles. C'est pourquoi nous n'avons dans un premier temps utilisé que les valeurs de la force de rupture.
2. La viscosité et la force de rupture ne sont pas du tout corrélées. Cet état de chose étant connu peut, en outre, être ramené au fait que pour certains types de fibres ce sont avant tout les domaines amorphes qui sont dégradés par la lumière. Il n'y a ainsi qu'une petite partie des chaînes moléculaires qui sont détruites. Ce sont pourtant ces domaines amorphes qui déterminent la force de rupture. Dans la fig. 2 nous avons représenté pour les 8 matériaux les forces de rupture relatives en fonction des viscosités relatives pour différents genres et temps

d'exposition. Il semble qu'il y ait une certaine corrélation que pour le PA 6 et PA 6.6 (fig. 3).

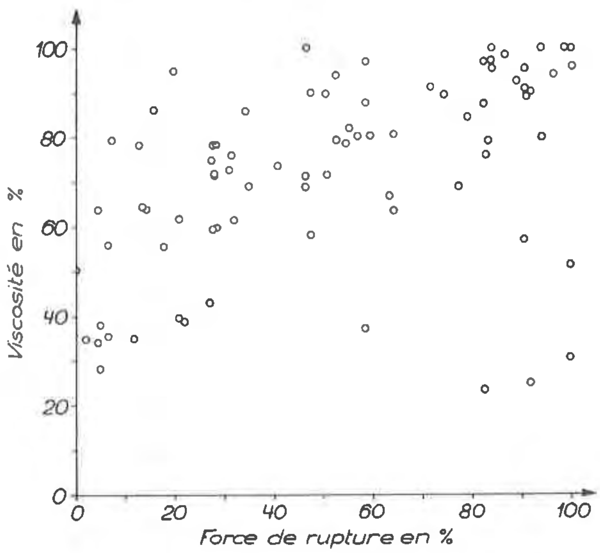

Fig. 2: Forces de rupture vs. viscosité de huit fils d'après divers genres d'illumination et temps d'exposition

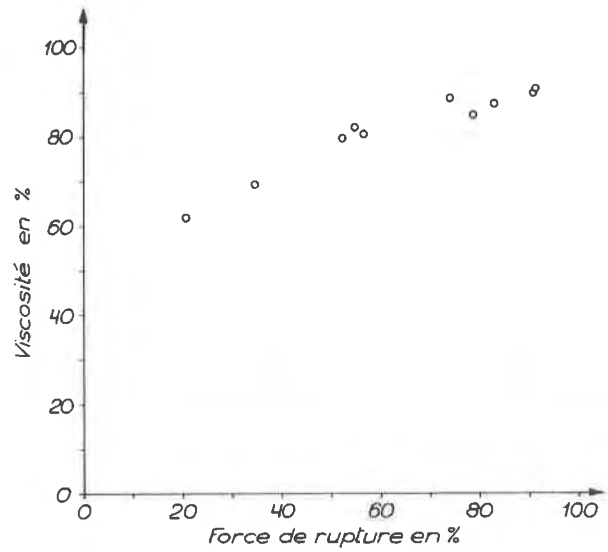

Fig. 3: Forces de rupture vs. viscosité de PA 6.6 d'après divers genres d'illumination et temps d'exposition

Tableau 3: Coefficients de Weibull calculés

Méthode		Matériau							
		1	2	3	4	5	6	7	8
extérieur	b_3	1470	1440	177	310	1070	106	88	pas de
	b_4	0,88	0,70	1,37	3,22	0,86	0,67	0,70	déchéance
UV	b_3	22	16,1	5,7	12	134	208	pas de	7,3
	b_4	0,89	0,79	0,68	0,65	0,45	1,25	déchéance	0,70
Fadeomètre	b_3	342	560	59	370	9400	27	24,6	1380
	b_4	0,93	0,88	1,1	1,70	0,52	0,44	0,43	0,42
Xenon	b_3	pas de	28000	765	753	11000	60	150	980
	b_4	déchéance	0,30	1,3	3,2	0,40	1,6	1,0	2,1
Xenon humide	b_3	459	604	80	284	387	95	86	660
	b_4	1,2	0,80	1,5	3,2	0,60	0,52	0,69	0,45
Xenon climat	b_3	2240	820	182	662	797	88	77	pas de
	b_4	0,90	1,1	1,34	1,02	0,88	0,42	0,87	déchéance
Xenon 3 KG 1	b_3	208	1820	44	254	1380	77	89	pas de
	b_4	0,92	0,56	0,67	0,89	0,44	0,46	0,34	déchéance
Xenon 4 KG 1*	b_3	177	194	36	70	330	138	192	153
	b_4	1,0	0,55	0,60	0,61	0,41	0,60	0,41	0,57
Xenon 5 KG 1*	b_3	149	212	50,4	82	694	162	174	122
	b_4	1,2	0,88	0,79	0,66	0,34	0,75	0,6	0,85

* cylindre extérieur en quartz au lieu de verre

Il s'est avéré que la formule (1) ne peut pas être appliquée sans autre, car d'une part il y a une forte dispersion des mesures, et d'autre part on ne dispose que de 6 valeurs pour le calcul. Pour cette raison nous avons dû faire deux hypothèses qui ne sont de toute évidence pas très correctes: nous avons posé $b_2 = 0$ et $b_1 = 100$ %, car cette dernière valeur a pu être vérifiée sur un échantillonnage important. Les deux paramètres restants b_3 et b_4 ont été calculés selon la méthode dite "d'approximation de paramètres non-linéaires par la méthode des plus petits carrés" d'après D. Marquardt (3, 4). Ils sont donnés dans le tableau 3.

A partir de nos raisonnements précédents et des hypothèses que nous venons de faire nous pouvons écrire une formule, qui permet de prédire le temps d'exposition extérieure à partir des tests de laboratoire:

$$T = a_3 \cdot \left(\frac{t}{b_3}\right)^{b_4/a_4} \tag{10}$$

Ce n'est pas une relation linéaire. Cette formule donne les rapports de temps suivants entre l'exposition extérieure et les tests de laboratoire:

Tableau 4: Comparaison entre exposition en laboratoire et extérieure

Méthode de laboratoire	temps en h	correspond à une exposition extérieure en jours selon le matériau
UV	2	1 - 140
Fadeomètre	10	9 - 55
Xenon	20	1 - 62
Xenon humide	20	17 - 134
Xenon climat	20	4 - 100
Xenon 3 KG 1	10	26 - 127
Xenon 4 KG 1	10	10 - 214
Xenon 5 KG 1	10	5 - 200

Dans la fig. 4 nous avons reporté les courbes calculées pour l'exposition extérieure ainsi que les points de mesure. Les fig. 5 à 9 montrent les résultats et les courbes de certains types d'exposition choisis.

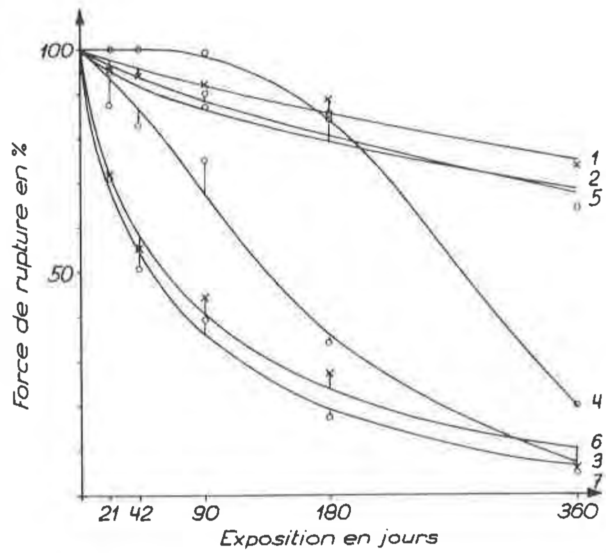

Fig. 4: Forces de rupture relatives (Exposition extérieure; les numéros des courbes correspondent aux fils cités dans le tableau 2)

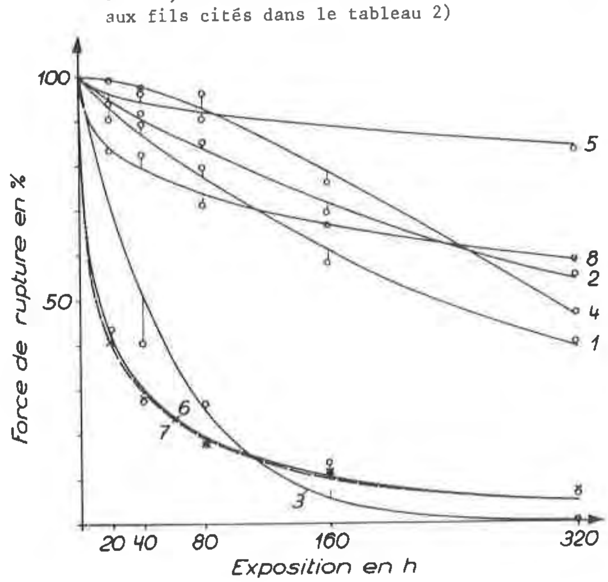

Fig. 5: Forces de rupture relatives (Exposition au Fadeomètre)

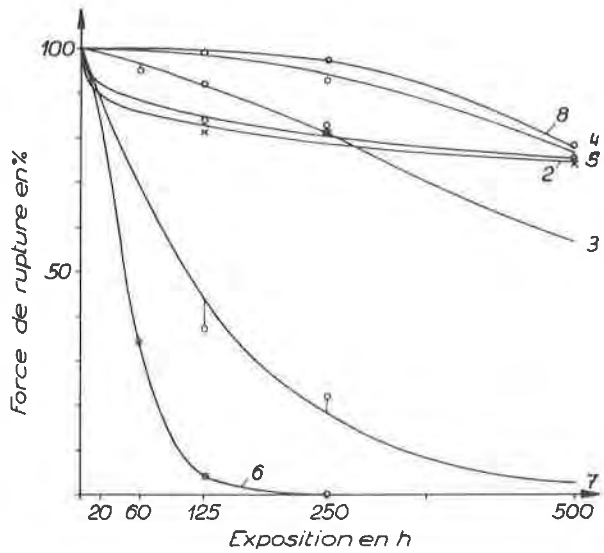

Fig. 6: Forces de rupture relatives (Exposition au Xenontest)

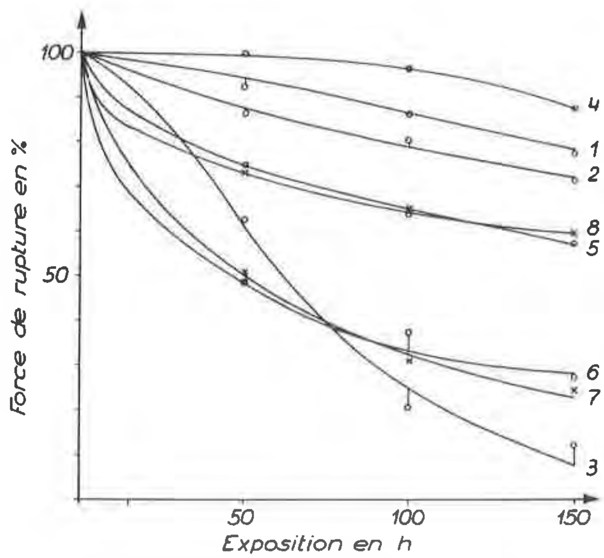

Fig. 7: Forces de rupture relatives (Exposition humide au Xenontest)

Les résultats montrent qu'aucune des méthodes utilisées permet de faire une prévision pour tous les matériaux. A quelques exceptions près (UV, Xenon) les tests permettent toutefois de constater qualitativement la sensibilité au rayonnement de certains matériaux. Pour les géotextiles qui ne sont que peu exposés à la lumière une telle qualification est amplement suffisante.

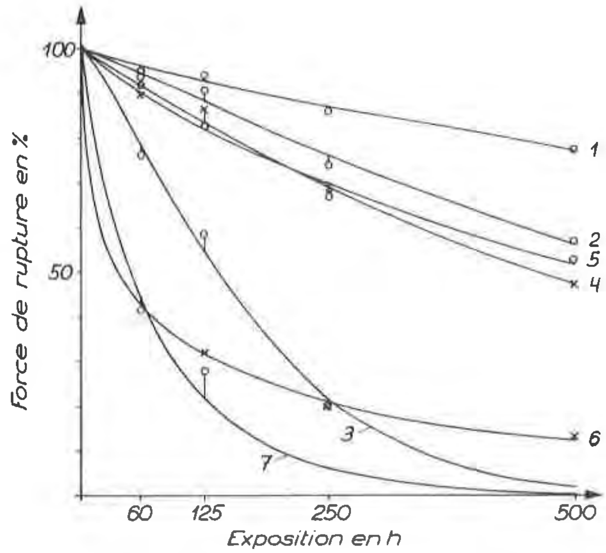

Fig. 8: Forces de rupture relatives (Exposition climatique au Xenontest)

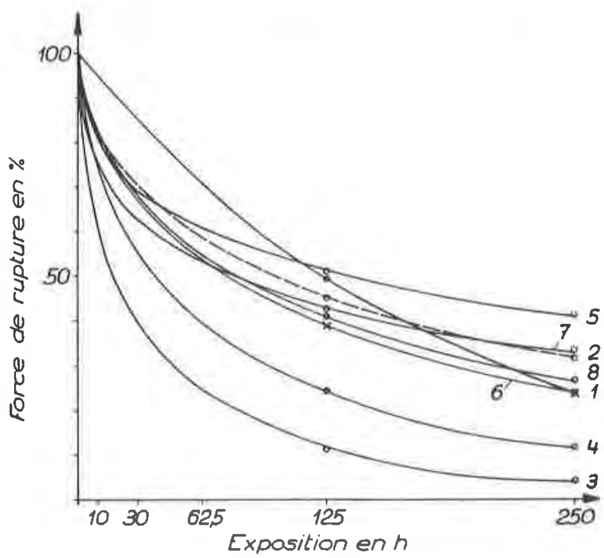

Fig. 9: Forces de rupture relatives (Exposition au Xenontest avec seulement 4 filtres KGl)

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Comparison Between Different Tensile Tests and the Plunger Puncture Test (CBR Test)
Comparaison entre différents essais de traction et l'essai de poinçonnement CBR

As an integral part in the preparation of guidelines for the use of geotextiles in Germany, comparative tests with different test methods are carried out on non-woven fabrics. The objective was to select a suitable test method, and led to a programme which systematically evaluated the influence of different sample widths on the result of the strip tensile test. The tensile tests were carried out on two products from each of 8 producers from Germany and abroad. Both mechanically bonded and heat bonded non-wovens were included. The results of the series of tensile tests were statistically evaluated; standard deviation and coefficient of variation of breaking load and extension at break were recorded. The modulusat different load steps was calculated and typical differences between different types of fabric registered. The results were compared with fabric uniformity tests. Correlation with the results of the CBR test (plunger puncture test) commonly used in Germany make comparisons between the various tests possible and allow the selection of a test suitable for the highway engineer.

Dans le cadre le l'établissement d'une notice pour l'emploi de géotextiles on a étudié en République Fédérale d'Allemagne des non-tissés par des essais comparatifs à l'aide de différentes méthodes d'essai. Le but de l'étude a été de choisir une méthode d'essai convenable. Cet objectif a exigéun programme d'essais devant permettre d'explorer systématiquement l'influence de différentes largeurs des bandes de 100 - 600mm sur les resultats de l'essai de traction sur bande. Les essais de traction ont été effectués avec deux produits à chaque fois de huit fabricants du pays et de l'étranger. On a étudié les géotextiles aiguilletés et thermopondés. On a réalisé une exploitation statistique des résultats recus de la serie d'essais de traction. Ont été déterminés, l'écart type et le coefficient de variation de la resistance à la déchirure ainsi que de l'allongement à la rupture. L'étude comprend la définition de le module pour differents niveaux de sollicitation et la présentation de différences typiques constatées entre les différents procédés de fabrication. Les résultats ont également été comparés avec des études de l'homogénéité. Des correlations avec les résultats de l'essai de poinçonnement (CBR) qui est usuel en RFA rendent les différents essais comparables et mènent au choix d'une méthode d'essai practicable en construction routière.

INTRODUCTION

Guidelines for the use of geotextiles in road construction are being prepared in Germany by a working party of the German Transport and Road Research Institute. During the course of the work, the suitability of standard textile tests for use in highway engineering was called in question. The Norwegians (1) had already answered this question in 1977 with the development of the CBR test, a test tailored to the needs of the highway engineer. Since 1980 this test has been increasingly been used in Germany for control purposes (2). The work described in this paper was intended to compare the results of different tensile tests as regards breaking load, extension at break and load/extension characteristics.

1 TEST PROGRAMME

1.1 SELECTION OF THE GEOTEXTILES

Only non-wovens were included in the test program on two grounds:

- the use of geotextiles for separation and filtration in highway engineering in Germany is confined almost entirely to non-wovens. Wovens and composite materials are used only in special cases e.g. reinforcement or structure drainage.

- The CBR test cannot be used to compare different types of geotextile (wovens, non-wovens, composites).

Non-wovens currently available can be divided into 2 groups of polymers and 2 of production methods. The principal raw materials are polypropylene (PP) and polyester (PES), production methods heat bonded and mechanical bonded.

First tests with wide samples (3) carried out on a mechanically bonded PES non-woven. In the programme described here, samples from a total of 8 producers were tested, covering all possible raw material/ production method combinations.

Table 1.

Producer	Product Name	Fibre Raw Mtrl. and Type	Type of Bonding
Hoechst	Trevira Spunbond	PES continuous	mechanical
ICI Fibres	Terram	PP+PE continuous	heat bonded
Chemie Linz	Polyfelt	PP continuous	mechanical
Naue Fasert.	Terrafix	PES staple	mechanical
Sodoca	Sodoca	PP continuous	mechanical
Fibertex	Fibertex	PP staple	heat + mech.
Lutravil	Lutradur	PES continuous	heat bonded
Du Pont	Typar	PP continuous	heat bonded

1.2 TYPES OF TEST

1.2.1 CBR TEST (PLUNGER PUNCTURE TEST)

The CBR test for geotextiles was first described by Alfheim and Sörlie (1), and Wilmers (2) compared the Grab Test with the CBR Test, now standardized in Germany as DIN 54307 E. The non-wovens reported on here were CBR-tested at the Building Materials Testing Station in Wetzlar in accordance with the DIN norm (ring diameter 150 mm, plunger diameter 50 mm, speed of penetration of plunger 60 ± 10 mm/min).

1.2.2 TENSILE TESTS

The only standardized German tensile test for non-wovens which records both breaking load and extension is the strip tensile test DIN 53857 Part 2. Gauge length is 200 mm, sample width 50 or 100 mm and crosshead speed 50-200 mm/min. The considerable necking down of non-wovens in this test makes interpretation of their load/extension behaviour a doubtful matter (see (3).

Sissons (4) attempted to exclude the influence of necking by the use of spreader bars (sample size 200 x 200 mm, crosshead speed 20 mm/min). This test is considered rather too complicated in Germany, and the possible influence of the pins on the results has not been fully clarified.

Rigo and Perfetti (3) presented another means of reducing the influence of necking down. In a series of experiments they tested samples of an mechanically bonded polyester non-woven with widths of up to 800 mm, gauge length 100 mm and a crosshead speed of 50 mm/min.

Based on the above information, the following test programme was carried out in the Federal Highway Research Institute (BASt) in Cologne:

- Strip tensile tests in accordance with DIN 53857 Part 2
- tensile tests with 100mm gauge length, sample widths of 100, 200, 300, 400, 500 and 600 mm, and a crosshead speed of 50 mm/min.

1.3 SCOPE OF TESTING

For the CBR tests, 10 samples per product were tested (as laid down in DIN 54307 E), this representing a total of 1600 individual tests. The following were measured:-

- sample weight
- push-through force
- extension at 100, 500, 1000 N. push-through, and push-through minus 300 N
- additionally, after exceeding push-through force, extension at push-through minus 300 and 500 N.

At each configuration of the tensile tests, 5 samples per product were tested in machine and cross directions, this respreenting a total of 1280 individual tests. The following were measured:

- sample weight
- max. load
- extension at max. load
- neck down at break

The load/extension curve was plotted mechanically.

2 RESULTS

2.1 CBR TESTS

The results of the CBR tests which had been carried out by the beginning of March 1982 are presented in the following table.

Table 2.

Producer	Type	DD [N]	ε [%] 100N	ε [%] DD	Weight [g/m²]
Hoechst	Tr. Sp. 200	2020	12	69	201
	Tr. Sp. 500	5152	3	56	509
ICI	Terram 1000	1611	4	35	136
	Terram 4000	4173	2	36	342
Chemie Linz	Polyfelt TS 500	1476	7	48	175
	Polyfelt TS 700	2352	6	48	285
Naue	ST 309	1793	12	53	362
	ST 509	2824	8	55	571
Sodoca	AST 250	2090	3	75	281
	AST 420	3088	4	88	428

Dia.1 shows the typical shape of the load/deformation curves of a mechanically bonded and a heat bonded continuous filament non-woven in the CBR test.

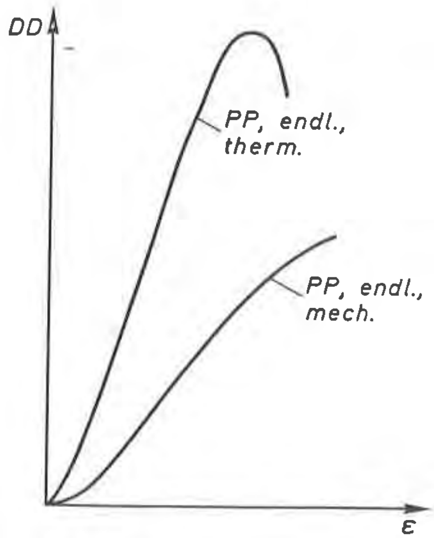

Fig. 1 Load/deformation curves

2.2 STRIP TENSILE TESTS

The results of the different tensile tests are shown in tables 3 - 5.

Table 3. Breaking load (kN/m)

Length [mm]			200		100				
Width [mm]			50	100	100	200	300	400	500
Product	Type	Direction of test							
Trevira Spunbond	200	mach.	8,2	10,6	9,2	11,0	11,7	12,4	11,5
		cross	[illegible]	12,0	12,4	12,8	[illegible]	[illegible]	11,2
	500	mach.	21,5	23,2	26,3	26,2	25,3	26,4	23,6
		cross	24,2	26,2	[illegible]	24,8	23,6	32,1	24,7
Terram	1000	mach	6,2	8,4	7,9	9,4	9,7	9,5	9,9
		cross	5,6	8,0	6,9	8,4	10,0	10,7	9,9
	4000	mach.	21,9	22,8	22,1	25,4	24,2	[illegible]	[illegible]
		cross	16,0	21,0	21,8	23,5	25,0	25,3	25,7
Polyfelt	TS 500	mach.	8,1	8,5	9,9	10,6	9,2	10,7	10,7
		cross	7,8	9,3	8,8	9,4	9,7	9,8	9,6
	TS 700	mach.	13,0	16,2	14,2	15,4	15,2	15,8	16,0
		cross	13,0	16,8	17,4	16,2	16,3	12,9	17,8
Terrafix	ST 309	mach	3,8	4,6	3,7	4,5	5,1	4,5	5,1
		cross	9,5	8,6	10,0	11,9	10,7	11,2	11,2
	ST 509	mach.	7,9	9,2	9,6	9,0	9,2	10,3	10,4
		cross	13,4	14,2	15,6	13,8	16,6	[illegible]	17,1
Sodoca	AST 250	mach.	10,4	11,9	11,8	[illegible]	12,4	[illegible]	13,9
		cross	15,4	17,0	[illegible]	[illegible]	17,0	15,9	15,4
	AST 420	mach.	19,0	20,6	21,6	13,3	20,9	21,5	20,9
		cross	21,6	22,8	26,2	25,6	23,1	26,4	25,1

Table 4. Extension at break (%)

Length [mm]			200		100				
Width [mm]			50	100	100	200	300	400	500
Product	Type	Direction of test							
Trevira Spunbond	200	mach.	55	75	79	68	73	75	70
		cross	58	67	80	64	67	65	63
	500	mach.	56	70	68	63	63	58	59
		cross	59	66	66	63	58	62	64
Terram	1000	mach.	25	26	29	27	29	32	28
		cross	21	27	28	28	28	27	28
	4000	mach.	24	27	30	32	31	31	31
		cross	28	25	26	28	28	28	31
Polyfelt	TS 500	mach	52	81	98	85	82	[illegible]	82
		cross	31	38	46	42	38	37	37
	TS 700	mach.	60	75	[illegible]	90	86	94	85
		cross	40	36	59	49	46	44	50
Terrafix	ST 309	mach	45	76	64	65	60	57	55
		cross	36	43	38	36	36	35	35
	ST 509	mach.	54	73	78	72	69	70	67
		cross	43	47	44	45	38	41	40
Sodoca	AST 250	mach.	81	105	107	83	98	94	105
		cross	76	81	74	84	79	87	[illegible]
	AST 420	mach.	78	100	114	111	122	102	128
		cross	111	87	124	123	115	116	119

Table 5, Neck down (mm)

Length [mm]			200		100				
Width [mm]			50	100	100	200	300	400	500
Product	Type	Direction of test							
Trevira Spunbond	200	mach.	12	15	29	132	226	325	423
		cross	15	13	42	137	228	325	426
	500	mach.	13	25	39	139	232	326	423
		cross	12	24	44	136	235	326	447
Terrum	1000	mach.	38	83	57	153	253	351	477
		cross	37	85	57	174	256	360	453
	4000	mach.	32	87	68	162	272	366	476
		cross	35	89	76	168	269	371	472
Polyfelt	TS 500	mach.	12	22	44	130	220	322	424
		cross	14	29	51	155	245	366	467
	TS 700	mach.	15	27	59	139	225	364	426
		cross	17	30	54	155	257	364	416
Terrafix	ST 309	mach	18	37	42	132	224	331	441
		cross	18	36	58	158	249	359	444
	ST 509	mach.	17	35	44	134	232	323	436
		cross	19	39	58	165	264	379	444
Sodoca	AST 250	mach.	15	22	40	140	249	332	446
		cross	13	32	57	142	251	352	448
	AST 420	mach.	11	31	34	138	222	327	423
		cross	15	33	24	137	234	340	442

Dia 2 shows the shape of three typical load/extension curves. To the mechanically and heat bonded fabrics in Dia 1 has been added the curve for a mechanically bonded staple fibre fabric.

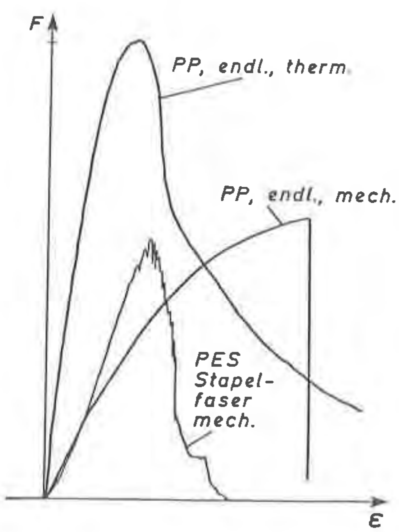

Fig. 2 Load/extension curves.

3 EVALUATION

At the time of preparation of this report, mid-March 1982, the testing in the BASt had not yet been completed. Fabrics from five of eight producers had been tested. This evaluation includes the correlation between load and extension for different strip tensile tests and the CBR test. The presentation in Las Vegas will evaluate all eight products.

3.1 COMPARISON OF DIFFERENT TENSILE TESTS

The results of the 50 X 200 mm strip test were first correlated with those of the 100 X 100 mm to 500 X 100 mm tests. The correlation coefficients are detailed in Tab. 6.

Table 6.

Strip [mm]	r F_R	r ε_R
100 × 100	0,951	
200 × 100	0,967	0,942
300 × 100	0,936	0,936
400 × 100	0,958	0,938
500 × 100	0,954	0,944

The correlation coefficients for F_R average over r = 0,953 and, for twenty value pairs, guarantee the existence of a formal relationship. Dia 3 shows this grafically with a plot of the road/extension curves for one product at widths of 100, 300 and 500 mm.

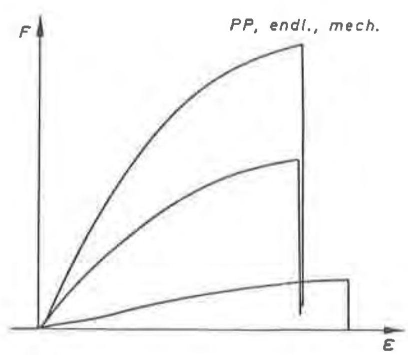

Fig. 3

Analysis of the regression coefficients gives the relationship

$$F_R \text{ (50x200)} = 0{,}8 \; F_R \qquad (1)$$

Dia 4 shows a plot of this relationship.

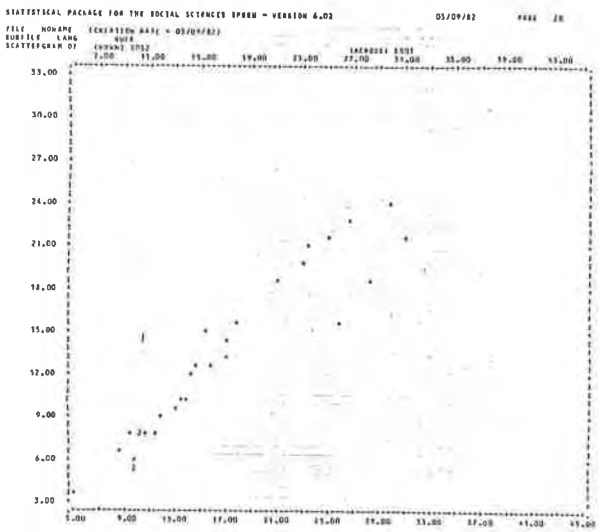

Fig. 4 Relationship Eq. (1)

Similar equations can be formulated to relate the results from different sample widths to one another.

Relating the breaking loads to the fabric weight leads to only a slight alteration in correlation.

The v_K of individual strip tests was calculated for the whole range of fabrics tested. Dia 5 shows that above a sample width of 200 mm, the results of the strip tensile tests are representative for the products in question.

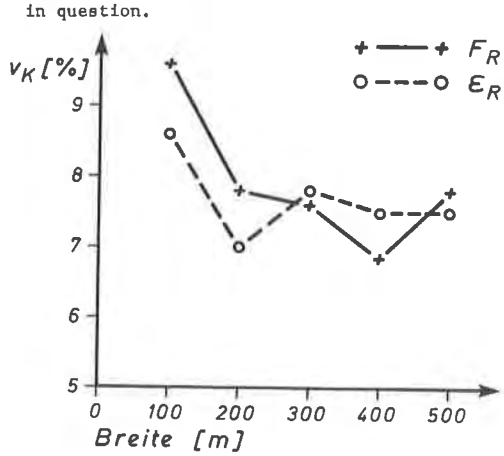

Fig. 5 Variation coefficients

4.2 COMPARISON OF THE CBR AND STRIP TENSILE TESTS

The push through force and the extension at push through were correlated with the breaking load and extension at break of strip tensile tests at differing widths and in different test directions (machine and cross).

Table 7.

Strip [mm]		r all	r mach.	r cross
50 x 200	F_R	0,853	0,878	0,824
	ε_R	0,913	[illegible]	0,945
500 x 100	F_R	0,886	0,826	0,951
	ε_R	0,874	0,853	0,925

The table indicates a poorer agreement between strip and CBR tests than between different strip tests. The agreement between strip and CBR appears to improve with increasing sample width. Further evaluation of these results is planned for the presentation of the paper.

5 CONCLUSIONS

The results discussed here are based on tests on five products which had been evaluated at the time of writing. The inclusion of the remaining three products in the intervening period may lead to alterations and additions at the presentation in Las Vegas.

1. The load/extension behaviour of non-wovens in the CBR test is dependent on the type of bonding. Heat bonded fabrics exhibit a relatively quick load uptake, the extension at push-through is lower, and the breaking point is gradual (not an abrupt tear). The initial extension and the extension at push-through of mechanically bonded fabrics is higher, and they tear abruptly when push-through force is exceeded.

2. The breaking load measured in a tensile test at a certain width can be converted to another width by a multiplication factor. This does not hold true for extension at break. For each product, however, the characteristic load/extension curves for different sample widths are similar and translation from one width to another is possible.

3. As in the CBR test, the load/extension curves of of heat bonded non-wovens in the tensile test exhibit a steeper slope i.e. a higher initial modulus than the mechanically bonded. The work done (the area under the curve) builds up more quickly at first in the case of heat bonded fabrics, but does not attain the value achieved by mechanically bonded geotextiles.

4. A feature of staple-fibre geotextiles are irregularities in the load/extension curves prior to breaking load or push-through force. This suggests that individual groups of fibres are torn or pulled out of the fibre mass.

5. Conversion between CBR and tensile tests is only possible to a limited extent, because of the different stress distribution in the different tests. This is particularly marked in the case of staple-fibre geotextiles.

6. Inhomogeneities in non-wovens have a more pronounced effect on the results of the CBR test than on those of wide width tensile tests.

7. Both types of test should be rated equally when carrying out selection and routine control tests.

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The Measurement of the Tearing Resistance of Geotextiles

La mesure de la résistance à la déchirure des géotextiles

One of the most important properties of a geotextile must be a great tearing resistance. The tearing behaviour is considered into the two following aspects : sollicitations applied with a slaw speed and with a fast one.

Because geotextiles have a great deformability, the usually textile methods are not suitable for all the geotextiles and, particularly for the nonwoven, they measured a tensile strenght.

The results obtained with several nonwoven fabrics, allow to propose a started tearing method with a slow speed and to developp a tearing equipment with a high speed.

These two types of test are able to give the data for a good evaluation of the tearing risk in the works where one of these kinds of sollicitation is to take into account.

Une des propriétés importantes que doit posséder un géotextile est la résistance à la déchirure. Le comportement au déchirement est considéré sous les deux aspects : sollicitations appliquées à vitesse lente et à vitesse rapide.

Compte-tenu de la grande déformabilité des géotextiles, les méthodes traditionnelles du textile ne conviennent pas aux géotextiles car ils conduisent à des essais de traction.

A partir des résultats des essais effectués sur divers géotextiles nontissés et tissés, on propose d'une part, une méthode de déchirure amorcée à vitesse lente (100 mm par minute) sur éprouvette trapézoïdale de grandes dimensions et d'autre part, la définition d'une machine prototype de déchirure à vitesse rapide (210 m par mn).

Ces deux types d'essais peuvent fournir les éléments indispensables à une bonne évaluation du risque de déchirure des géotextiles pour leurs utilisations dans les ouvrages où ces types de sollicitations sont à craindre.

INTRODUCTION

Dans leur utilisation en Génie Civil, les géotextiles peuvent au cours de leur manipulation, de leur mise en oeuvre, ou même durant leur fonctionnement être soumis à des sollicitations localisées se traduisant par l'apparition de perforations de formes et de dimensions variables. Compte tenu des caractéristiques physico-chimiques des polymères entrant dans la composition des géotextiles, une très faible énergie est suffisante pour provoquer une coupure localisée. Suivant l'importance de ces coupures, elles peuvent se traduire par la rupture d'un nombre plus ou moins important de fibres, filaments, fils, crins et intéresser tout ou partie de l'épaisseur du géotextile. Ces discontinuités pourront devenir des amorces de déchirure sous l'action de contraintes de traction qui peuvent se développer au cours de leur mise en oeuvre ou de leur fonctionnement. Ces déchirures sont alors préjudiciables au bon fonctionnement des géotextiles qui doivent assurer une bonne continuité de leurs propriétés tant hydrauliques que mécaniques.

Il est donc important de caractériser le comportement à la déchirure des géotextiles. Suivant le mode de perforation (pieu d'ancrage, cailloux tranchants, souches, chutes de pierres, passages d'engins...) deux processus de déchirure peuvent être envisagés : la déchirure au clou et la déchirure amorcée. Ces deux types de mécanisme ont été étudiés depuis longtemps par les textiliens et font l'objet de normes pour les articles destinés à l'habillement et pour les tissus industriels.

Dans le cadre de cette communication, seule la déchirure amorcée est étudiée. Après une analyse critique des méthodes existantes les deux modes de sollicitation de l'effort sont présentés, c'est-à-dire un essai à vitesse lente (100 mm/minute) et un essai à vitesse rapide (3,5 m/seconde, soit 210 m/mn).

1 ANALYSE DES METHODES EXISTANTES

Un examen des méthodes existantes des déchirures amorcées fait apparaître que la forme des éprouvettes testées est de deux types. Dans le premier, l'éprouvette a une forme rectangulaire et l'amorce de la déchirure est faite au milieu d'un côté et perpendiculairement à ce côté. Les deux languettes ainsi formées sont chacune serrées dans une des mâchoires de l'appareil. Dans le second, l'éprouvette est trapézoïdale. L'amorce de déchirure est perpendiculaire au milieu du plus petit des deux côtés parallèles du trapèze. Les côtés non parallèles sont serrés dans les mâchoires.

Les contraintes devant propager la déchirure à partir de l'amorce de déchirure sont obtenues par des appareils soit de type pendulaire qui engendrent une déchirure à vitesse rapide, soit par des presses de traction où la vitesse de déchirure est lente.

Les essais préliminaires ont montré que les appareils pendulaires (type Eldmendorf) ne fournissent généralement pas, pour l'ensemble des géotextiles, une énergie suffisante pour propager la déchirure dans l'axe de la déchirure.

Si on applique la contrainte avec une presse, c'est à-dire à vitesse lente, à des éprouvettes du type languette, on obtient une déchirure se propageant suivant la ligne de moindre énergie ou une rupture d'une des languette par traction comme le montre la photo de la figure 1. (Résultats rejetés par la norme).

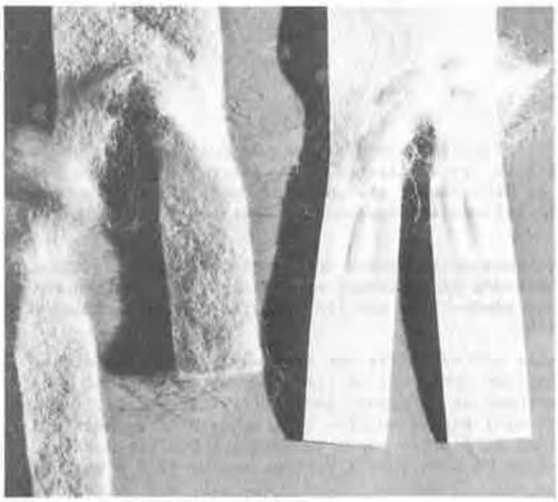

Fig. 1 Essai de déchirure sur languette.

Tableau 1. Comparaison des forces de rupture et des résistances au déchirement

Géotextiles	Masse surfacique g/m2	F daN/m	R_E daN	R'_E daN/m	$\frac{F - R'_E}{F}$ %	F_G daN/m	R_G daN	R'_G daN/m	$\frac{F_G - R'_G}{F_G}$ %
NTAFLPET	210	980	33,2	858	12,4	1 740	165	434	75
"	270	1 400	52,2	1 492	- 6,6	2 235	175	460	79,4
"	340	1 900	56,9	1 626	14,4	2 600	215	565	78,2
NTTSFLPP	200	1 260	50	1 200	4,8	1 320	84	221	83,2
"	270	1 560	44	1 260	19,2	2 000	-	-	-
NTTSFLPEPP	200	1 500	60	1 720	- 14,7	1 915	180	474	75,2
"	280	1 800	70	2 000	11	2 365	180	474	80
NTAFLPP	200	1 000	40	1 400	- 18,3	1 010	110	289	71,4
"	370	2 000	70	2 000	0	1 789	215	565	68,4

F = Force de rupture par traction, éprouvette de 50 mm x 200 mm
F_G = Force de rupture par traction, éprouvette de 500 mm x 100 mm

R_E = Résistance au déchirement, éprouvette Edana
R_G = Résistance au déchirement, éprouvette C F G

NTAFL = Nontissé aiguilleté filaments continus NTTSFL = Nontissé thermosoudé filaments continus
PE = Polyéthylène, PP = Polypropylène, PET = Polyester

Les essais effectués sur des éprouvettes trapézoïdales selon la méthode Edana 70-0-75 ont conduit à des constatations qui mettent en cause la signification de ce mode d'essai en tant que mesure du comportement à la déchirure. Il a été en effet constaté tant pour des aiguilletés que pour des tissés que la rupture de l'échantillon ne se produisait pas par déchirure, mais par traction, ainsi que le montre la courbe de la figure 2.

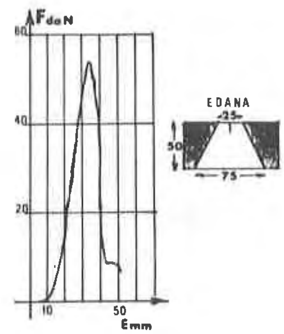

Fig. 2 Courbe de déchirure obtenue sur éprouvette EDANA pour un nontissé aiguilleté.

Il ne paraît donc pas opportun de retenir cet essai pour les géotextiles puisqu'il ne conduit pas toujours à une déchirure pour tous les matériaux testés. Nous avons donc été amenés à procéder à des essais sur des éprouvettes de grandes dimensions et de forme trapézoïdale pour être assurés que la concentration de l'effort soit toujours située au fond de la déchirure.

2 ESSAIS A VITESSE LENTE

2.1 Influence de la dimension de l'éprouvette

Soit F la force de rupture d'un géotextile obtenue sur une bande de 5 cm x 20 cm et évaluée en daN/m et R_E la force de déchirement correspondante à des éprouvettes trapézoïdales de type EDANA. La hauteur de l'éprouvette est 5 cm, la longueur de l'amorce de la déchirure est 1,5 cm. On peut donc calculer à partir de R_E la force de rupture R'_E par "traction" qui correspond à une bande de 3,5 cm exprimée en daN/m. La comparaison des valeurs de F et R'_E du tableau 1 fait apparaître qu'elles ne diffèrent pas de plus ou moins 20 %. Soient maintenant F_G la force de rupture d'un géotextile obtenue à partir d'éprouvettes de grandes largeurs (50 cm x 10 cm) exprimée en daN/m et R_G la force de déchirement obtenue avec des éprouvettes trapézoïdales de grandes dimensions (9 fois Edana). La hauteur du trapèze formant ces éprouvettes est de 45 cm et la longueur de l'amorce de déchirure est de 7 cm. On peut alors, comme précédemment, calculer à partir de R_G la force de rupture R'_G correspondant à une éprouvette de 38 cm de large et exprimée en daN/m. Les valeurs correspondant à divers géotextiles figurent dans le tableau 1. On constate que les valeurs de R'_G sont très inférieures à celles de F_G, de l'ordre de 70 à 80 %. On peut donc admettre que, contrairement aux essais avec éprouvettes Edana qui conduisent à des résistances de déchirement qui sont en fait des forces de rupture par traction, ceux effectués sur des éprouvettes trapézoïdales de grandes dimensions donnent effectivement des résistances de déchirement.

La photo de la figure 3 représente la déchirure d'une grande éprouvette et montre la propagation de la déchirure parallèlement à l'axe de l'amorce de déchirure. La courbe effort-déformation de la figure 4 est bien représentative d'un mécanisme de déchirure et non pas d'une traction comme l'est la courbe de la figure 2 obtenue au cours d'un essai de déchirure sur éprouvette Edana.

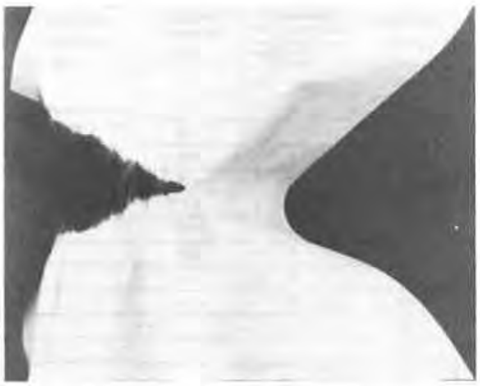

Fig. 3 Essai déchirure à vitesse lente sur macrotrapèze CFG.

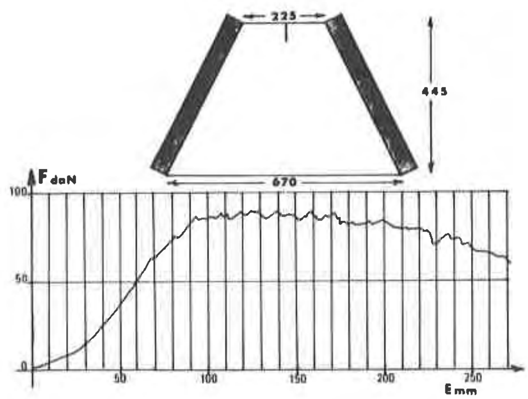

Fig. 4 Courbe de déchirure sur macro-trapèze pour un nontissé aiguilleté.

Des essais de déchirure faits avec des vitesses de traction différentes ont montré que la résistance de déchirement, dans le domaine des vitesses compatibles avec le matériel d'essai (c'est-à-dire compris entre 50 et 500 mm/minute) est indépendante de cette vitesse. On a par exemple obtenu la même valeur de 20 daN pour un nontissé thermosoudé de filaments continus de polypropylène que la vitesse de traction soit de 50 ou 500 mm/minute.

Que ce soit pour des essais à vitesse lente ou à vitesse rapide, l'analyse ci-dessus fait ressortir qu'il est donc nécessaire d'utiliser des grandes éprouvettes. D'autre part, pour que la concentration de l'effort soit bien située au fond de la déchirure, il paraît nécessaire d'utiliser des éprouvettes de forme trapézoïdale.

2.2 Proposition d'un méthode d'essai et résultats

A partir des données obtenues, on propose une méthode de déchirement amorcé à vitesse lente. Elle est adoptée par le Comité Français des Géotextiles et est en cours de normalisation à l'AFNOR (Agence française de normalisation).

Son principe est d'exercer un effort de traction sur les côtés non parallèles d'une éprouvette trapézoïdale ayant une amorce de déchirure au milieu du plus petit des côtés parallèles.

Les dimensions de cette éprouvette sont : petite base : 250 mm, grande base : 670 mm, hauteur : 445 mm, côtés non parallèles : 500 mm. Il est à prévoir une bande de 30 mm sur les côtés non parallèles pour pouvoir serrer l'éprouvette dans les mâchoires.

La vitesse de traction est de 100 mm/minute.

Le nombre d'éprouvettes est d'au moins : 6.

Pour obtenir la force de déchirement, sur la courbe force de déchirure - longueur de déchirure, on partage la zone de déchirure en 5 parties égales. Dans chacune, on relève la force correspondant au pic maximum et la force de déchirement est la moyenne des valeurs des forces correspondant à ces cinq pics.

Les valeurs de force de déchirement obtenues selon cette méthode pour un certain nombre de matériaux représentatifs des géotextiles : nontissés aiguilletés de filaments continus, nontissés de fibres courtes, nontissés thermosoudés, tissés de fils sont rassemblées dans le tableau 2.

Tableau 2. Résistance au déchirement de quelques géotextiles en daN.

Géotextiles	Masse au m2 en g	Sens production	Sens travers	Géotextiles	Masse au m2 en g	Sens production	Sens travers
NTAFLPET	150	120	80	NTTSFLPP-PE	100	80	60
"	210	210	120	"	140	120	80
"	270	200	150	"	200	160	110
"	340	260	170	"	240	200	160
"	550	400	270	"	280	200	160
NTAFLPP	130	70	80	NTTSFLPP	200	20	20
"	200	100	120	Tissé PP	100	98	94
"	240	120	130	"	140	126	88
"	340	150	180	"	200	100	79
"	370	250	180	"	340	125	118
"	600	315	264	"	540	143	228

Suivant la nature des géotextiles, les résistances au déchirement sont très différentes. Elles peuvent varier de 20 à 400 daN.

3 ESSAIS A VITESSE RAPIDE

L'essai décrit ci-dessus à vitesse lente peut simuler un certain nombre de situations réelles rencontrées sur les chantiers, mais il ne permet pas d'appréhender le comportement des géotextiles dans un certain nombre d'autres cas réels si celui-ci est soumis à des efforts dynamiques. On peut citer par exemple le cas d'emploi d'un géotextile dans une tranchée drainante soumis à une tension brutale lors de sa mise en oeuvre ou par suite de la chute d'un bloc ou objet lourd, alors qu'il n'est pas encore en contact avec les parois de la tranchée.

Afin de mettre en évidence l'influence de la vitesse de l'application de l'effort, il a été nécessaire de concevoir un matériel permettant d'effectuer des essais sur des éprouvettes de grandes dimensions. Un premier appareillage expérimental a été réalisé et utilisé par le CEMAGREF.

Il s'agissait de disposer d'une machine simple dont les premiers enseignements devaient permettre de décider de l'intérêt de l'essai dynamique et par suite de la mise au point d'un matériel d'essai de déchirure à vitesse rapide.

Dans cet appareillage, figure 5, la force dynamique appliquée à l'éprouvette résulte de la chute libre d'un pendule venant frapper le mors pivotant du bâti de fixation de l'échantillon. Ce dernier est de dimension comparable au macrotrapèze, il est rectangulaire ; la propagation de la déchirure peut se produire suivant la plus grande dimension de l'éprouvette, soit sur 35 cm.

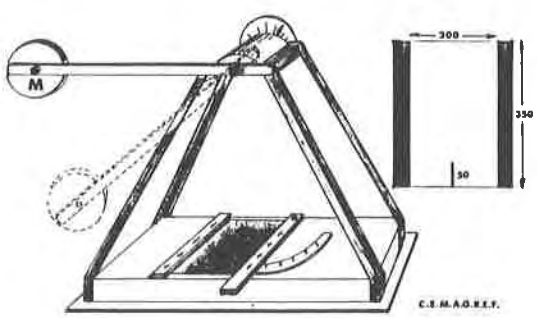

Fig. 5 Appareillage expérimental et déchirure dynamique. Matériel CEMAGREF. M peut varier de 1 à 10 kg

On a donc procédé à une série d'essais dynamiques en faisant varier la masse du pendule de 1, 5 et 10 kg. Vingt-deux échantillons appartenant à quatre grands types de géotextiles : nontissés de fibres continues, nontissés de fibres courtes sur tissés de bandelettes, liés mécaniquement, nontissés thermosoudés et tissés ont été testés.

Le tableau 3 donne les résultats obtenus en fonction du type de produit et de l'énergie appliquée.

Leur comparaison avec les valeurs de force de rupture obtenues pour un certain nombre de géotextiles à l'essai lent de déchirure sur macrotrapèze, met bien en évidence le comportement différent selon le type d'essai. Par exemple, alors que les forces de rupture à l'essai CFG pour un nontissé des fibres continues lié mécaniquement et pour un nontissé de fibres continues thermosoudé sont identiques, on enregistre avec l'essai dynamique une déchirure totale à l'énergie de 50 joules pour le second alors que sur le premier, même à 100 joules, on n'obtient pas de déchirure.

Il est certain que la propagation d'une déchirure fait intervenir les liaisons entre fibres et la mobilité des fibres les unes par rapport aux autres. La résistance à la déchirure peut donc avoir une valeur différente selon qu'elle est mesurée lentement ou dynamiquement.

Tableau 3. Résultats comparés des longueurs de déchirure obtenues à l'essai dynamique et des forces de déchirure obtenues à l'essai lent

GEOTEXTILE	MASSE SURFACIQUE g/m^2	ESSAI CEMAGREF LONGUEUR DE DECHIRURE (CM)			ESSAI ITF FORCE DE DECHIRURE daN	OBSERVATIONS SUR ESSAI CEMAGREF
		10 J	50 J	100 J		
NONTISSE DE FIBRES CONTINUES POLYESTER AIGUILLETE	150	0	0	0	80	
	210	0	0	0	120	ALLONGEMENT DES FIBRES LA MASSE DEPASSE LE MORS MOBILE
	340	0	0	0	170	
	550	0	0	0	270	
NONTISSE DE FIBRES CONTINUES POLYPROPYLENE AIGUILLETE	200	0	2,5	3,5	120	
	270	0	0	2	130	
	370	0	0	0	180	
NON TISSE DE FIBRES CONTINUES POLYPROPYLENE ET POLYAMIDE THERMOSOUDE	70	26	34	34	80	
	210	23	34	34	110	
	280	10,5	34	34	160	
NONTISSE FIBRES CONTINUES POLYPROPYLENE THERMOSOUDES	140	5	11,5	34		
	210	7	34	34		
	270	7,5	17,5	34		
NONTISSE FIBRES COURTES	420	0	3,5	4,5		
NONTISSES FIBRES COURTES POLYPROPYLENE ARMES SENS FABRICATION FILS POLYESTER	300	0	1,8	4		MOBILISATION ARMATURES
		1	4,1	2		
	370	0	0	0,8		MOBILISATION ARMATURES
		1	3	5		
NONTISSES POLYPROPYLENE THERMOSOUDE	200	11	34	34	20	
TISSE DE FILS POLYPROPYLENE	200	0	0	4		GLISSEMENT DE L'ECHANTILLON DANS LES MORS
	250	0,5	4	12		
	275	1	7,5	13,5		

Fig. 6 Nontissé de fibres continues liées par aiguilletage après essai dynamique. Energie 100 J.

Le matériel utilisé par le CEMAGREF, de conception simple et conçu dans un premier temps pour aborder le comportement des géotextiles à la déchirure dynamique a permis malgré ces caractéristiques modestes, de mettre en évidence les principaux points suivants :

- Les énergies disponibles sont faibles, il est donc normal de ne pas obtenir de déchirure dans un certain nombre de nontissés liés mécaniquement. Au choc, une partie de l'énergie est absorbée par le géotextile, qui s'allonge, la masse échappe et une autre partie de l'énergie est dissipée dans la remontée du pendule. On obtient un réarrangement des fibres au fond de la déchirure amorcée mais celles-ci ne sont pratiquement pas rompues, figure 6 ; le géotextile s'allonge.

- Pour les nontissés thermosoudés, aux fibres très liées, autorisant peu de mouvement des fibres les unes par rapport aux autres, on obtient toujours une déchirure (figures 7 et 8). La longueur de déchirure est fonction de la masse surfacique et de l'énergie appliquée.

Fig. 7 Nontissé de fibres continues thermosoudées après essai dynamique. Energie 50 J.

- Avec les tissés les conditions de serrage de l'éprouvette n'ont pas permis d'éviter un glissement des fibres dans les mors. Les résultats sont de ce fait assez peu significatifs.

En conclusion, l'essai dynamique fait apparaître un comportement des géotextiles très différent de celui de l'essai à vitesse lente. Pour permettre le jeu des liaison entre fibres, il doit être réalisé sur des éprouvettes de grandes dimensions et par souci d'homogénéité avec l'essai lent, de format trapézoïdal.

4 PROPOSITION D'UN MATERIEL D'ESSAI

Une réflexion nourrie par les enseignements des essais du CEMAGREF et de l'ITF, a permis de réaliser un matériel étudié et mis au point dans le Centre d'Atelier des Prototypes d'ANGERS/FRANCE.

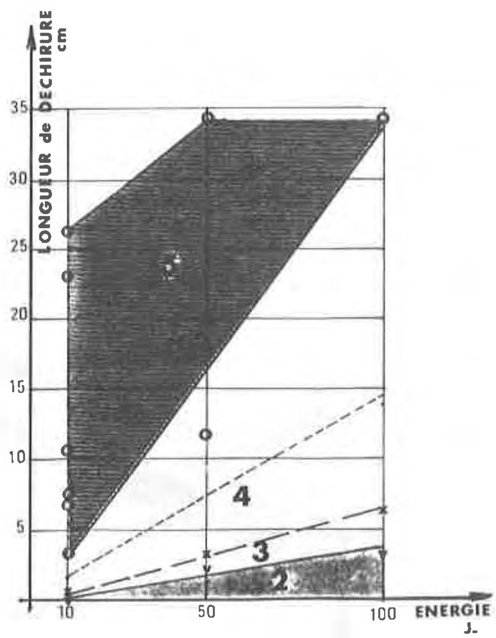

Fig. 8 Longueur de déchirure à l'essai dynamique en fonction de l'énergie appliquée et du type de géotextile
2 - nontissé fibres continues aiguilleté
3 - nontissé fibres courtes aiguilleté sur tissé de bandelettes
4 - tissé
5 - nontissé fibres continues thermosoudé

4.1 Principe de la machine

L'échantillon de géotextile préalablement découpé en forme de trapèze est introduit et pincé dans les mors fixes et mobiles.

La position du mors mobile correspond à un tissu tendu selon sa petite base et dont la déchirure a pu être amorcée.

Le mors mobile accéléré à 120 m/s^2 atteint sa vitesse nominale après avoir parcouru 0,05 m. Au-delà, il progresse à vitesse constante guidé sur rails par un ensemble de galets reprenant avec de très faibles frottements toutes les dissymétries de l'effort de déchirure.

La disposition horizontale des mors évite l'application de tout effort parasite avant la déchirure.

4.2 Description de l'appareil figure 9

La machine comprend un châssis mécanosoudé formé d'un fer plat de forte épaisseur disposé dans un plan vertical appuyé à l'une de ses extrémités sur un socle.

La partie supérieure du plat reçoit un profil de guidage. Ce profil supporte un mors mobile en alliage léger équipé de galets.

En regard du mors mobile, le mors fixe est relié au châssis par deux capteurs d'efforts.

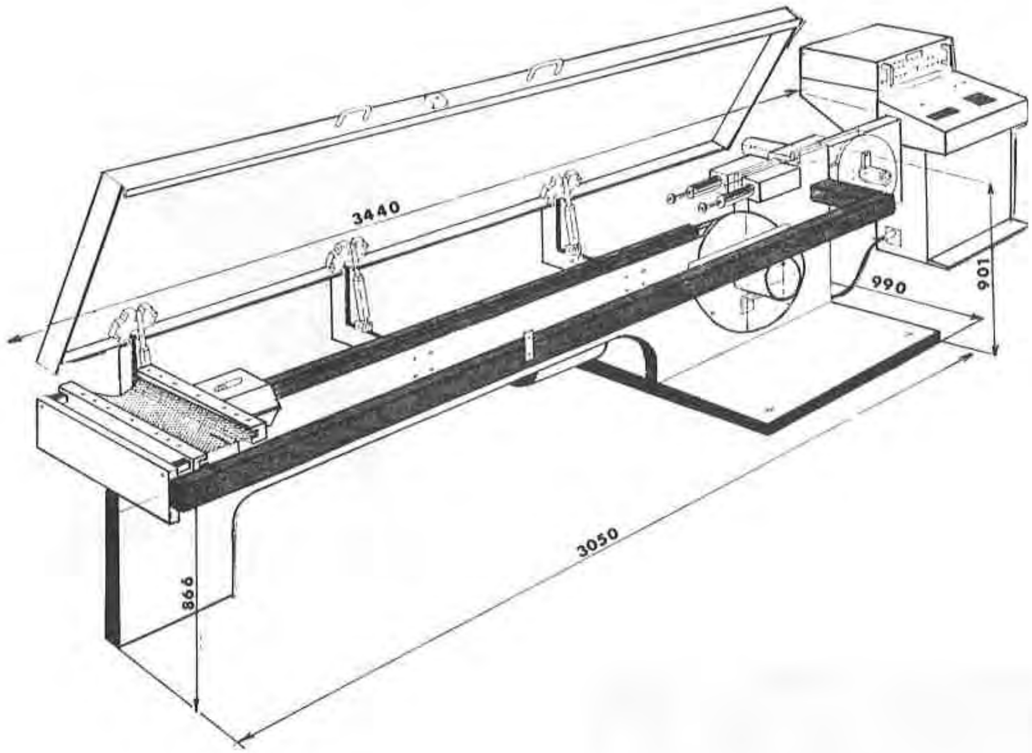

Fig. 9 Machine pour essai de déchirure à vitesse rapide (3,5 m/s), conçue et réalisée par le C.E.C.P. d'ANGERS. Matériel des Laboratoires des PONTS ET CHAUSSEES (France).

Ces deux mors sont disposés dans un plan horizontal.

La cinématique comprend un moteur asynchrone, un réducteur, un embrayage électromagnétique, une poulie à gorge, sur lequel s'enroule un câble solidaire du mors mobile.

En fin de course, le circuit de l'embrayage est ouvert et l'énergie cinétique du mors mobile est absorbé par deux amortisseurs.

Un codeur et deux capteurs d'efforts associés à un enregistreur de signaux transitoires donnent accès à l'énergie de déchirure.

Divers organes de sécurité imposent une procédure stricte dans la préparation et le déroulement de l'essai.

Tous les organes de commande, de contrôle et d'enregistrement sont regroupés sur un pupitre.

A la cinématique propre à l'essai de déchirure peut se superposer une cinématique adaptée à l'essai de traction. On peut aussi mettre en place dans ce cas une charge sur l'échantillon, permettant d'appliquer une contrainte normale sur le géotextile, en simulation de cas réels.

Les puissances installées permettent d'atteindre une vitesse linéaire nominale de 3,5 m/s, soit de l'ordre de 2 000 fois supérieure à celle de l'essai de déchirure à vitesse lente. L'effort nominal de déchirure peut atteindre 3 000 N. et l'allongement autorisé avant rupture totale est de 200 %.

Les délais de fabrication n'ont pas permis de disposer de ce matériel actuellement en cours de montage et dont la livraison est prévue pour mi-Avril 1982. Les résultats d'essais seront présentés en Colloque.

5. CONCLUSIONS

Les recherches menées par l'Institut Textile de France, le CEMAGREF et les Laboratoires des Ponts et Chaussées sur les essais de déchirure, ont permis de proposer un essai de déchirement à vitesse lente (V = 100 mm/mn) applicable à tous les géotextiles sans distinction (tissés ou nontissés).

De plus, il a été mis en évidence l'importance de la vitesse d'application de l'effort. Ceci a abouti à la réalisation d'un matériel d'essai nouveau permettant de bien caractériser le comportement des géotextiles à ce type de sollicitation rapide (V = 210 m/mn).

Il n'a pas été fait part des études menées dans le même esprit sur la déchirure au clou. Rappelons simplement que les travaux réalisés conduisent à proposer un essai au clou sur des éprouvettes de grandes dimensions 200 mm x 300 mm seules susceptibles de rendre compte du comportement de tous les géotextiles à ce type de sollicitation.

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Creep Behavior of Geotextiles Under Sustained Loads

Le fluage des géotextiles sous charges permanentes

Selection of design tensile stress and tensile strain of geotextiles depends upon their time dependent behavior under sustained (static) and/or repetitive (dynamic) loading conditions. Creep of six typical geotextiles under static loads was investigated to develop their creep parameters using an analogy of a four-element rheological model based on Rate Process Theory, and by using an empirical three-parameter creep equation based on curve-fitting of the laboratory data. The parameters were then used to predict the long-term creep behaviors of the six geotextiles. Both methods predicted the time required to reach failure strains as being much shorter than the normal design lives of geotextiles uses at stress level as low as thirty percent of the ultimate. The creep strain-time relationships predicted by the Rate Process Theory appeared to be more consistent with the experimental data than those predicted by the empirical method.

Le choix, pour les géotextiles, d'une contrainte de tension admissible et de leur allongement correspondant, depend de leur comportement qui est fonction du temps, sous les deux conditions de chargement suivantes: charge permanente (cas statique) et charge cyclique (cas dynamique). Le fluage de six géotextiles types sous charge statique a ete étudié afin de trouver les paramètres de fluage; un modèle rhéologique à quatre elements fondé sur la théorie du "Rate Process" a été utilisé ainsi qu'une équation empirique à trois variables obtenue à partir d'une courbe tracée suivant les resultats de laboratoire. Ces deux méthodes ont montré que le temps nécessaire pour obtenir des deformations à la rupture, est beaucoup plus court que la durée de vie admissible des géotextiles soumis à des niveaux de contraintes aussi bas que 30% de celui de la rupture. Les relations de fluage, entre déformation et temps, que l'on a établies à partir de la théorie du "Rate Process" paraissent d'avantage compatible avec les résultats obtenus par expérience qu'avec ceux obtenus par la méthode empirique.

INTRODUCTION

The time-dependent tensile stress-strain behavior of geotextiles under sustained and/or repetitive loads are important for their satisfactory performance in civil engineering applications. For example, an earth wall reinforced with geotextile may fail by excessive deformation due to the unchecked creep of the geotextile even though adequate factors of safety are provided against reinforcement rupture and pullout (1). Similarly, in a road stabilization application, failure may occur due to excessive deformation of the road embankment as a result of excessive creep in the geotextile. If the dead load is much greater than the live load, such as under a thick road embankment, the static creep would be more important in controlling the deformation. However, if the live load is greater than the dead load, such as under a thin road embankment, the dynamic creep would be more important than the static creep in controlling the deformation (2). The design tensile load and tensile strain selected for applications such as these would depend upon the static and/or dynamic creep characteristics of the geotextiles.

There have been attempts by engineers and researchers to characterized the static creep behavior of geotextiles by using simple rheological models and curve-fit methods. Nevertheless, there is a need for a means of not only relating the sustained load and creep strain, but also of predicting the behavior of geotextiles that is based on a rational, analytical theory. One theory which has the potential of fulfilling this need is the Rate Process Theory.

The purpose of this paper is to describe a laboratory investigation to determine the validity of the Rate Process Theory in predicting the tensile creep behavior of geotextiles under sustained load. The observed creep behavior was analyzed by using an analogy of a four-element rheological model whose spring and dashpot constants were determined on the basis of the Rate Process Theory. The creep data were also analyzed on the basis of a curve-fit method which results in a three-parameter phenomenological equation similar to the one used for soils by Singh and Mitchell (3). The parameters developed from the analyses were then used to predict the time for the geotextiles to reach their failure strains under sustained load.

CREEP AS A RATE PROCESS

The Rate Process Theory formulated originally by Eyring (4) has had wide applications in recent years to many processes involving time-dependent rearrangement of matter such as the deformation of materials under stress. M. Hogan (5) used this theory to investigate the creep characteristics of several plastic laminae and it was found to be a highly satisfactory engineering hypothesis applicable to that particular material.

Herrin and Jones (6) and Hady and Herrin (7) used this theory to describe the shear-deformation and creep characteristics of bituminous materials. They found that the behavior of asphalt seemed to be satisfactorily explained by the theoretical considerations suggested by the Rate Process Theory. This theory has also been successfully used by several investigators to describe the creep and consolidation behavior of soils under stress (8,9,10).

Coleman and Knox (11) observed in their theoretical analysis of the strength properties of textile fibers that it may be possible to predict the stress-strain-time behavior of some textile fibers under both instances of static and dynamic loading by using the Rate Process Theory.

The details of the derivation of the theory is not presented in this paper due to limitation of space. The basis of the relationship is that the atoms and molecules participating in a deformation process are constrained from movement relative to each other by virtue of the energy barriers separating equilibrium positions as shown schematically by Curve A in Figure 1(a). When an external force is applied to the system, flow of the material occurs and it is assumed that the flow takes place by the movement of atoms, molecules or aggregates of molecules (called flow units) into vacancies in the material, or by displacement of the vacancies themselves within the material (6). The displacements of flow units to new positions require that they become "activated" through acquisition of sufficient energy, ΔF, known as the free energy of activation, to surmount the energy barrier.

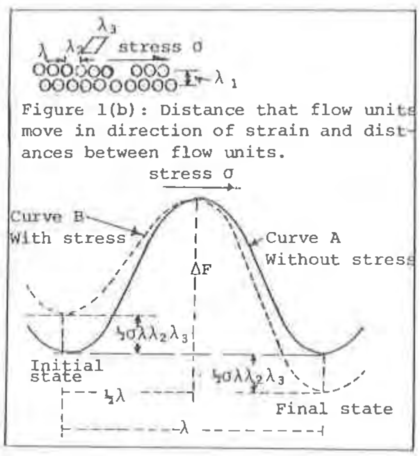

Figure 1(b): Distance that flow units move in direction of strain and distances between flow units.

Figure 1 (a): Energy barriers for strain with and without external applied stress

It is assumed in this theory that the flow units are arranged as shown in Figure 1(b). From statistical mechanics, it is known that the flow units in fact are not at rest but vibrate with a frequency of kT/h, as a consequence of their thermal energy, and the specific rate of process is given by Eq. 1 (4).

$$K_r = \frac{kT}{H} e^{\frac{-\Delta F}{RT}} \qquad (1)$$

where: K_r = specific frequency of activation
k = Boltzmann's constant (1.38×10^{-16} erg $^\circ K^{-1}$)
h = Planck's constant (6.624×10^{-27} erg sec $^{-1}$)
T = absolute temperature, $^\circ K$
R = the universal gas constant = 1.98 cal $^\circ K^{-1}$ mole $^{-1}$
ΔF = free energy of activation.

When an external energy is added to the system on application of a force, the original energy barrier becomes distorted as shown by Curve B in Figure 1(a), such that the energy barrier height is reduced in the direction of force and raised the same amount in the direction opposite to the force. Eyring, et al. (4) examined the nature of this phenomenon at considerable length and obtained for the rate of strain the following relationship.

$$\dot{\varepsilon} = 2\frac{V_h}{V_d}\left(\frac{kT}{h} e - \frac{\Delta F}{RT}\right) \sin h \frac{V_h}{2kT}\sigma \qquad (2)$$

where: ε = the rate of strain
σ = applied stress
V_h = $\lambda_1\lambda_2\lambda_3$ = flow volume of a flow unit
V_d = $\lambda\lambda_2\lambda_3$ = volume of a flow unit
λ = distance the flow units move in the direction of strain
$\lambda_2\lambda_3$ = the cross sectional area of the flow units
λ_1 = distance between flow units.

When a material is subjected to a constant tensile stress of sufficient magnitude the material undergoes a continuous deformation with the passage of time. If the unit strain observed for the body is plotted as ordinate and time as abscissa there is obtained, in general, a curve of the form shown in Figure 2(a). On application of the stress, σ, an instantaneous deformation, ε_o, occurs which remains constant with time, see Figure 2(b).

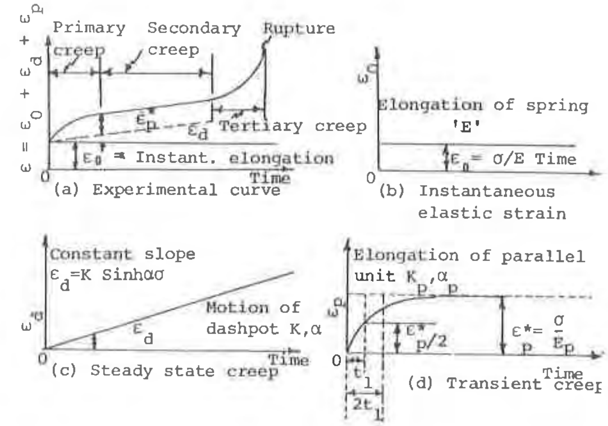

Figure 2: Illustration of creep

With the elapse of time, deformation continues with variable and constant rates, simultaneously. The variable rate of deformation is referred to as the "transient creep component" and the constant rate of deformation is known as the "secondary creep component" of the creep action. These are represented in Figures 2(d) and 2(c), respectively. Following the secondary creep is another phase which is commonly referred to as the "tertiary creep". Tertiary creep occurs at a rapidly increasing rate and eventually leads to failure by rupture.

Engineering interest in the creep properties of a material is largely in that sustained period of time, known as the secondary creep range, during which the creep rate is practically constant (5). Since no engineering use could be made of a material that is in a process of failure, the nature of the tertiary creep is not of any engineering interest. However, the stress level and time at which it develops must be predicted so that it can be avoided.

The creep of a material can be conveniently analyzed by using an analogy of the four-element mechanical model shown in Figure 3. The instantaneous deformation, on application of the load, is represented in the model by the elastic spring E. This constant deformation is graphically represented by Figure 2(b). The transient creep component is represented in the model by the unit consisting of the spring E_p and the dashpot K_p, α_p in parallel. The secondary creep component is represented by the open dashpot K, α.

Of the two constants K and α of the viscous element, K specifies the rate of flow of the dashpot and is expressed in reciprocal seconds and the constant α denotes the resistance of the element to external force and has

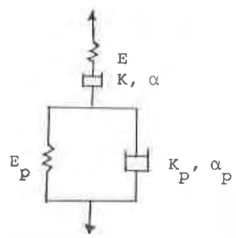

Figure 3: Burger Model.

the units of reciprocal of stress. The point to be noted is that in most rheological models, the viscous element is represented by a single constant known as the viscosity. However, in the rheological model used in this analysis, the viscous element is represented by two constants K and α whose values are determined by the use of the Rate Process Theory. The advantage of this is that the nonlinearity of the viscous element can be represented as subsequently discussed.

Let E be the spring modulus of the open spring (which is the usual modulus of elasticity of fabrics obtained by static loading) and E_p represent the like property of the parallel spring. Furthermore, let σ be the stress applied to the four-element model and σ_d be the stress acting on the parallel viscous element than $(\sigma-\sigma_d)$ will be the stress acting on the parallel elastic element. When the stress σ is applied, the open elastic element E experiences an instantaneous strain ε_o which is given by

$$\varepsilon_o = \frac{\sigma}{E} \tag{3}$$

With the elapse of time, the open viscous element slips at a rate of flow which is given by Eq. 4 derived from the Rate Process Theory.

$$\frac{d\varepsilon_d}{dt} = \dot{\varepsilon}_d = K \sinh \alpha\sigma \tag{4}$$

where: ε_d = strain in the open viscous element

$K = 2 \frac{V_h}{V_d} \frac{kT}{h} e^{-\Delta F/RT}$

$\alpha = V_h/2kT$

t = time.

The steady-state condition of the two open elements subjected to constant stress is given by Eqs. 3 and 4.

The two parallel units representing the transient component of creep deformation each experience the same unit strain at any moment. That is,

$$\frac{d\varepsilon_p}{dt} = \dot{\varepsilon}_p = \frac{1}{E_p} \frac{d(\sigma-\sigma_d)}{dt} = K_p \sinh \alpha_p \sigma_d \tag{5}$$

where: ε_p = creep strain of the parallel elements

$K_p = 2 \frac{V_{hp}}{V_{dp}} \frac{kT}{h} e^{-\Delta F/RT}$

$\alpha_p = \frac{V_{hp}}{2kT}$

From Eq. 5, the following relationships can be derived

$$E_p = \frac{\sigma}{\varepsilon_p^*} \tag{6}$$

$$\ln \frac{\tanh \frac{\alpha_p \sigma}{2} \left(1 - \frac{\varepsilon_p}{\varepsilon_p^*}\right)}{\tanh \frac{\alpha_p \sigma}{2}} = K_p E_p \alpha_p t \tag{7}$$

where: ε_p^* = transient creep at t = infinity.

Equation 7 expresses the variation of the transient creep strain ε_p as a function of time for a constant stress σ. Then, the total strain at any time t, due to a constant stress σ, is given by the sum

$$\varepsilon = \varepsilon_o + \varepsilon_d + \varepsilon_p \tag{8}$$

THREE-PARAMETER CREEP THEORY

Singh and Mitchell (3) found the simple phenomenological equation derived from creep data to hold for creep behavior of a variety of clays. The data obtained from constant stress creep tests of the clays were plotted as shown in Figure 4 and the following creep equation was derived from the slopes of these plots:

$$\dot{\varepsilon}_{(t,\sigma)} = A e^{\bar{\alpha}\bar{\sigma}} \left(\frac{t_1}{t}\right)^m \tag{9}$$

for $t_1 = 1$, Eq. 9 reduces to

$$\dot{\varepsilon}_{(t_1,\sigma)} = A e^{\bar{\alpha}\bar{\sigma}} \cdot \frac{1}{t^m} \tag{10}$$

where: $\dot{\varepsilon}_{(t,\sigma)}$ = strain rate at time t and deviator stress σ

$\bar{\sigma}$ = ratio of deviator stress of interest to maximum deviator stress

t = time

$m, \bar{\alpha}, A$ = experimentally obtained constants.

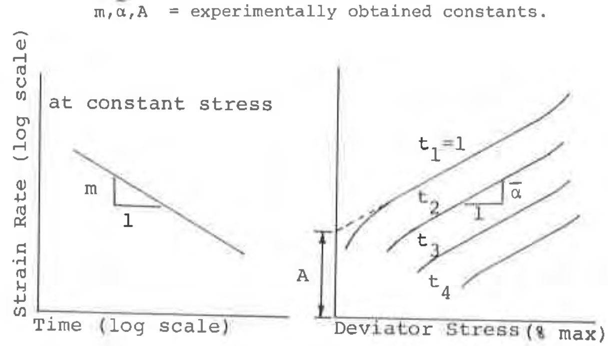

Figure 4 : Method of plotting creep data to empirically determine the constants m, $\bar{\alpha}$, A for three-parameter creep equations. After Singh and Mitchell

Integrating Eq. 10 within the time limit $t = t_1 = 1$ to any time t, creep strain can be expressed as:

$$\varepsilon = \varepsilon_1 + \frac{A}{1-m} e^{\bar{\alpha}\bar{\sigma}} (t^{1-m} - 1) \text{ for } m \neq 1 \tag{11}$$

$$\varepsilon = \varepsilon_1 + A e^{\bar{\alpha}\bar{\sigma}} \ln t \text{ for } m = 1 \tag{12}$$

where ε_1 = creep strain at unit time.

Singh and Mitchell (3) proposed these relationships for creep stresses from 30% to 90% of the initial soil strength, depending upon the susceptibility of the soil to fail in creep rupture (tertiary creep). Note that $\bar{\alpha}$ cannot have negative values, because negative values of $\bar{\alpha}$ would imply that creep strain rate and creep strain are both lower for higher stresses; this phenomenon would be opposite of the normal creep behavior of both soils and geotextiles.

APPARATUS AND EXPERIMENTAL PROCEDURE

The apparatus used for experimental work was a MTS closed-loop testing machine, automatic load and elongation (x-y) recorder, and special fabric clamps consisting of 229 mm (9 inches) wide by 57 mm (2-1/4 inches) high jaws with serrated faces.

Specimen size used for the creep tests were 200 mm (8 inches) wide by 100 mm (4 inches) initial gauge length. The specimen was installed in the two special jaws mounted in the tensile test machine, and a predetermined tensile load applied under a load-controlled mode. The load was applied by a ramp function such that the desired load was reached in 0.5 seconds and held constant thereafter until the end of the test. The initial and the creep displacement were measured on the x-y recorder. The load and displacements were checked regularly by a digital voltmeter and a measuring scale respectively to check the displacements recorded on the x-y recorder.

The sustained load and the load level used on the four nonwoven and two woven geotextiles selected for the test are presented in Table 1. A total of 12 creep tests consisting of two stress levels for each of six geotextiles were performed.

Table 1. Stress Levels for Creep Tests

Fabric	Geotextile Construction	Filament	Sustained Load N/cm (lbs/inch)	Load Level* %
NW-1	Nonwoven, Resin bonded	Polyester, Continuous	32 (18) 44 (25)	40 57
NW-3	Nonwoven, Heat bonded	Polypropylene, Continuous	47 (27) 86 (49)	35 63
NW-5	Nonwoven, Needlepunched	Polypropylene, Continuous	0.11 (2.2)+ 0.20 (3.8)+	37 57
NW-6	Nonwoven Needlepunched	Polypropylene, Staple	26 (15) 46 (26)	33 56
W-4	Woven	Polypropylene, Monfilament	126 (72) 180 (103)	31 44
C-1	Woven, with Needled Nap	Polypropylene, Slit Film	77 (44) 117 (67)	36 55

*Stress level is the sustained stress as % of the maximum strength.

+Stress normalized to weight per unit area in $\frac{N/cm}{gm/m^2}$ $\left(\frac{lbs/inch}{oz/yd^2}\right)$.

The laboratory room temperature and humidity were 68° to 82°F and 20% to 58% respectively.

RESULTS

Table 2 shows the creep of the six geotextiles tested for 20 hours under constant stress. Lowest creep was exhibited by polyester resin bonded NW-1 and the highest creep was exhibited by polypropylene needlepunched NW-6. Creep was most sensitive to load levels for the continuous filaments polypropylene geotextiles, NW-3 and NW-5, which showed increased creep of 3 to 5 times when the sustained load was doubled. For NW-1 which was polyester and NW-6 which had staple filaments, creep increased by only 1% to 2% for increase in the sustained load levels from 50% to 57% and 33% to 56% respectively. The woven W-4 and C-1 had lower creep than most nonwovens.

Table 3 presents the constants for the three-parameter creep equation developed from the creep data. The values of m did not differ greatly from one fabric type to another. Greatest variation was found in the values of A with lowest values for NW-1 and highest for NW-6 which had the lowest and the highest creep strains respectively. The values of the constant $\bar{\alpha}$ were all positive.

Table 2. Creep of Geotextiles

Fabric	Creep Measured in 20 Hours			
	Load Level %	Creep %	Load Level %	Creep %
NW-1	40	3	57	4
NW-3	35	5	63	27
NW-5	33	9	57	31
NW-6	33	20	56	22
W-4	31	11	44	16
C-1	36	5	55	8

Table 3. Constants for Three-Parameter Creep Equation

Fabric	m	A %/minute	$\bar{\alpha}$
NW-1	0.62	0.01	3.59
NW-3	0.74	0.13	2.99
NW-5	0.61	0.06	4.67
NW-6	0.84	0.85	1.23
W-4	0.73	0.29	2.15
C-1	0.73	0.20	1.18

Table 4 gives the constants for creep of geotextiles from Rate Process Theory. The constant E represents the static modulus instantaneously after application of the stress. The magnitude of transient creep depends upon the constant E_p and the steady-state creep depends upon the constants K and α.

Table 4. Constants for Creep of Fabrics by Rate Process Theory

Fabric	K_p 1/second	α_p m/kN	E_p kN/m	K 1/second	α m/kN	E kN/m
NW-1	11×10^{-7}	0.74	173	5×10^{-8}	0.42	66
NW-3	8×10^{-7}	1.03	109	3×10^{-8}	0.60	116
NW-5	61×10^{-7}	0.02*	1.8*	10×10^{-8}	0.01*	0.6*
NW-6	5×10^{-7}	2.51	22	53×10^{-8}	0.23	21
W-4	15×10^{-7}	0.40	198	29×10^{-8}	0.11	182
C-1	18×10^{-7}	0.34	240	35×10^{-8}	0.08	163

*Normalized to gm/m^2.

Figures 5 to 10 show comparisons of the experimental creep curves and those determined by the three-parameter creep equation and by the Rate Process Theory. The creep curves determined by the Rate Process Theory appear to be more consistent with the experimental curves.

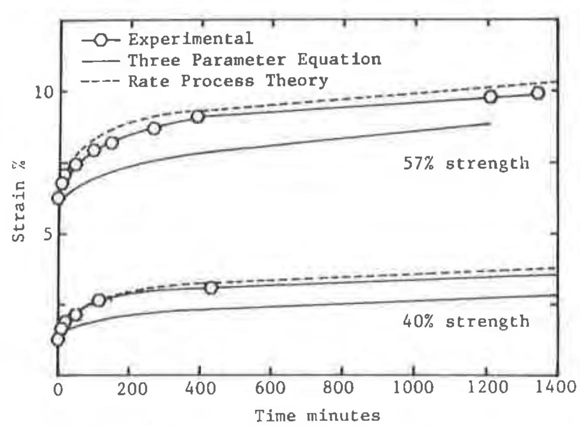

FIGURE 5 : Creep strain vs. time for NW-1 fabric

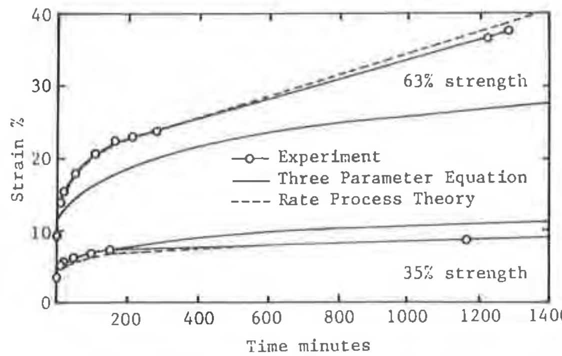

FIGURE 6: Creep strain vs. time for NW-3 fabric

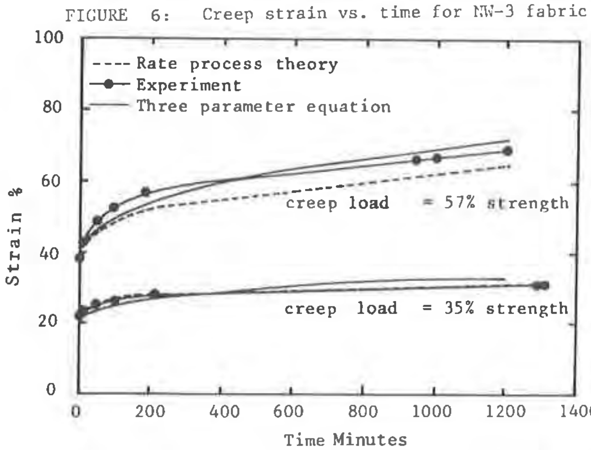

Figure 7: Creep strain vs time for a Fabric NW-5.

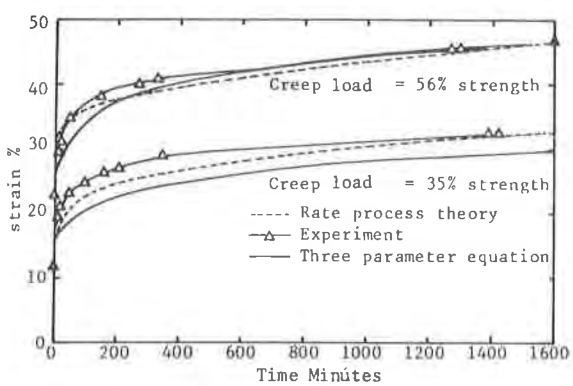

Figure 8: Creep strain vs time for Fabric NW-6.

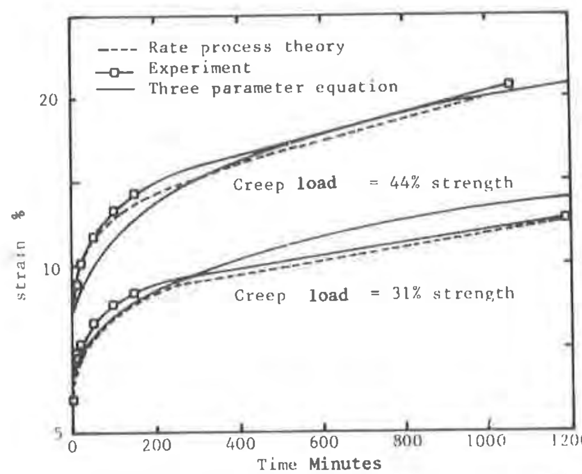

Figure 9: Creep strain vs time for Fabric W-4.

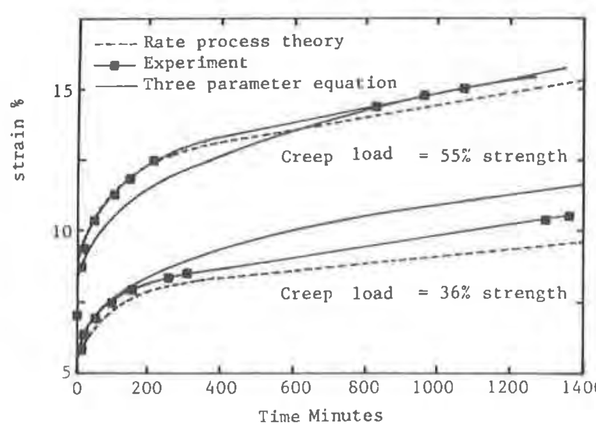

Figure 10: Creep strain vs time for Fabric C-1.

ANALYSIS OF RESULTS

The basic difference between the three-parameter creep equation and the creep equation derived from Rate Process Theory is that in the former the strain rate is considered as continuously decreasing with passage of time; and in the latter the creep rate is considered as continuously decreasing during the transient stage until a minimum value is reached, and then remains constant at this minimum value during the secondary stage until the beginning of the tertiary creep. During tertiary creep, strain rate increases very rapidly and failure occurs by rupture.

On the basis of the creep parameters presented in Tables 3 and 4, lengths of time at which the geotextile would creep to given strains can be calculated. Table 5 shows a comparison of the time estimated from the three-parameter creep equation and the Rate Process Theory for the geotextiles to creep to failure strains equal to those determined from wide strip tensile tests of the geotextile using specimens 200 mm (8 inches) wide and 100 mm (4 inches) initial gauge length.

Table 5. Estimated Time to Reach Ultimate Strain+ in a Creep Test

Fabric	ε_1* %	Ultimate+ Strain %	Load Level %	Time Required to Reach Ultimate Strain: Three-Parameter Equation	Time Required to Reach Ultimate Strain: Rate Process Theory
NW-1	4.0	23.0	40	485 days	22 days
NW-3	3.5	53	35	673 days	21 days
NW-5	23.0	98	37	107 days	19 days
NW-6	12.0	60	33	142 days	15 days
W-4	7.0	28	31	6 days	3 days
C-1	5.0	15	36	4 days	4 days

*ε_1 strain at time t = 1 minute

+ultimate strain measured by wide strip tensile test.

For the nonwoven geotextiles, the three-parameter creep equation predicted much longer times (142 to 485 days) than those predicted from the Rate Process Theory (15 to 22 days) for the failure strains to be reached by creep under sustained static loads. However, for the woven geotextiles both methods predicted creep times of the same order of magnitude.

The predicted time to reach failure strains under sustained loads are much shorter than the normal design lives of geotextile uses. There have been instances where geotextiles are known to resist static loads in the field without significant creep. For example, the first geotextile reinforced earth retaining wall designed by Bell (12) is performing satisfactorily without evidence of significant creep nearly a decade after its construction.

For both methods of creep characterization, determination of creep rate as a function of time is needed in order to develop the parameters. The creep strain rate was determined at any given time by measuring the slope of tangent to the experimental creep-time curve. Therefore, that the parameters determined by the Rate Process Theory and by the three-parameter creep equation are dependent upon the shape of the creep curve could introduce significant errors in the values of the creep constants. The results given in Tables 3 and 4 can best be considered as qualitative because of the limitation in the recording device used in the experiments. It was not possible to record the creep curve continuously for the full length of the experiment and the shape of the experimental creep curve was extrapolated after the first few hours to the observed creep strains at the end of the test (see Figures 5-10).

More important, however, may be the differences between the in-situ field conditions and the laboratory conditions. In the field the geotextile is confined by the soil while it is free in the laboratory strip tensile test. Also, field temperatures are usually much lower than the temperatures in the laboratory.

CONCLUSIONS

Creep-time curves predicted by the four-element rheological model based on Rate Process Theory appear to be more consistent with the experimental curves than those predicted by the three-parameter equation based on the empirical method.

Time to reach failure strains under sustained load predicted by the empirical method was much longer than the time predicted by the method based on Rate Process Theory for nonwoven geotextiles. However, for woven geotextiles, both methods predicted time to failure strains of the same order of magnitude.

Both methods predicted much shorter lives of geotextiles under sustained load than the normal design life of geotextile uses and shorter than indicated by actual field experience. This inconsistency needs further study with longer duration, more accurate tests which also consider temperature effects and simulate soil confinement.

ACKNOWLEDGEMENT

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Tensile Strength and Creep Behavior of Geotextiles in Cold Regions Applications

Résistance à la traction et comportement au fluage des géotextiles dans les régions froides

In recognition of the need to determine mechanical properties of geotextiles used in cold region applications, a research program was conducted to 1) investigate the influence of freeze-thaw cycles in a freshwater and saline water environment on the load-strain-strength characteristics of geotextiles, and 2) evaluate the load-strain-strength and creep characteristics of geotextiles at subfreezing temperatures. Five geotextiles, each with different construction and/or material characteristics were employed in the program. A wide strip tensile test was used to characterize the load-strain-strength behavior of the geotextiles. Creep characteristics were evaluated at various percentages of ultimate tensile strength. Based on the results presented it is concluded that the load-strain-strength and creep characteristics are not adversely affected by subfreezing temperatures in a temperature range associated with many cold region engineering applications. Freeze/thaw cycling in a dry, distilled water, or saline water environment has no appreciable influence on the load-strain-strength characteristics of the geotextiles tested.

Afin de déterminer les propriétés méchaniques des géotextilés utilisés dans les régions froides, on a établi un programme de recherche pour 1) étudier dans un milieu d'eau douce et d'eau saline, l'influence des cycles gel-dégel sur les caractéristiques charge-déformation-résistance des géotextiles, et 2) évaluer les caractéristiques charge-déformation-résistance et le comportement au fluage des géotextiles à des températures sous 0°=C. Cinq géotextiles ont été étudiés au cours du programme. Un essai de traction sur éprouvette large a été utilisé pour caracteriser le comportement charge-déformation-résistance. Le fluage a été évalué pour differents pourcentages de la résistance limite à la traction. Sur l'observation de ces resultats on peut conclure que les caractéristiques charge-déformation-résistance et le comportement au fluage sont peu affectés par des températures sous 0°=C. Les cycles gel-dégel dans un milieu sec, d'eau distilleé ou d'eau de mer ont peu d'influence sur les caractéristiques charge-déformation-résistance des géotextiles utilisés.

INTRODUCTION

In the past decade considerable attention has been focused on the cold regions of the world owing to the abundance of natural resources which exist there, in particular oil and gas reserves. Engineering/construction practices for building foundations, roadways, and embankments in cold regions of the world are intimately associated with freezing related phenomena of initially unfrozen ground, and thawing related phenomena of initially frozen ground. Freezing of initially unfrozen frost-susceptible soil can result in heave at the ground surface and disruptions of embankments or foundations placed on or wholly within the zone of freezing. Thawing of ice-rich soils results in loss of bearing strength and settlement (termed thaw instability).

Recently, engineers concerned with arctic and subarctic problems have incorporated geotextiles in their design/construction recommendations for embankments and foundations (1). The geotextiles have been used primarily as filter and separation and/or reinforcing layers. Further, it has been suggested by engineers that the geotextiles might act as a capillary transmission barrier when placed in frost-susceptible soils (2,3).

While successful applications of geotextiles in cold regions have been made, many questions remain unanswered relative to the performance of geotextiles in a cold environment. For example, the influence of freeze-thaw cycles or subfreezing temperature on load-strain-strength and creep characteristics of geotextiles is largely unknown. Also little is known about the ability of geotextiles to prevent thaw instability.

In recognition of the need to determine mechanical properties of geotextiles used in cold regions, a research program was conducted to investigate the influence of freeze-thaw cycles in a freshwater and saline water environment on the load-strain-strength characteristics of geotextiles, and evaluate the load-strain-strength and creep characteristics of geotextiles at subfreezing temperatures. The results from the research program are presented herein.

GEOTEXTILES TESTED

Five geotextiles, each with different construction and/or material characteristics, were employed in the research program. These geotextiles are described in Table 1.

Table 1. Geotextiles Selected for Research Program

Geotextile	Filament	Geotextile Construction	Nominal Weight gm/m^2 (oz/yd^2)
Bidim C-34	Polyester Continuous	Nonwoven Needlepunched	272 (8)
Stabilenka T-100	Polyester Continuous	Nonwoven Resin Bonded	100 (3)
Typar 3401	Polypropylene Continuous	Nonwoven Heat Bonded	136 (4)
Fibretex 300	Polypropylene Continuous	Nonwoven Needlepunched	300 (8.8)
Propex 2002	Polypropylene Slit Film	Woven	150 (4.4)

The geotextile types were selected to insure that the load-strain-strength characteristics associated with several geotextile constructions and fiber polymer compositions could be compared in a meaningful way.

TEST PROCEDURES AND EQUIPMENT

Tensile Tests

A strip tensile test was used to characterize the load-strain-strength behavior of the geotextiles. A 200 mm (8.0 in.) sample width was employed to insure the results obtained would simulate, to as great a degree as practical, the plane-strain loading conditions which exist in the field (4). Five test specimens of a given geotextile type for a given test condition were trimmed to 200 mm (8.0 in.) in width by 220 mm (8.5 in.) in length with an accuracy of ± 4 mm (± 0.15 in.) The weight of each test specimen was recorded. The specimens were secured in 230 mm (9.0 in.) wide tensile test grips at a grip spacing of 100 mm (4.0 in.). The grips were placed in the load frame and a tensile test was conducted at a rate of strain of 10%/min. A pretension strain of 0.5 to 1.0% was applied to the specimen prior to the conduct of the test.

Both MTS and Instron test systems were employed in the program. The Instron was used for specimens which exhibited high elongation at failure. MTS test systems were used to test all the other geotextiles. Load and displacement were measured with linear variable differential transformers and load cells during the conduct of the test.

Each geotextile type was tested under the following conditions:

(1) room temperature [22°C (71°F)] in a dry state

(2) subfreezing temperature [-12°C (10°F)] in a dry state

(3) room temperature in a saturated surface dry (wet) state

(4) room temperature [22°C (71°F)] in a saturated surface dry (wet) state following 50 freeze/thaw cycles in a dry state

(5) room temperature [22°C (71°F)] in a saturated surface dry (wet) state following 50 freeze/thaw cycles in distilled water

(6) room temperature [22°C (71°F)] in a saturated surface dry (wet) state following 50 freeze/thaw cycles in saline water

The freezing temperature condition was achieved by placing the geotextiles in a walk-in cold room for 24 hours, and testing in the cold room. The saturated surface dry (wet) condition was achieved by soaking the geotextiles in water for 24 hours and toweling the specimens just prior to testing. Freeze/thaw cycling for either distilled or saline water was achieved by placing a specimen in a sealed plastic bag filled with either distilled or saline water and placing the bag on a rack in a freeze/thaw chamber. Thermistors were placed inside select specimen bags to monitor actual bag temperatures and insure complete freezing and thawing.

The tensile strength of each specimen was normalized to a nominal mass per unit area to account for sample variability. The normalized strength, S_N, is given by:

$$S_N = (M_n/M)S \qquad (1)$$

in which, M_n = nominal mass per unit area of all specimens of a specific geotextile type
M = mass per unit area of the specimen
S = ultimate geotextile specimen strength

The normalized strengths of five specimens of a given geotextile type for a given test condition were averaged. The significance of the average values when compared with the average normalized strength of other test cases was determined using a student's T-distribution assuming a 90% confidence level.

Creep Tests

The creep characteristics of the geotextiles considered were evaluated at room temperature [22°C (71°F)] and subfreezing temperature [-12°C (10°F)] when loaded to various percentages of ultimate wide strip tensile strength. Three test specimens of a given geotextile type for a given test condition were trimmed to dimensions of 152 mm (6.0 in.) by 305 mm (12.0 in.). The samples were secured in grips with a grip spacing of 76 mm (3.0 in.). The loads were applied over a 152 mm (6.0 in.) specimen width. The three test specimens, secured in their grips, were connected in series for dead weight loading. Staples were spaced vertically at 64 mm (2.5 in.) in the center of the specimen to serve as an initial length for the measurement of deformation with time.

Tensile tests were conducted for each geotextile type to determine an average ultimate strength. The procedures described in the preceding section were employed except that the specimen size was 190 mm (7.5 in.) by 150 mm (6.0 in.) and a 76 mm (3.0 in.) grip spacing was used. Five specimens of each geotextile type were tested (for dry strength only). After the specimens were tested, the ulitmate strength obtained for each specimen was normalized to a nominal weight per unit area (Eq. (1)). The normalized strengths of the specimens for a geotextile type were averaged. The average normalized strength was used to determine the load required for each set of three creep samples, as shown below:

$$\text{Load} = P_u(M_a/M_n)N_{ave} \qquad (2)$$

in which, P_u = percent of ultimate strength
M_a = average mass per unit area of the three specimens hung in series
M_n = nominal mass per unit area of all specimens of a specific geotextile type
N_{ave} = average normalized ultimate strength for a geotextile type

Each geotextile type was tested at four different load levels, specifically, 20, 35, 50, and 65% of ultimate 152 mm (6.0 in.) wide strip tensile strength. The loads associated with a given test condition were applied instantaneously to the three test specimens in series and deformation readings were taken at 1, 2, 5, 10, 30, 60, 120, 240, 1440, and 2880 min. Thereafter, readings were taken every week up to approximately 15 weeks or until failure occurred. A test was also terminated when no measurable deformation occurred over a period of one week. The subfreezing temperature condition was achieved by conducting the creep tests in a walk-in cold room.

TEST PROGRAM RESULTS

Tensile Load-Strain-Strength

Tensile axial load versus strain relationships for the geotextiles tested at temperatures of 22 and -12°C (71 and 10°F) are shown in Figure 1. The results indicate that for the geotextiles tested the needlepunched geotextiles have high elongation and intermediate strengths. The woven geotextile has the highest strength but the lowest elongation. The heat bonded geotextiles have strength and elongation characteristics intermediate to those of the needlepunched and woven geotextiles. The modulus and strength of the heat bonded geotextiles increased with decreasing temperature. The elongation at

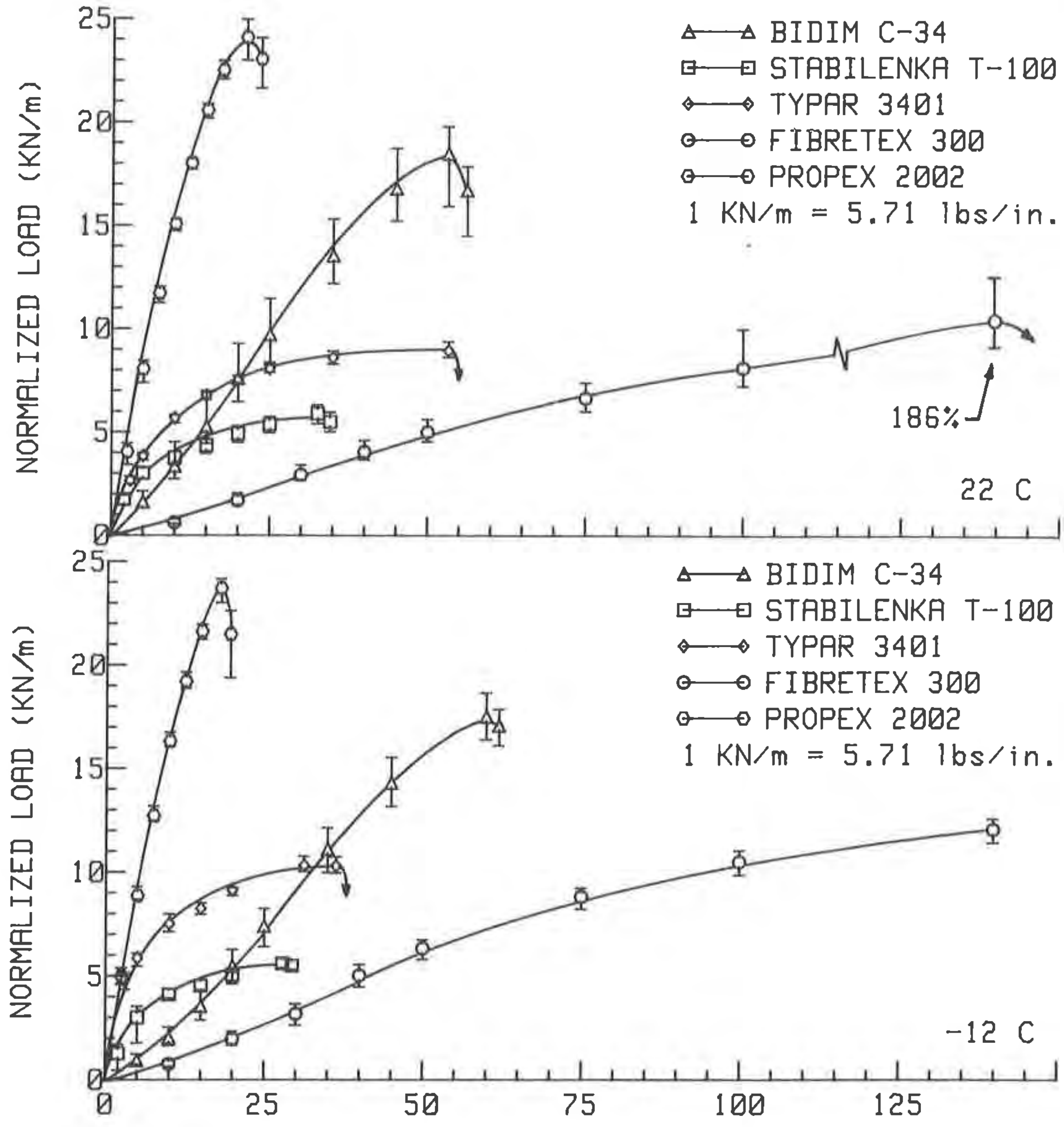

FIGURE 1: AXIAL LOAD VERSUS STRAIN (DRY CONDITION).

failure decreased significantly for the heat bonded geotextiles and all polypropylene geotextiles. No other statistically significant (90% probability) deviations in load-strain-strength characteristics were observed. The effects of geotextile construction appears to be much greater than the effect of temperature on the geotextile for small strains.

A summary of all the tensile load-strain characteristics of the geotextiles tested is given in Table 2. The elongation at failure for the heat bonded polyester geotextile increased upon wetting. The reasons for this are not clear. All other results indicate that normalized strength, percent elongation at peak strength, and normalized secant modulus at 10% strain do not change appreciably when the geotextiles considered were tested in a dry or wet condition at room temperature or following 50 freeze/thaw cycles in a dry, distilled water, or saline water environment.

Creep Strain

Typical creep strain versus time relationships for the geotextiles tested at 22° and -12°C (71° and 10°F) at a

Table 2. Load-Strain Characteristics of Geotextiles

Geotextile	Property	I	II	III	IV	V	VI
Bidim C-34	Normalized Strength (kN/m)*	18.6	17.5	16.0	16.1	15.3	16.7
	% Elongation at Peak Strength	53.2	60.0	59.9	54.8	59.0	54.5
	Normalized Secant Modulus at 10% strain (kN/m)	33.5	20.3	25.2	28.4	21.9	33.6
Stabilenka T-100	Normalized Strength (kN/m)	5.9	5.6	5.3	5.7	5.8	5.4
	% Elongation at Peak Strength	32.6	27.9	44.0	43.8	44.0	43.9
	Normalized Secant Modulus at 10% strain (kN/m)	37.7	41.0	30.8	33.8	33.1	31.2
Typar 3401	Normalized Strength (kN/m)	8.9	10.3	8.8	9.1	8.9	8.5
	% Elongation at Peak Strength	53.3	31.3	43.8	48.2	47.8	36.7
	Normalized Secant Modulus at 10% strain (kN/m)	56.9	75.2	61.9	62.2	60.4	59.2
Fibretex 300	Normalized Strength (kN/m)	10.3	-	9.9	9.9	9.7	9.6
	% Elongation at Peak Strength	186	-	166	187	187	181
	Normalized Secant Modulus at 10% strain (kN/m)	6.1	7.5	7.5	5.8	4.9	6.1
Propex 2002	Normalized Strength (kN/m)	24.2	23.7	24.4	25.4	24.7	24.2
	% Elongation at Peak Strength	21.2	18.0	19.8	20.4	21.1	21.1
	Normalized Secant Modulus at 10% strain (kN/m)	151	163	162	162	151	148

I = Control, Dry Condition, 22°C (71°F)
II = Control, Dry Condition, -12°C (10°F)
III = Control, Wet Condition, 22°C (71°F)
IV = 50 Cycles Freeze/Thaw, Dry Environment, Wet Condition
V = 50 Cycles, Freeze/Thaw, Distilled Water Environment, Wet Condition
VI = 50 Cycles, Freeze/Thaw, Saline Water Environment, Wet Condition

Note: Samples which were subjected to freezing and thawing were cycled between -15 and +15°C (5 and 58°F)

*1 lbs/in. = 0.175 kN/m

load level of 50% of the 152 mm (6.0 in.) wide strip tensile strength are shown in Figure 2. Sixty minute creep strain versus load level relationships are shown in Figure 3. As shown in Figure 2 a reduction in temperature resulted in a decrease in creep strains for geotextiles with polypropylene fibers. At this load level creep strains for geotextiles with polyester fibers were not significantly influenced by temperature. At load levels of 20% ultimate strength or below, however, the decrease in creep strain with temperature was not statistically significant for any geotextile. The influence of temperature on creep strains is apparently related to filaments and not to the geotextile construction. The results shown in Figure 3 indicate creep strains were greatest for geotextiles with polypropylene fibers. All of the polypropylene materials experienced tertiary creep and failed at load levels of 50% or 65% of wide strip tensile strength at 22°C (71°F). The geotextiles with polyester fibers did not fail until the load level was taken to 80% of wide strip tensile strength at 22°C (71°F). Significantly, however, none of the geotextiles failed at load levels as great as 65% at -12°C (10°F).

The geotextile construction process appears to dominate the magnitude of the short-term creep strain at any time. Needlepunched geotextiles have the greatest short-term creep strains at a given load level. Heat bonded and woven geotextiles have the lowest short-term creep strains.

CONCLUSIONS

Based upon the test results presented for the five geotextiles considered in the research program, the following conclusions have been reached: 1) the mechanical properties of geotextiles, in terms of load-strain-strength and creep characteristics, are not adversely affected by subfreezing temperatures in a temperature range associated with many cold regions engineering applications; 2) Freeze/thaw cycling in a dry, distilled water, or saline water environment has no appreciable influence on the load-strain-strength characteristics of geotextiles; 3) the geotextile construction dominates the short-term creep strain; 4) other factors being equal polyester geotextiles have the lowest creep rates and highest thresholds of tertiary creep; and 5) a reduction in temperature results in decreased creep rates for polypropylene geotextiles.

ACKNOWLEDGEMENTS

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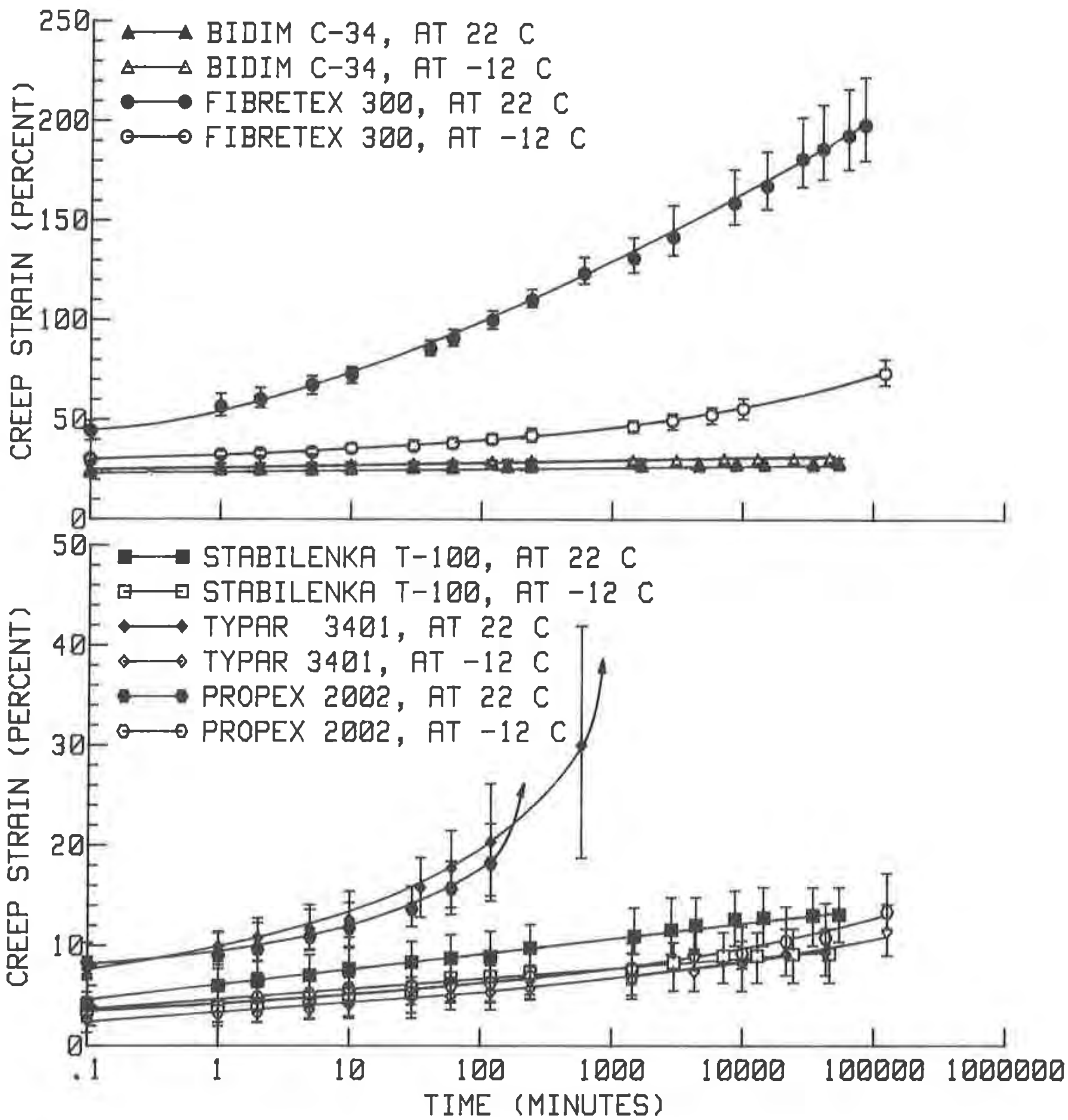

FIGURE 2: CREEP STRAIN VERSUS TIME AT 50% WIDE STRIP TENSILE STRENGTH.

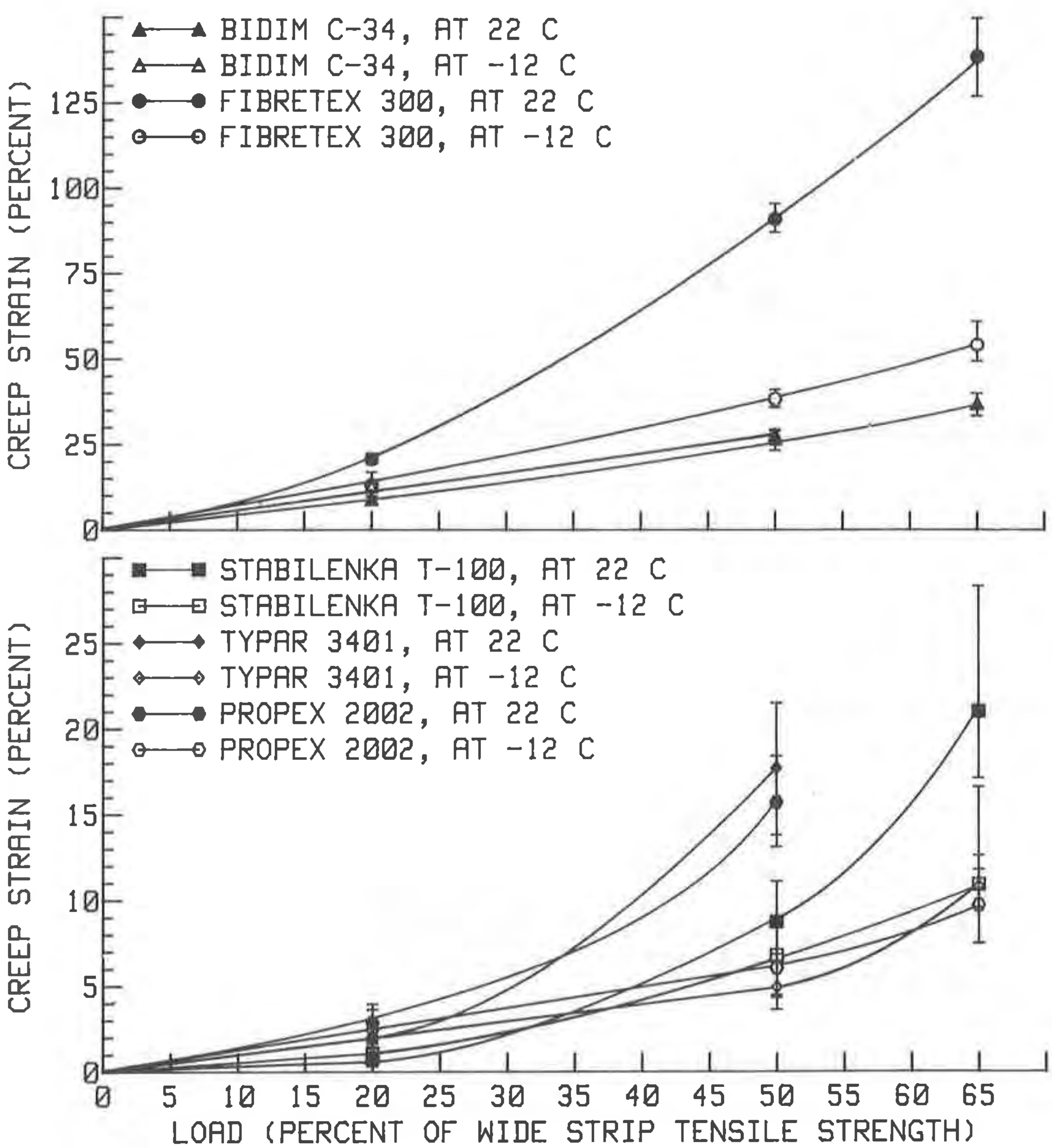

FIGURE 3: SIXTY MINUTE CREEP STRAIN VERSUS LOAD LEVEL.

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Resistance to Area Change as a Measure of Fabric Performance

La résistance des tissus au changement de superficiel comme mesure de performance

The paper is concerned with the resistance of fabrics to area change, a parameter with at least two important effects on the performance of geotextiles. When conformability is required, in order to fit a fabric to a three-dimensional configuration, this can only be accommodated by area change. On the other hand a loss of support occurs when an underlaid fabric grows in area.

The paper includes a discussion of the energy-area strain relations and the results of preliminary experiments conducted at UMIST on area changes in conventional fabrics.

Cet article traite le problem de la resistance des tissus au changement de superficie. Ce parametre influence le performance des Geotextiles au niveau de conformité quand un changement de superficie est avantageux, tandis q'une perte de support est inevitable quand un Geotextile sous - posé s'agrandi.

L'article examine les relation energie/surface/extension et présente les resultats des experiences faite a l'UMIST sur des tissus conventionnels.

INTRODUCTION

Textile engineering and civil engineering, which have come together in the new technology of geotextiles, both derive from ancient useful arts. But whereas civil engineering embraced the science of mechanics at any early date and became the senior branch of engineering, the production of textiles remained as a craft and is only now developing a sound basis of mechanical understanding. Although geotextiles have made rapid progress through empirical cooperation between civil engineers and textile manufacturers, with some use of design calculations modified from other contexts and different materials, further advances will require a proper treatment of the mechanics of the systems involved. This will need the tripartite work of applied mathematicians textile engineers and civil engineers, in the same way that bridge-building and other public works have required a combination of mechanics, metallurgy and materials science, and structural engineering.

The present paper is not a definitive account of a completed piece of research. It is an attempt to stimulate thought about the formulation of ways of characterising and analysing the mechanical behaviour of textile materials, so that the tripartite alliance can function productively. An analogy with soil mechanics may be helpful. At first, the civil engineer regarded soil as inert ballast, to be shovelled by navvies rather than studied seriously, but then it was recognised that the mechanical response of the soil was important in relation to the stability of structures; but it also became clear that the behaviour of granular solids was complicated and that scientific progress could only be made when the right modes of description and analysis had been found. As a result, the subject has developed as a specialised sub-branch of mechanics, and a modern text-book will start with an account of the necessary and specific mathematical formulation and go on to engineering design methods and calculations. Similarly, the fabrics used in geotextile applications cannot be treated solely as commercial products of the textile industry, or the concern of fibre chemists, but must be studied as mechanical systems.

Textile materials differ in almost every possible way from the idealisations introduced in the study of the mechanics of materials. They are inhomogeneous, lacking continuity; anisotropic; non-linear even at small strains; easily deformable, suffering large strains and displacements and often achieving success, not failure, through buckling; and, in the literal mathematical sense, complex mechanically, with deformations combining elasticity and viscosity in a reversible and irreversible time-dependent sequence, further complicated by plasticity and frictional slip. Except in the value of an early sketch of a full work, there is little to be gained by treatments which neglect these complications.

In order to reduce an impossible generality to manageable proportions, the simplifications to be looked for are:

(a) the separation of modes of deformation, so that tensile modes can often be neglected as showing no deformation, under low forces and flexural modes neglected, as showing no resistance, under high forces;

(b) the recognition of symmetries which limit the degree of anisotropy;

(c) the contrary assumptions of constant volume deformation of the fibres themselves, and zero resistance to the reduction of volume in the spaces between fibres.

But perhaps the greatest simplifications comes in the fundamental approach, by recognising that textile fabrics must be treated as two-dimensional forms in their own right, and not as a special case of a three-dimensional continuum. There can be no direct recourse to the mechanics of solids. The fabrics are structures with real finite element dimensions of the same order of size as the material thickness, so that structural analysis, which can be disregarded in elasticity theory because the structure is invisible at the molecular level, becomes significant. The most obvious example of this is that bending of a woven fabric does not occur in parallel layers running from extension through a neutral plane to compression, since the fibres and yarns are interlacing from one side of the fabric to the other.

Nevertheless, except when dealing with high-speed dynamic effects where the wavelength becomes comparable to or less than the structural dimensions, a textile fabric can be treated as a two-dimensional continuum. This must be the starting-point for a rational mechanics of geotextiles.

THE MECHANICS OF SHEET MATERIALS

The definitions

The basic treatment of the mechanics of a two-dimensional continuum has been discussed by Shanahan, Lloyd and Hearle (1) and further commented on by Lloyd (2a). A sheet can suffer six independent modes of deformation, which can be superposed by simple addition to give any more complicated form of deformation at a point in the sheet. These modes of deformation are illustrated in Figure 1. In the plane there are two orthogonal tensile modes and a shear mode. Out of the plane, there are two orthogonal bending modes and a twisting mode. In the most general anisotropic material (or in an isotropic material) the choice of axis is immaterial from the point of view of mechanics, but, if there is some structural symmetry, as for example in an orthogonally woven fabric, then simplifications result when the mechanical axes are chosen to coincide with the structural axes.

With six forms of "strain", and correspondingly six "stresses", there is a formal similarity with the three-dimensional system, with its three tensile modes and three shear modes. The six elements of the stress matrix will relate to the six elements of the strain matrix through the modulus or compliance matrices, which, because of their symmetry, reduce from 36 elements to 21 independent elastic constants. Examples of the reduction in the number of independent constants through symmetry are given in the Table 1.

Table 1 Influence of symmetry

Material	Number of independent constants
No symmetry	21
Orthogonal (rectangular, i.e. different x and y)	13
Orthogonal (square, i.e. equal x and y)	6
in plain isotropic	4
isotropic solid	2

In a more general sense, the constants must be regarded as symbols for non-linear functional relationships, probably most conveniently dealt with by recording the strain energy as a general function of six modes of deformation (plus time and hysteresis).

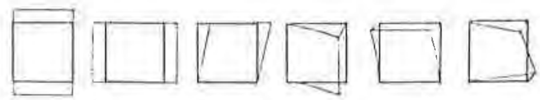

Figure 1. The six independent modes of deformation of sheet material.

Interaction of in-plane and out-of-plane effects

The analogy with the three dimensional case breaks down in one important respect. It is not possible to continue all forms of uniform deformation over finite areas. There is no problem for purely planar deformation, and bending about one axis causes no mathematical problem but does give rise to the physical problem of multiple occupation of space when the material bends back on itself. But twisting of a finite area necessarily involves length changes in the plane; and bending in two directions simultaneously must be accompanied by area changes in the plane.

AREA CHANGE AS A MODE OF DEFORMATION

The analogy with bulk modulus

Giroud (2b) points out that in most geotextile applications the forces are large in comparison with bending resistance, so that it is only necessary to consider resistance to deformation in the plane of the fabric. This simplification will be adopted for the remainder of this paper.

In dealing with bulk materials, although a variety of elastic constants may be used in different circumstances, it is recognised that deformation may be divided into the two categories of dilatational strain and shear strain, giving respectively changes in volume and shape. For isotropic elasticity, a bulk modulus and a shear modulus characterise the material completely. In some applications, shear is dominant and volume changes do not occur; in other applications volume changes are important and the shape alters easily to accommodate these.

In a two-dimensional sheet there will also be shear strain and dilatational strain, with the latter corresponding to area change. The concept of area modulus is not as easy to grasp as that of bulk modulus, because there is no simple experiment corresponding to the change of volume of a solid subject to a hydrostatic pressure, as in figure 2(a). Nevertheless the concept is mechanically identical, as shown in figure 2(b) and (c) with an area change resulting from an equal tensile stress in all directions.

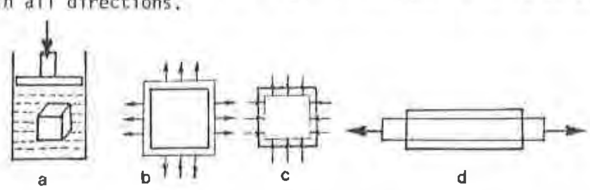

Figure 2 (a) The simple means of measuring volume change
(b) Uniform area extension.
(c) Uniform area compression.
(d) Area change in a solid sheet by change of shape.

In an anisotropic material, the change of volume or area will not occur by means of equal tensile strains in all directions and zero shear, but will involve a change of shape under hydrostatic loading.

Two confusing features

Appreciation of the concept of area change may be blocked by two difficulties.

Firstly in a solid continuum, like a rubber sheet, the area change is achieved by change of shape at substantially constant volume, as shown in figure 2d, and so is related to the shear modulus of the material. However, this does not apply to a structured fabric where in-plane behaviour is not directly linked to thickness change.

Secondly, in the simplest model of a fabric, namely the square pin-jointed assembly of figure 3(a), area increase is strongly resisted, since it can only occur through increase in length of the rods, but area decrease is easy through shearing as in figure 3(b). There is not a discontinuity at the origin, since the area change dA/A associated with shear θ is given by:

$$dA/A = 1 - \cos\theta \approx -\tfrac{1}{2}\theta^2 \qquad (1)$$

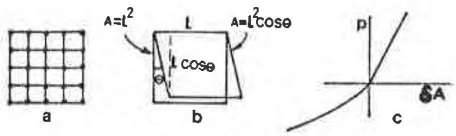

Figure 3. (a) A pin-jointed model of a fabric.
(b) Shear deformation.
(c) Relation between uniform stress p and area change δA.

Shear in either direction leads to area reductions as a second-order quantity, but a quantity which plays an important role in circumstances where second-order effects might be expected to be negligible. The plot of area change against hydrostatic stress will be of the form illustrated in figure 4(c). The initial area modulus is an inadequate description: the reduction in modulus with increasing strain must be considered. We may note, in passing, that figure 4(c) could also represent the pressure-volume relation of a similarly structured solid.

We must also note that the typical statement "shear strains do not cause a change of volume" (3) found in a standard text-book is not necessarily true in a structured material at large strains. In woven fabrics, shear causes area reduction, and the cross-term, the analogue of Poisson's ratio, is important at large strains. Nevertheless although the application of a shear stress may be the easiest way of causing area change, the direct relation shown in figure 3(c) is a simpler way of comparing materials with different forms of anisotropy in terms of the ease with which they can accommodate area change.

Area change is a significant response in geotextile applications in several different ways, which can be recognised as likely to occur in the sixteen applications listed by Giroud (2b).

The significance of area change in geotextile support

Whenever there is any element of support involved, as indicated schematically in figure 5, deformation involves an increase in area of the fabric. If one makes a rough approximation and assumes spherical symmetry and a vertical displacement h spread over an area of radius R, it can be shown that:

$$\text{area strain} = \varepsilon_A = \delta A/A = h^2/R^2, \qquad (2)$$

$$\text{energy of fabric deformation} = \pi R^2 U(h^2/R^2) \qquad (3)$$

where the function $U(\varepsilon_A)$ is the area strain energy per unit area, potential energy of loading = -Fh,

where F is the weight applied.

Equilibrium will occur at the minimum value of the total energy $\pi R^2\, U(\varepsilon_A) - Fh$.

In particular instances, there may be special geometrical constraints to be included. Furthermore there may be other energy terms to be included in the sum such as those due to compression of the underlying substrate and to frictional resistance to relative movement between substrate and fabric. Whereas, in the simple situation, the minimum energy would occur with a maximum value of R, since this minimises the strain, the other terms may serve to limit R. However, such details go beyond the scope of this paper, which is only concerned to demonstrate the significance of resistance to area change as a fabric property. What is clear is that a knowledge of values of $U(\varepsilon_A)$ would be of greater value to the design engineer, than other information on fabric mechanical properties. In many instances, this knowledge would enable any problem to be solved to an acceptable accuracy, although one must recognise that in some circumstances, such as those illustrated in Figure 6, the required area change could only occur when the necessary changes in shape are not prevented by the constraints on the system.

The above discussion relates to displacement under load, which may be of direct relevance, particularly when it leads to the development of corrugations. However, the analysis could be carried further into situations where failure occurs when the area change becomes so large that the fabric bursts. There will also be energy absorption associated with the change; this may be a nuisance, increasing the power consumption of vehicles or even slowing them to a halt, or it may be an asset as in the absorber function listed by Giroud (2b).

Area change and conformability

Any thin sheet material will have low bending stiffness and be flexible. But, as discussed in a preliminary account of current research (2c), double curvature into rounded surfaces requires an associated area change. If this area change is resisted, then any attempt to bend simultaneously in more than one direction will force a material into sharp point discontinuities. This happens with paper, but not with woven or knitted fabrics. Nonwoven fabrics occupy an intermediate position.

Some measure of conformability is required in geotextiles, in order to give ease of handling, to allow the fabric to mould itself on rough surfaces or to fit round corners, and to avoid the damage which can occur at sharp points. In order to achieve this desirable behaviour, a mode of easy area change is required. It might seem that this is in direct conflict with the requirements for resistance to area change in order to give support, but a suitable compromise may result from differences in response in tension and compression, or at high and low strains: a high area modulus is needed under high tensile stress, but not necessarily in compression or under low tensile stress.

The interaction with permeability

Associated with area change, there will be a change in the space within a fabric, and thus in its permeability. Consequently there will be significant interactions between area change and the various hydraulic functions of geotextiles.

MEASUREMENT OF FABRIC PROPERTIES

The problem

Although the concept of area change is theoretically as simple as that of volume change in a solid, the practical problem is that it is difficult to apply a uniform biaxial tension as in figure 2(b), because any grip needed to impose extension in one direction impedes extension in the other direction, and it is impossible to apply compression as in figure 2(c) without the specimen buckling. There is thus no direct way of determining the response shown in figure 3(c).

It is necessary to use other test methods and then calculate the area response functions. However the eleven tensile test methods shown by Giroud (2b), and others, involve a variety of combined strains and need to be analysed carefully.

Linear, elastic behaviour

In a linear elastic material at small strains, the three independent in-plane directions lead, in the most anisotropic material, to six independent elastic constants, and the area modulus will be a function of these parameters. In practice, for orthogonal materials it may be easiest to measure tensile moduli and Poisson's ratios in three directions and then calculate the required quantities. Alternatively shear moduli can be found.

A full account of the theory will be published elsewhere, but some useful results are given here. For linear elastic behaviour, the energy of fabric deformation and the area strain are related by:

$$U = \tfrac{1}{2}\ \text{Vol. Am. } \varepsilon_A^{\ 2} \qquad (5)$$

where Am is defined as the effective area modulus which is a function of loading ratio along two normal directions (w) and the elastic constants of the fabric.

The values of Am for the state of hydrostatic stress (n=1, corresponding to the minimum energy per unit area strain) and the case of uniform circumferential strain $\left(n = \frac{E_1(1+\mu_2)}{E_2(1+\mu_1)}\right)$ can be shown to be:

Hydrostatic stress
$$Am = \frac{E_1E_2}{E_1(1-\mu_1)+E_2(1-\mu_2)} \qquad (6)$$

Uniform strain
$$Am = \frac{E_1(1+\mu_2)+E_2(1+\mu_1)}{4(1-\mu_1\mu_2)} \qquad (7)$$

where E_s and μ_s have their usual engineering meaning.

With regard to the denominator of equation 7, it should be emphasized that the product of the Poisson's ratios can not exceed unity and the modulus never assumes a negative value.

Non-linear and inelastic behaviour

When the material response is non-linear, there can be no simple relations of the type given in the last section. With three principal strains, $\varepsilon_1,\varepsilon_2,\varepsilon_3$, one has:

$$\text{area strain} = \varepsilon = f_1(\varepsilon_1,\varepsilon_2,\varepsilon_3) \qquad (8)$$

Equation (8) can be derived explicitly from the strain geometry, but its form depends on the particular strain definitions adopted.

$$\text{specific energy of deformation} = U = f_2(\varepsilon_1,\varepsilon_2,\varepsilon_3) \qquad (9)$$

Equation (9) must be determined experimentally, or calculated theoretically from structural mechanics, since it represents the material properties.

Any chosen value of ε_A will define a set of values of $(\varepsilon_1,\varepsilon_2,\varepsilon_3)$ and it is necessary to find which combination gives the minimum value of U. This minimum would be the value of $U(\varepsilon_A)$, which could be used directly in equations such as (3). Alternatively, we could plot the stress-strain curve by using the relation:

$$\text{effective hydrostatic stress} = dU(\varepsilon_A)/d\varepsilon_A \qquad (10)$$

In practice instead of being forced to carry out the complete search to find the precise minimum, it may be better to study the relations between area change and deformation energy in various particular directions. In addition, the multi-valued character of the functions must be examined when hysteresis or time-dependence are material properties.

Experimental methods

Errors due to edge effects, to non-uniform deformation, and to fabric buckling are the bane of textile testing, and it is not possible to mention all the problems in this short paper. Only a broad outline of methods can be given.

Simple uniaxial extension, carried out on a standard laboratory tensile tester, can be adopted to study area change if the lateral contraction is measured. It is also preferable to measure the axial strain in the central region of uniform deformation. A satisfactory but time-consuming method, which we have adopted, is to draw a square on the fabrics and then photograph this at successive known stages of the tensile test. By cutting samples at different angles and stretching in the bias direction it is possible to determine the area changes associated with shear as well as those due to extension along the principal axes of the fabric.

One special experimental trick should be noted. If a single piece of fabric is extended on the bias it exerts a sideways pull on the grips. This can be avoided by mounting together two samples facing in opposite directions.

Further development along this approach to the problem would involve the direct study of fabric shear (with specimens cut in different directions), the construction of a suitable biaxial tester, and, most important, the search for a convenient automatic method of monitoring the strains.

An alternative approach to the problem is to go directly for some experimental arrangement akin to figure 4. A laboratory bursting tester would be one means. A penetration tester with a plunger pressed on to a clamped fabric specimen is another which we have used in other studies (4). In current work a ball penetration tester is being used.

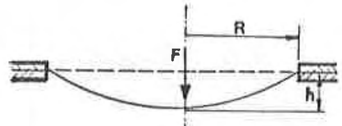

Figure 4. Support displacement due to area change under load.

Fabrics were clamped between two rigid horizontal plates with corresponding circular openings and a symmetrically positioned ball was pressed against the fabric. A curve giving the load corresponding to any given depth of deformation (δ) can be obtained from the CRE machine.

The curve can be used to determine the energy for any given value of δ which in turn is used to calculate the area strain from the geometry.

By selecting a suitable ball size, too large to penetrate the fabric but sufficiently small to deform the fabric into conical rather than a spherical shape, the fabric deforms under uniform circumferential strain corresponding to equation 7. The area modulus thus obtained can be compared with the results of the simple tensile tests along different directions.

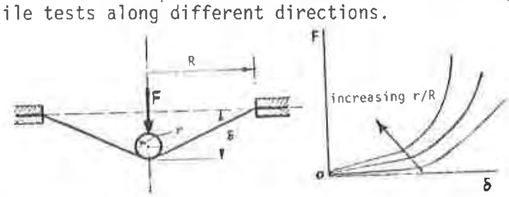

Fig.5(a) Schematic experimental set up.
(b) Typical load-deformation curves.

Experimental results

We are not in a position to give comprehensive data on the range of fabrics of interest in geotextiles. Indeed, since the research is being carried on in a broader context of fabric mechanics related to more traditional uses of textiles, the particular fabrics so far studied are not likely candidates for engineering use. Nevertheless some experimental results may serve to indicate general trends.

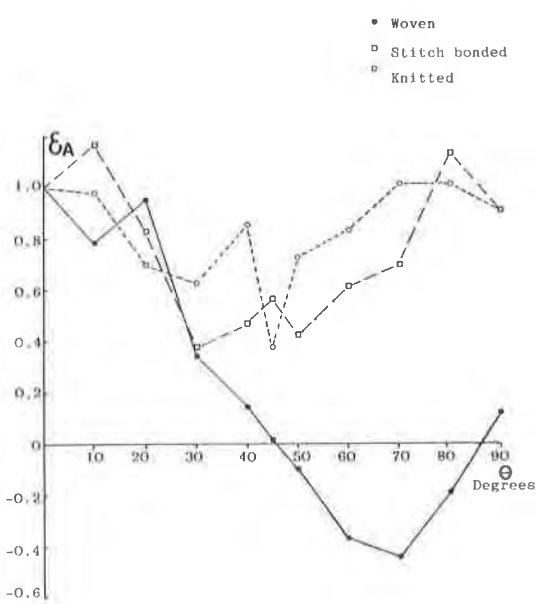

Fig.6 Area strain per unit tensile strain along different directions.

The experimental results given in figures 6 and 7 are obtained from uniaxial tests using 40 x 40 cm samples. The strains were measured by photographic means and the corresponding energies calculated from the CPE machine charts. The values given are relative to the corresponding values along direction θ=0 (warp direction, machine direction and wale direction)

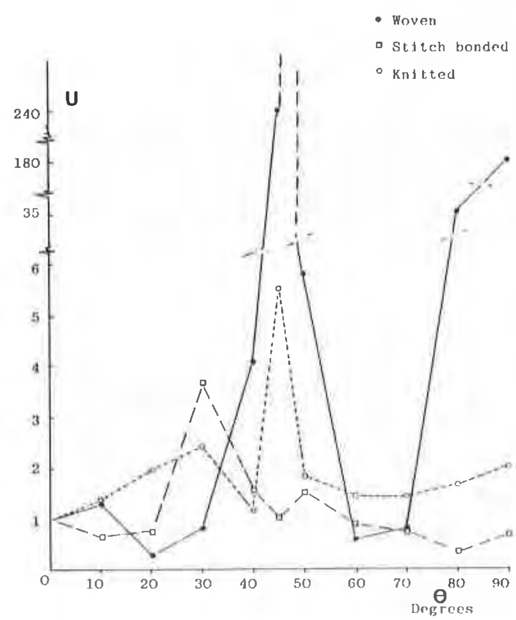

Fig.7. Energy of fabric deformation per unit area strain along different directions.

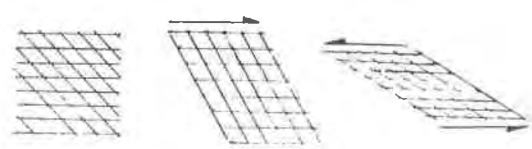

Fig. 8. An oblique lattice which can change area by shear.

CONCLUSION

The anelastic properties of fibre materials, the slippage between fibres in assemblies, and the anisotropic structure of fabrics makes the complete specification of their mechanical properties a formidable task. It is easy to take refuge in tests which are simple to perform, but often these have little direct relevance to behaviour in geotextile uses, although they may be of some value as broad indication of "strength" and are always useful as a means of monitoring the maintenance of quality. Despite the difficulties, a satisfactory treatment of the fabric mechanics is a prerequisite for real geotextile engineering.

The underlying form of the general relations, discussed in the first part of this paper, is a necessary starting point in the search for simplification. We have concentrated attention on resistance to area change which can be seen as an important mechanism in any support function of geotextiles, while conversely ease of area deformation is needed to ensure conformability. In the best materials non-linearity of response will give a good combination of properties.

The experimental results quoted illustrate the fact that in all three fabrics tested, the ease of area increase under tension reduces along directions other than the principal ones.

The woven fabric shows a reduction in area at angles greater than 45 degrees, which is a result of the high Poissons ratios along these directions.

The energy of fabric deformation per unit area change increases dramatically around 45 degrees for knitted and woven fabrics (in the latter, with regard to zero area change between positive and negative values, the energy approaches infinity), while the peak for stitch bonded fabric appears around 30 degrees corresponding to the minimum ε_A per unit tensile strain.

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Assessment of Soil Fabric Friction by Means of Shear

Evaluation du frottement sol-textile à la boîte de cisaillement

This paper considers the frictional effects of placing a geotextile at the shear interface in a soil. It details an indexing method obtained by shearing a standard sand in a shear box, without loss of shear area, across a flat, fixed and supported fabric, varying both normal loads and rate of strain. It describes the scope of the investigation and discusses the results.

Cette communication examine l'effet sur le frottement de placer un géotextile au niveau du plan de cisaillement dans un sol. Elle présente une méthode d'indexation obtenue en cisaillant un sable standard dans la boîte de cisaillement, sans perte de surface de cisaillement, à travers un géotextile plan. La force normale et la vitesse de déformation sont variées. Le rapport décrit les recherches qui ont été faites et discute les résultats.

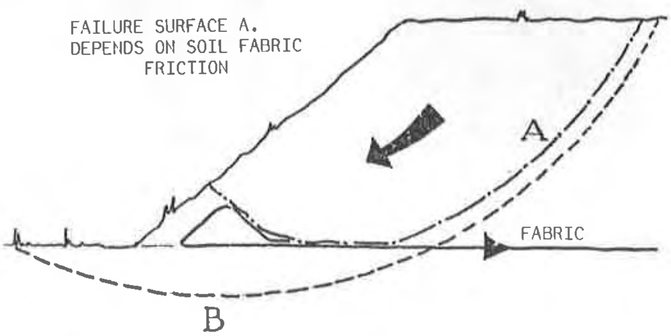
INTRODUCTION

When a fabric is placed in soil to perform the function of filtration, separation or reinforcement, its surface properties may be such that they act as a plane of discontinuity, so encouraging slippage or movement. One of the critical factors in preventing the fabric from becoming a feature of discontinuity is that of soil/ fabric friction. It will be readily appreciated that geotextiles with similar strength and filter characteristics can often have greatly varying surface textures which might well indicate differing frictional properties, therefore an indexing system for comparison is necessary. The introduction of high strength reinforcing fabrics - particularly those used in embankments - further emphasizes the need to understand more fully the frictional characteristics of fabrics, especially when estimating and predicting failure mechanisms.

Purpose and Scope of Investigation

The purpose of the investigation was to assess what effect the inclusion of a geotextile has at the shear interface in a soils system, in order to provide the basis of a frictional indexing system.

In this investigation we examined four different fabrics of varying constructions and surface textures with one standard soil, under three loading conditions, and compared these results against the shear value of the soil itself; and also against the frictional resistance of the soil and a polished mild steel plate. The four fabrics tested were representative of the principal geotextile constructions available, but the apparatus is capable of assessing the soil/fabric/soil frictional behaviour of all types of fabrics used in Civil Engineering.

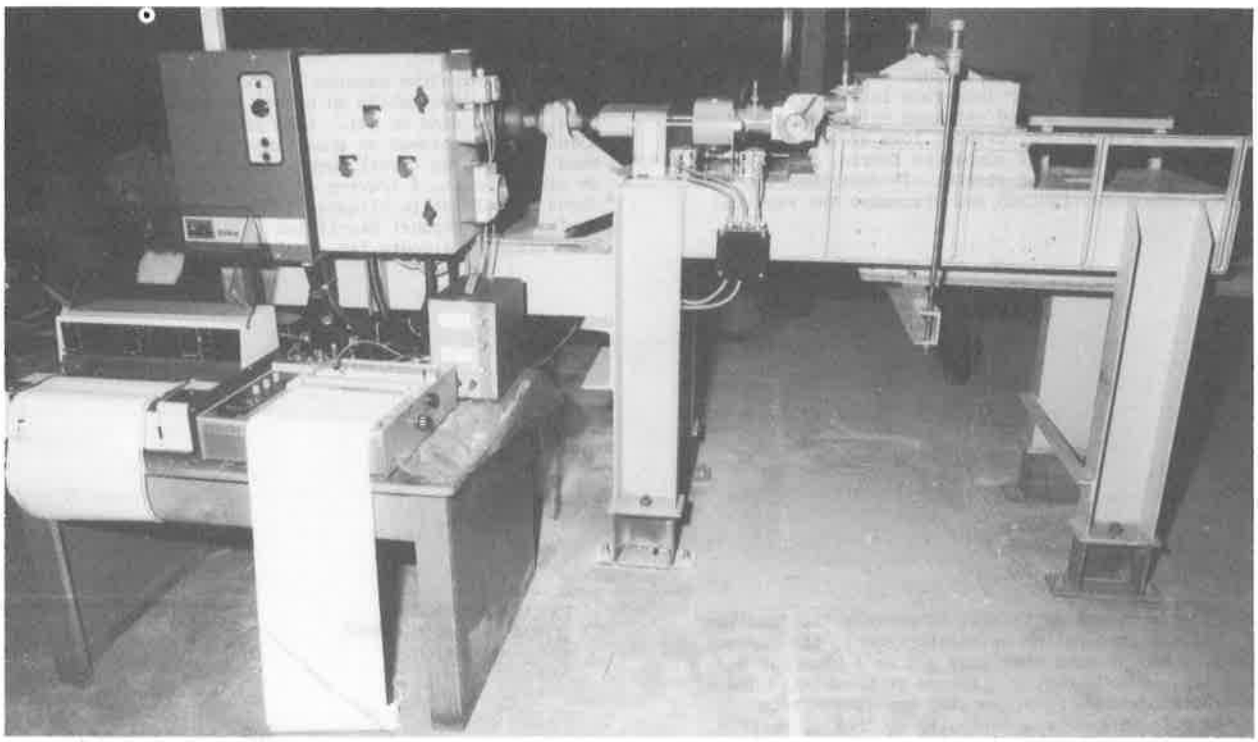


Outcome of the Investigation

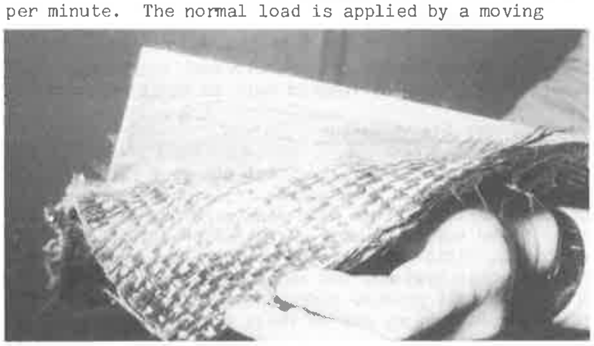
The investigation demonstrated that all four fabrics tested possessed good to adequate frictional characteristics, in spite of their very dissimilar surface textures. These tests, however, were performed with a single soil, rigid fabric, system; further investigations with dual soils systems, and cohesive and undrained soils, might well produce differences in frictional resistance between any one of these fabrics and another.

Methods, Materials and Equipment

The investigation utilized a large shear box modified to allow the inclusion of a geotextile. The shearing mechanism on this equipment permits strains, and varying loads and rates of strain, beyond those normally encountered in direct shearing of soils.



The apparatus consists of a 0.1 m^2 square shear box which is pulled over a larger lower box 0.35 x 0.40 m which has been fitted with a fabric clamping frame. The maximum available depth of the upper box is 10 cm and 12 cm in the lower box. The shear force is applied by a 1.2 kW variable speed motor through a 60:1 reduction gear by a positive buttress thread drive. The range of strain rate can be varied from 5 mm per minute to 100 mm per minute. The normal load is applied by a moving cantilever yoke capable of applying a vertical load between 30 kN/m^2 to 300 kN/m^2. The shear force is monitored by a 100 kN low range sensitive load cell. The standard soil selected was a Leighton Buzzard sand 0.60 - 0.85 mm (US sieve 20 - 30). The sub-rounded and close graded nature of the sand assisted in achieving reasonable void ratio reproducibility during the test and allowed the dry sand to be placed without extensive compaction procedure. Leighton Buzzard sand is a widely used standard sand both within the UK and elsewhere and readily available commercially.



In the experiments, the lower box was fitted with a series of rigid wooden plates and the upper shear box fitted with a Leighton Buzzard sand (BS 12).

The fabrics assessed were:

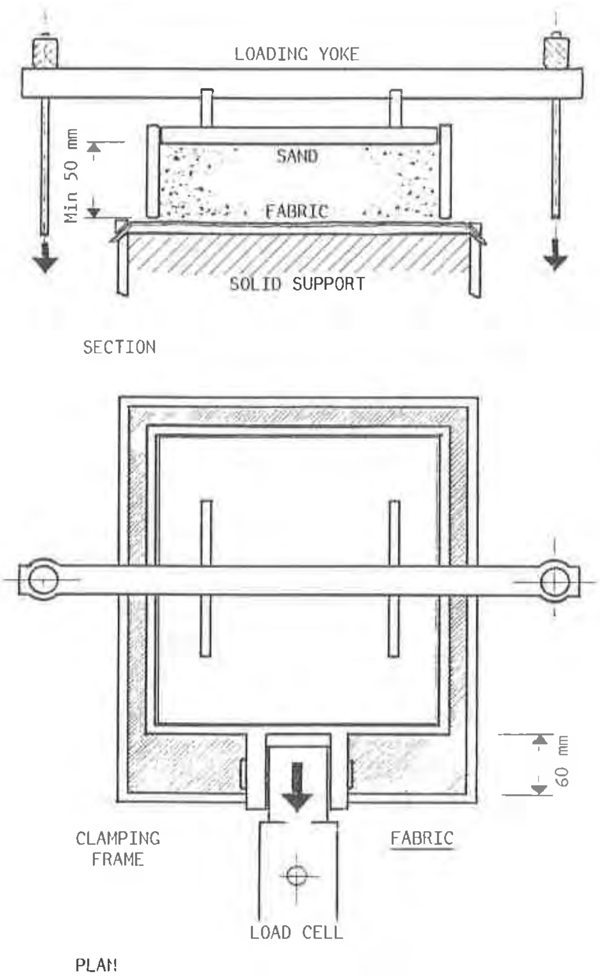
1 Needle-punched polypropylene.

2 Heat-bonded polypropylene/polyethylene.

3 Heavy weight woven polyester construction.

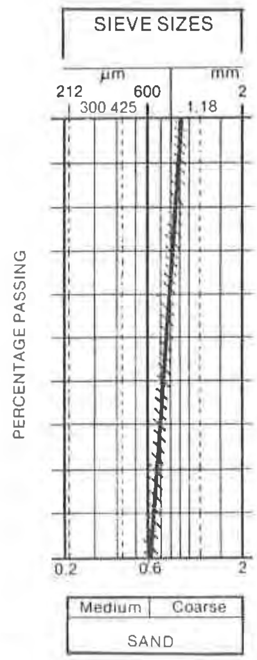
4 Light weight woven polypropylene construction.

The upper wooden plate which supports the fabric in the lower box should be not less than 1 cm. thick and the fabric should be firmly glued to its surface.



SOIL-FABRIC SHEAR BOX

Hardwood-ply on blockboard will form a good base. The glue selected should be one which hardens to a brittle state and applied so that it does not penetrate through the fabric. Synthetic solvent glues should not be used as they may affect the fabric and have poor resistance to shear because they are not rigid when set. The fabric is cut to extend beyond the wooden plate and under the clamping ring.



PARTICLE SIZE DISTRIBUTION

LEIGHTON BUZZARD SAND USED AS THE STANDARD SOIL.

The upper box is then placed in position over the fabric and the sand is carefully poured and levelled to a depth of not less than 50 mm. The maximum normal load is applied to the sand/fabric system for a period of 15 minutes prior to the shear force being applied. The normal load is then reduced to the lowest level and the shear box pulled across the fabric at a constant rate of strain. Due to the extended area of the lower box the full 60 mm. of travel can be utilised without loss of shear area; this allows a clearly defined residual shear force level to be established.

The normal load is then removed, the box repositioned and the experiment repeated at the intermediate normal load level, and again at the ultimate normal load level.

Initially, the sand was removed after each shearing procedure and the fabric replaced, but subsequently it was found that when using dry, non-cohesive soil, at least three shear passes could be made on the same fabric without affecting the results.

To establish boundary controls once all four fabrics had been tested, the fabric was first replaced by a polished mild steel plate, and frictional values ascertained at differing normal loads and rates of strain. Then similar tests were performed after the mild steel and wooden plates in the lower box had been removed and replaced by standard soil.

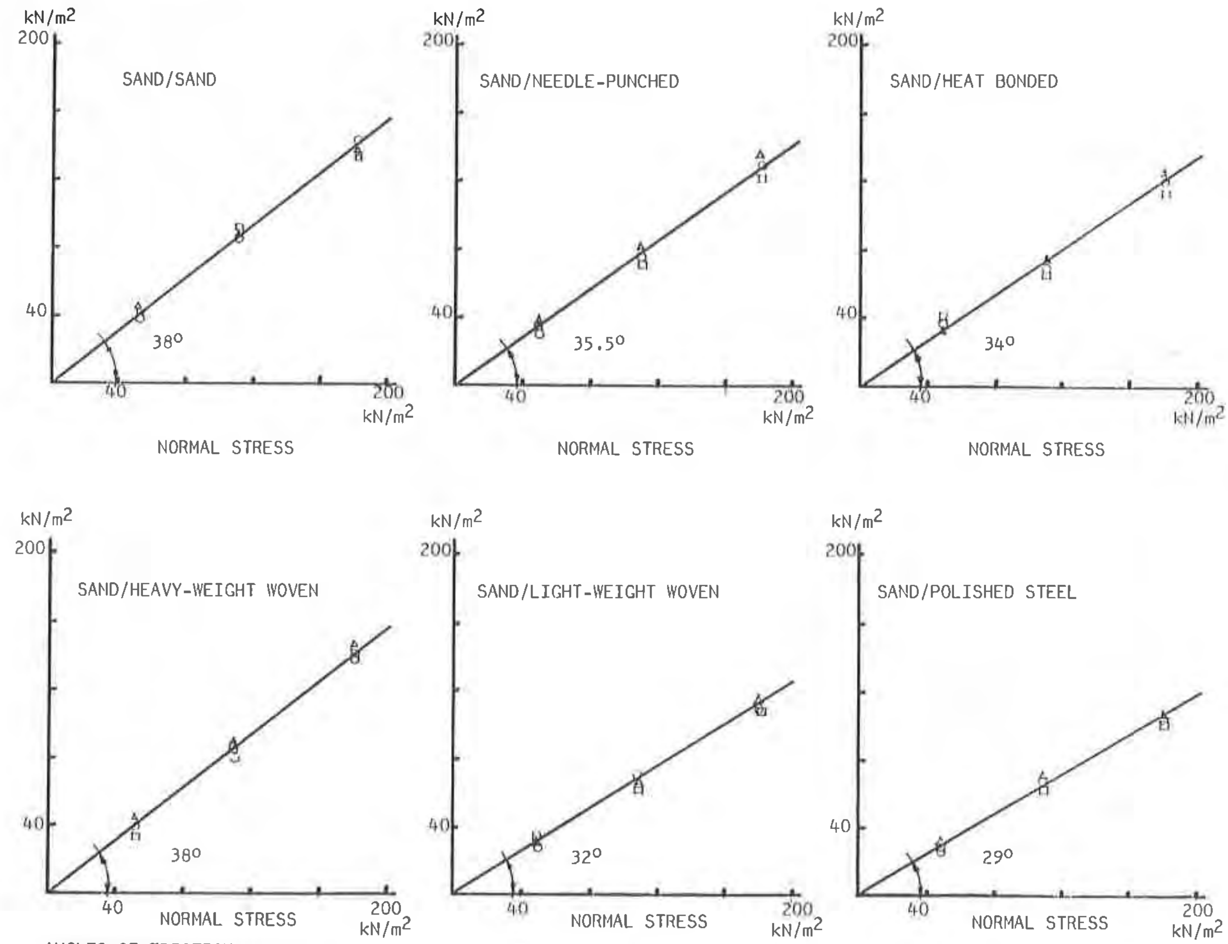
Results

The results were logged by recording the shear load on a single pen plotter, and by measuring the horizontal and vertical movement of the upper shear box by transducer readings to a three pen plotter.

The three shear stresses and normal load for a particular rate of strain régime were then plotted and a line passing through the origin was drawn connecting the points.

Discussion of Results

The results tend to confirm work done on smaller shear boxes with matching upper and lower boxes. However, the ability to achieve high strain without loss of shear area is important as most fabrics do not establish a realiable residual shear level until movements in excess of 10 mm. have been induced.



ANGLES OF FRICTION BETWEEN - SAND - GEOTEXTILES - STEEL

SAND - LEIGHTON BUZZARD SAND 0.85 - 0.60 mm (US SIEVE 20 - 30)
VOID RATIO UNDER TEST 0.60 - 0.65

RATES OF STRAIN
△ 10 mm/min
○ 25 mm/min
⌑ 75 mm/min

Coarse grain sand used as a standard soil shows very little sensitivity to the rates of strain. This was to be expected and is confirmed by J H Schmertmann*.
The object of these experiments however was to produce an indexing system and therefore the lack of effect of rate of strain can be regarded as a positive result. In practice, though, soil is very rarely of the type used in experiments, and in soils with cohesive characteristics and undrained conditions, the rates of strain may, in fact, have some relevance.

In many reinforcing applications the frictional resistance of the fabric is mobilized only after fabric extension caused by significant movement of the structure. It is therefore essential that the residual soil/fabric friction value be used when calculating structural stability.

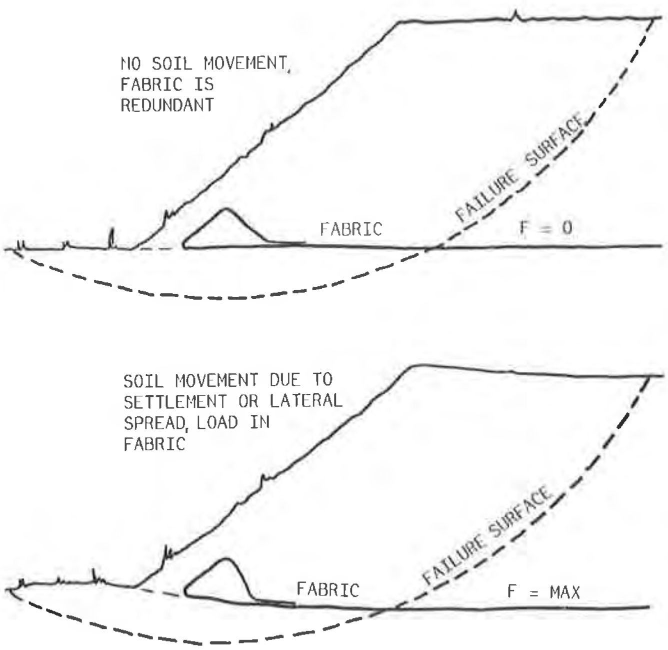
*Laboratory Shear Strength of Soil A.S.T.M. STD 740

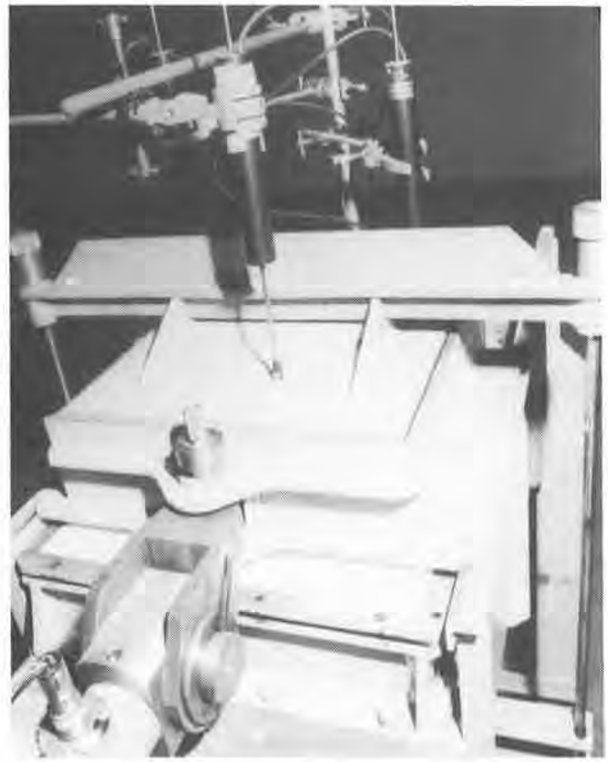
Conclusions

Many specifying authorities and geotextile committees (eg: ASTM, RILEM) have expressed the view that test procedures should be developed to assess soil/fabric friction characteristics in order that they can be better defined.

SOME TYPICAL SOIL SUPPORTED FABRIC FRICTION RESULTS OBTAINED ON A SMALL CONVENTIONAL SHEAR BOX APPARATUS

	SOIL/SOIL	HEAT BONDED FABRIC	NEEDLE PUNCHED FABRIC	MONO FILAMENT WOVEN FILAMENT
FINE SAND 0.1 - 0.2 mm	43°	40°	42°	42°
FUEL ASH	44°	36°	38°	40°
COARSE SAND 0.2 - 1.0 mm	45°	39°	44°	40°





The work described in this paper follows closely the principles advocated, viz index testing by standard soil in a shear apparatus.

Practical experience over the last ten years would indicate that the majority of geotextiles have adequate frictional resistance for typical separation and filtration applications and it is the author's view that using the indexing procedure described a lower limit of say Tan soil/fabric $\div$ Tan soil/soil 0.75

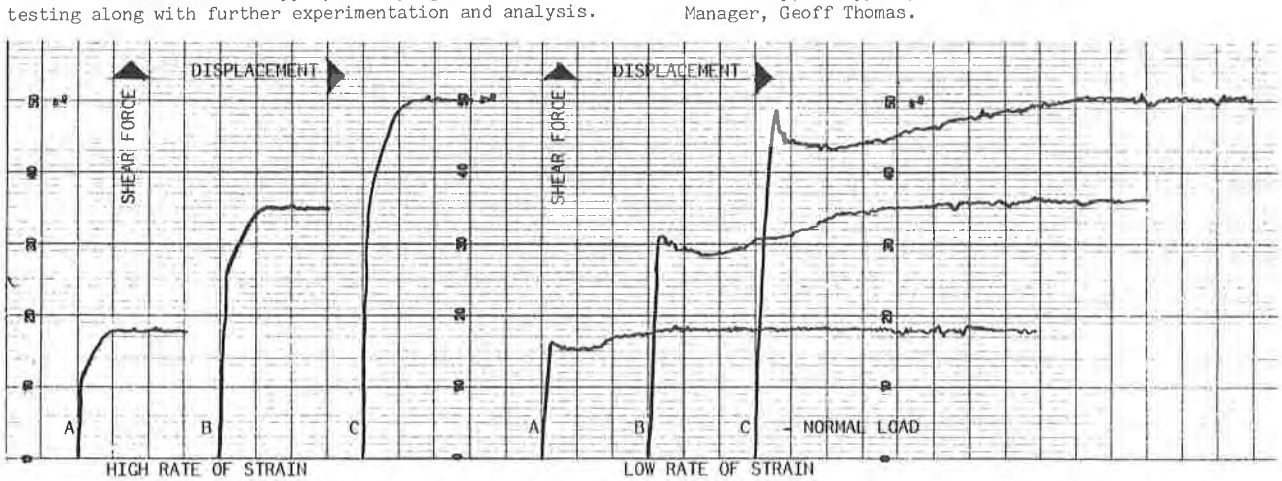
could be applied as a standard for geotextiles when used for filtration and separation.

In the case of geotextiles designed for reinforcing applications - where their performance particularly concerning load transfer from soil to fabric is critical in the structure - more exacting standards are required. This will necessitate an appropriate programme of testing along with further experimentation and analysis.

However, it must be borne in mind that in most geotextile reinforcing applications the area of fabric in contact with the soil and consequently its ability to generate a frictional anchorage even at moderately normal loads, usually far exceeds the breaking load of the fabric. Nevertheless, the engineer should be encouraged to use wherever possible any simple anchoring system such as returning the fabric round a granular bun providing that the cost for this anchorage is not excessive.

ACKNOWLEDGEMENT

Thanks are expressed to Staff of ICI Physical Testing Laboratory, Pontypool, and in particular to Group Manager, Geoff Thomas.



TYPICAL INITIAL AND RESIDUAL SHEAR FORCE BEHAVIOUR WITH RATE OF STRAIN

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Load-Extension Testing of Geotextiles Confined In-Soil

Proprietés d'extension sous charge de géotextiles placés dans le sol

The paper describes an apparatus capable of testing in-soil, geotextiles subject to first-time loading, cyclic loading, creep and stress relaxation under a variety of environmental conditions. Comparative data from unconfined in-isolation and confined in-soil tests on woven, non-woven and composite geotextiles demonstrate that to obtain data for design purposes in-soil testing is essential.

L'article décrit un appareil capable de soumettre a l'essai de géote tiles dans le sol quand ils sout exposes à premiere charge, charge cyclique, fluage et relaxation de tension, sous une varieté de conditions exercées par l'env ronnement. Des données comparatives d'essais isoles non restreints, et d'essais compressés dans le sol, sur des géotextiles tissés, non-tissés et composites, demontrent que pour obtenir des données pour ce qui concerne la conception, des essais le sol sont essentiels.

INTRODUCTION:

Geotextiles are manufactured by a wide range of processes. This imparts quite different load-extension properties to the materials. With woven geotextiles, the properties of the constituent fibres may dominate the overall behaviour, however, with non-woven and composite geotextiles the dominant factor is their internal structure. Where the internal structure does dominate, it is usually the case that this structure is liable to change when subject to stress, either by tensile stressing in the plane of the geotextile or by compressive stressing transverse to it. Adding to this, the fact that many geotextiles are anisotropic, causing them to have a degree of sensitivity to boundary strain conditions and confining stresses greatly in excess of that found for most other engineering materials. It is essential therefore to fully appreciate the imposed edge boundary conditions and confining stresses imposed during any test on geotextiles and to match whenever possible the imposed test conditions with those operating when the geotextile is functioning in the soil.

It is of course recognised that load-extension testing of geotextiles may not always be directed towards determining their in-soil behaviour. For example, regular testing will usually be carried out by the manufacturer in the factory to ensure that the manufacturing process is producing a material within some predetermined limits of variability. Also engineers on site may selectively test geotextiles to ensure that those materials delivered on site conform to the manufacturers' or clients' specified minimum properties as is required by particular contract conditions. Such tests need not exactly replicate operational conditions for the geotextile and unconfined in-isolation tests are adequate (1). Such tests should, however, not be used as a basis for comparing the in-soil performance of geotextiles, particularly when they are manufactured by different processes, nor should they be employed to obtain properties for use in the design of soil-geotextile systems. For such purposes, data from specially developed test apparatus must be employed.

This paper describes the detailed construction of an apparatus in which the load-extension properties of geotextiles when confined in-soil can be determined and the sensitivity of various types of geotextiles to in-soil test conditions is demonstrated using data obtained from first time loading and creep tests.

IN-SOIL TEST APPARATUS

Constructional Features

Basically the apparatus consists of two metal boxes containing air activated rubber pressure bellows which are placed either side of a test specimen of geotextile and clamped together by two metal side plates, Fig. 1. A layer of soil is placed between the bellows and the geotextile and when the bellows are pressurised, a lateral confining stress is imposed on the geotextile by the soil. The apparatus is designed to exert a maximum confining stress of 250 kN/m^2.

Several problems arise with this arrangement and the solutions adopted for these are:

(i) Puncture of the bellows by the soil: To avoid the soil puncturing the bellows,

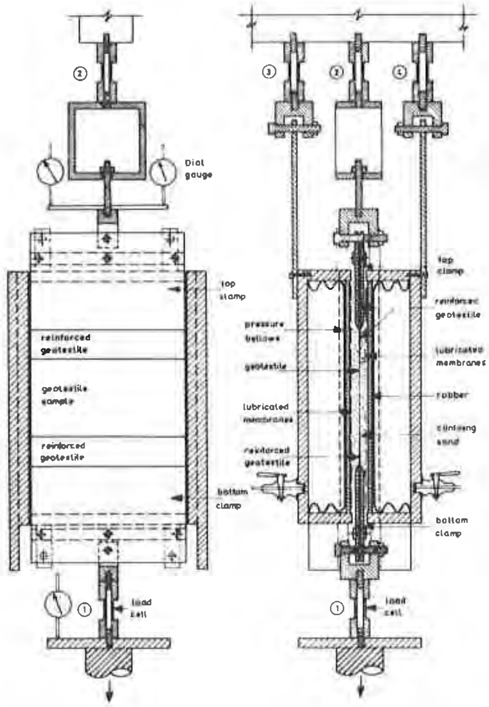


Fig. 1. Layout of the in-soil apparatus.

a layer of stiff rubber is placed between the soil and the bellows. This layer of rubber also helps to ensure the uniformity of the lateral confining stress.

(ii) Transfer of applied axial load through the soil to the body of the apparatus: Firstly to minimise axial load transference into the apparatus, thin layers of rubber membrane separated by silicone grease are placed between the stiff rubber and the soil to reduce soil/ rubber friction. Secondly, the whole apparatus is fixed to the load frame and load cells are incorporated into the fixtures to measure any residual transfer of axial load. This arrangement is identified as load cells 3 and 4 in Fig. 1. Thirdly, the net axial load imposed on the geotextile is measured independently by load cell 2 and a check made that the load applied through load cell 1 equals the combined load measured in load cells 2, 3 and 4. Calibration tests have been previously reported by McGown et al (2).

(iii) Maintenance of contact between soil and geotextile during straining: The end clamps are the same thickness as the soil layer in contact with the geotextile. The leading edges of the clamps are sloped back

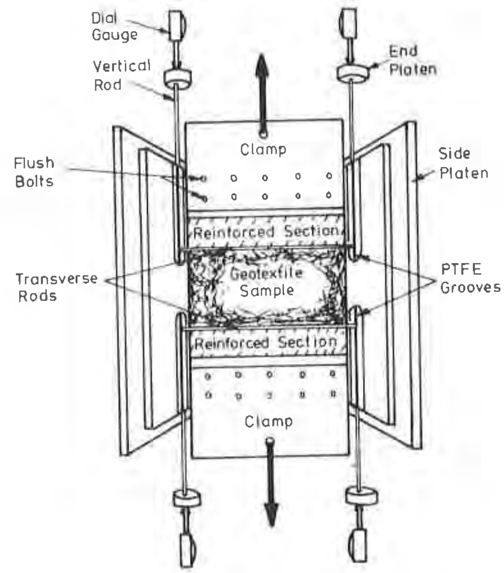


Fig. 2. Scheme for measuring the extension of test specimen in-soil.

and flush bolted into the very stiff resin reinforced ends of the test specimen. The reinforced ends of the specimen extend beyond the clamp into the soil. The soil is therefore initially in contact with the unreinforced and the exposed reinforced sections of the geotextile. The length of the exposed end of the reinforced geotextile is chosen to be greater than the maximum extension of the unreinforced section during the test. This ensures that the extent of the confining soil is greater than that of the unreinforced section of geotextile at all times during the test.

(iv) Measurement of the extension of the unreinforced section of the test specimen within the soil: It cannot be assumed that slippage between the clamp and specimen does not occur, nor that the very stiff reinforced sections of the geotextile are rigid. The extension of the unreinforced section of geotextile must therefore be measured directly. To do this, a thin steel rod is bonded onto each of the leading edges of the reinforced geotextile. The tips of the rods are machined to fit into slots in the two side plates. These slots are lined with P.T.F.E. which has extremely low frictional resistance. Further steel rods are located in the slots and extend out of the apparatus to end platens. The transverse rods connect into the rods within the slots and any movements are therefore transferred out of the apparatus. Dial gauges resting on the end platens outside the apparatus measure the internal movements. The arrangement is shown schematically in Fig. 2.

Test Specimen Sizes

To establish suitable sizes and shapes of test specimens, an unconfined in-isolation test programme was undertaken with various types of geotextile. To ensure that local variations in geotextile construction were taken into account, a minimum dimension of 100 mm was adopted. Tests were then conducted on test specimens with a height of 100 mm and various widths up to 500 mm. It was found that needle punched geotextiles were the most critically affected by the shape and size of the test specimen but that for test specimen sizes beyond 200 mm wide and in the strain range of 0 to 40 per cent, little significant difference was recorded in their measured load-strain properties, Fig. 3. Thus 200 mm wide by 100 mm high test specimens were adopted as standard minimum sizes, (3).

For test specimens with the minimum standard dimensions, the in-soil apparatus was designed with a 10 mm thick soil layer on each side of the test specimen. This limited the types of soil that could be tested to sand sizes and finer. To accommodate larger soil particle sizes and allow the testing of larger geotextile specimen sizes, another version of the apparatus was designed to test up to 500 mm wide by 250 mm high geotextile specimens with a 25 mm thick layer of soil on each side of it. The two sizes of apparatus are shown mounted in their loading frames with their associated data logging equipment in Fig. 4.

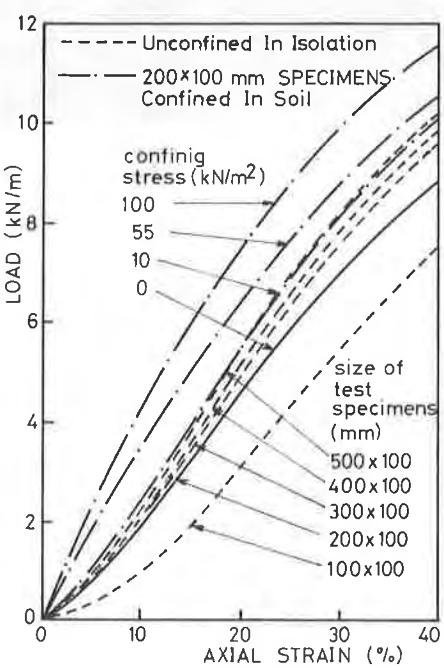


Fig. 3. Unconfined in-isolation and confined in-soil load-axial strain data for Bidim U24.

Test Conditions

As these tests are primarily intended to measure properties of geotextiles appropriate for use in design calculations, it is important that the environmental conditions of the test be similar to the operational conditions, (1). For this reason the geotextile specimens should be tested in a soaked condition and at temperatures relating to the in-soil temperatures, which in United Kingdom is approximately 10 °C. To

Fig. 4. General view of in-soil apparatus

accomplish this temperature control, insulated temperature cabinets have been constructed to fit around the apparatus and tests may be conducted within these at temperatures from 0 °C to + 40°C.

When fitted in the loading frames shown in Fig. 4, axial loads up to 10 tonnes may be applied at constant rates of strain in the range 20 to 0.0002 per cent per minute. When the load frames are fitted with a suitable activator, cyclic loading may be carried out. To accommodate creep load testing, specially designed rigs were constructed, which are shown schematically in Fig. 5. A creep test

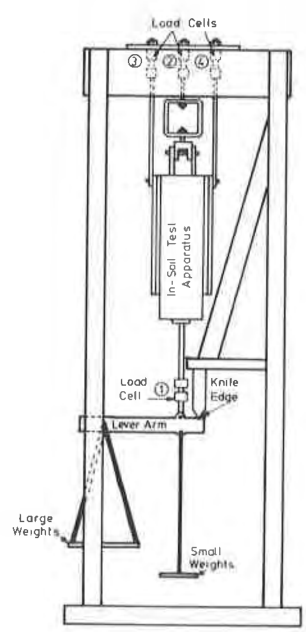


Fig. 5. Creep test rig.

Table 1. Basic Characteristics of Geotextiles Tested.

CHARACTER-ISTIC	LOTRAK 16/15	TERRAM 1000	BIDIM U24	PROPEX 6067
Method of Construction	Woven tapes	Non-woven Melt bonded filaments	Non-woven needle punched filaments	Composite Woven and needle punched
Polymer(s) Composition	100% Polypropylene	67% Polypropylene 33% Polyethylene	100% Polyester	100% Polypropylene
Specific Gravity	0.91	0.9	1.39	0.91
Weight/ Unit Area (g/m^2)	120	140	210	650
Nominal Thickness (mm)	0.3	0.7	1.9	3.5

set up with a 200 x 100 mm sample size apparatus is shown in Fig. 6. By modifying the lever system to provide fixed extension of the geotextile with measurement of the applied loads, stress relaxion tests may also be conducted in this set-up.

Now to illustrate the sensitivity of various types of geotextile to in-soil test conditions, data from first time loading tests on four commercially available geotextiles and creep tests on two of them are detailed.

FIRST TIME LOADING TESTS

Tests were conducted on four geotextiles manufactured by various processes and possessing differeing properties, as identified in Table 1. All the test specimens were 200 mm wide. In the opposite direction they had 100 mm unreinforced central section with a 100 mm resin impregnated reinforced section on either end. The clamps were bolted through the outer 60 mm of the reinforced ends to leave 40 mm of reinforced geotextile exposed on each side of the central section. Unconfined in-isolation tests were carried out on some of these specimens in a dry condition at 20 ^oC and at a constant rate of strain of 2 per cent per minute. The others were tested under the same environmental conditions but were confined in-soil at a pressure of 100 kN/m^2. The confining soil was Leighton Buzzard sand with a particle size range of 0.3 to 2.0 mm, mean diameter of 0.85 mm and uniformity coefficient of 1.22.

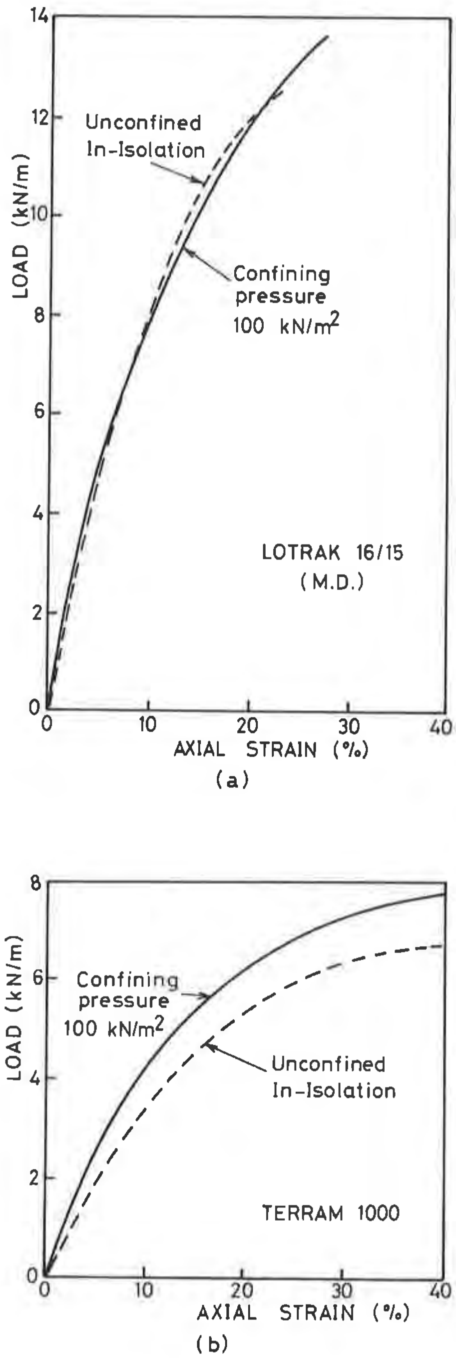
The test data obtained from these comparative tests are shown in Fig. 7. They demonstrate that the structured non-woven and composite geotextiles exhibit significant changes in the shape of their load-strain curves when tested in-soil. The woven geotextile does not show much change as it depends for its strength on aligned tapes which are not greatly affected by embedment in the sand used. The changes in the shape of the curves of all the materials are quantified in terms of the percentage change in the initial and secant slopes in Table 2. These data show that the difference between unconfined in-isolation testing and confined in-soil testing can indeed be very significant.

Fig. 6. General view of creep test rig.

In order to demonstrate for structured geotextiles the relative importance of in-soil confining stresses to the width to height ratio of test specimens, further tests were carried out on Bidim U24 specimens at confining stresses of 10 and 55 kN/m^2. These data have been plotted over the unconfined in-isolation test data obtained for this geotextile from test specimens with different width to height ratios, as shown in Fig. 3. From this it can be ssen that the in-soil tests with 10 kN/m^2 confining stress in-soil on 200 x 100 mm specimens produced a load-strain curve similar to that of a 500 x 100 mm wide unconfined in-isilation test specimens, however, confining stresses of 55 and 100 kN/m^2 on 200 x 100 mm specimens produced quite different shapes of curve in the strain range 0 to 40 per cent. Thus testing such a geotextile confined in-soil cannot be replicated simply by testing wide specimens.

Table 2. Changes from Unconfined In-Isolation Load-Strain Curves Due to Confinement In-Soil at 100 kN/m^2 Confining Stress

MEASURED VALUE	LOTRAK 16/15	TERRAM 1000	BIDIM U24	PROPEX 6067
Initial Slope	+8%	+78%	+270%	+254%
5 per cent Secant slope	+1%	+46%	+206%	+ 39%
20 per cent Secant slope	-1%	+16%	+64%	+ 16% (18% strain)



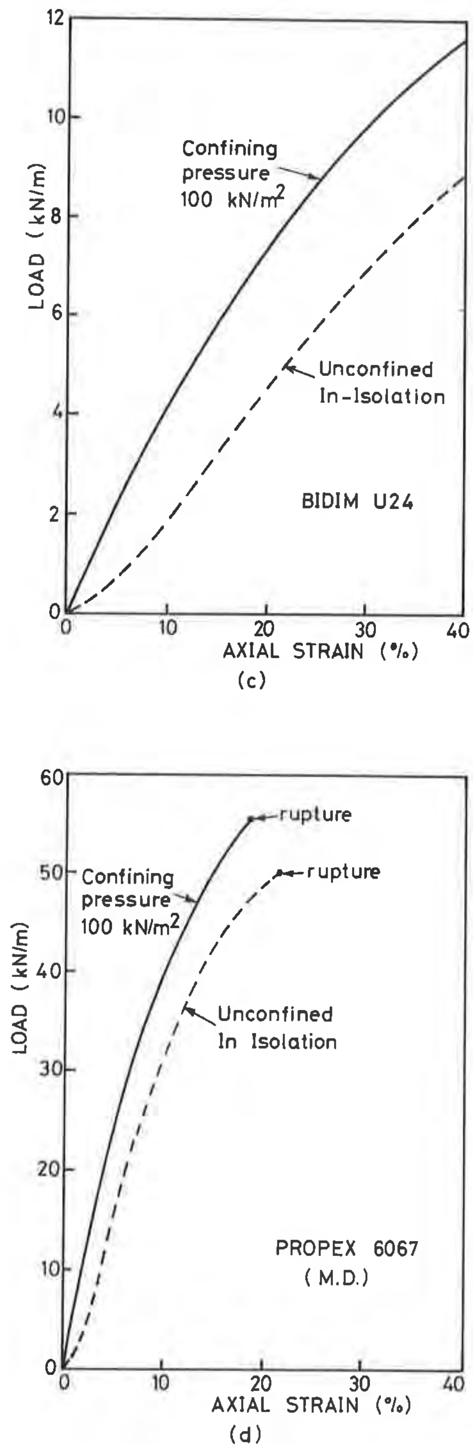


Fig. 7. Load strain data for the geotextiles tested.

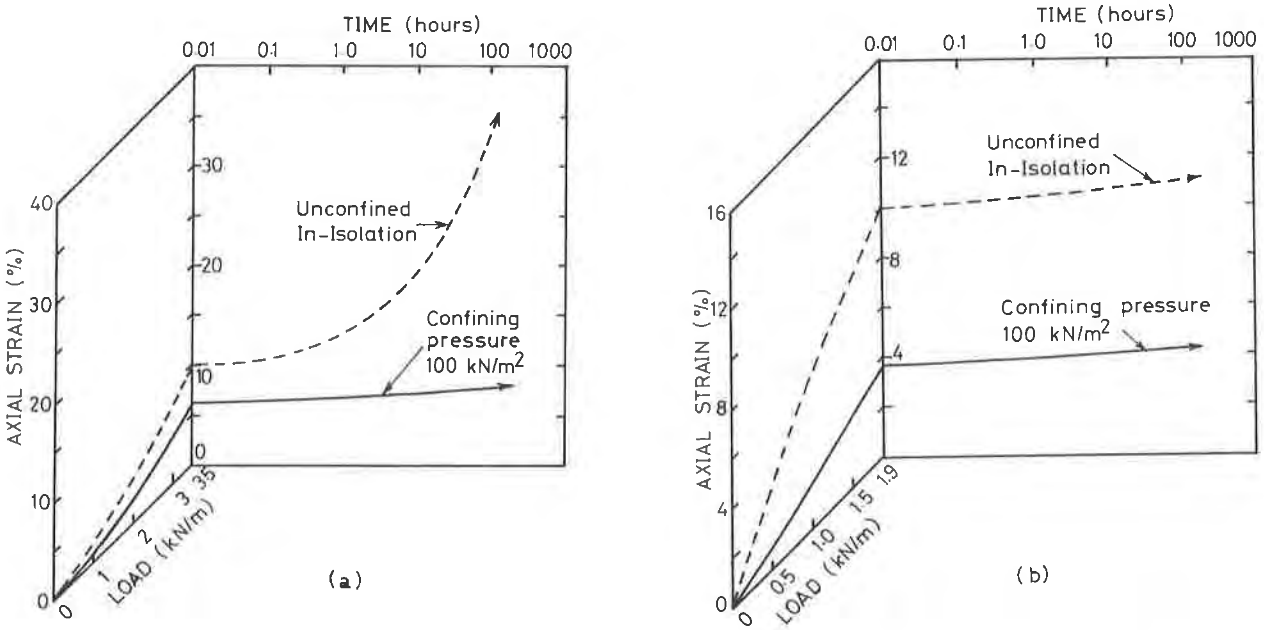


Fig. 8. Creep test data for (a) Terram 1000 and (b) Bidim U24.

CREEP TESTS

Further specimens of Bidim U24 and Terram 1000 prepared in the same way as the first time load test specimens, were subjected to creep testing unconfined in-isolation and confined in-soil under the same environmental conditions as before. The confining soil was the same and the confining stress was maintained at 100 kN/m^2. The level of loading used was that to produce 10 per cent strain in each geotextile when tested unconfined in-isolation. The loads for the Bidim and Terram specimens were 1.9 and 3.5 kN/m respectively, as indicated in Fig. 7.

The comparative data obtained from these tests are given in Fig. 8. The substantial reductions in strain when the geotextiles are confined in-soil can be seen to have two components; a reduction in initial strains and a reduction in creep strains. Clearly unconfined in-isolation creep testing grossly overestimates long term operational strains in these geotextiles.

CONCLUSIONS

1. The in-soil test apparatus described in this paper is capable of testing geotextiles subject to first time loading, cyclic loading, creep and stress relaxation under a variety of environmental conditions when confined by soil.

2. Comparative data from unconfined in-isolation and confined in-soil tests, conducted at a rate of strain of 2 per cent per minute on dry geotextiles at 20 °C when confined in a uniform sand, showed that highly structured non-woven and composite geotextiles significantly change the shape of their load-strain curves when tested in-soil. The woven geotextile with a simpler structural arrangement did not exhibit such a change.

3. Further comparative first time loading test data obtained under the same conditions for a needle punched geotextile, showed that in-isolation testing of very wide unconfined test specimen does not replicate confined in-soil behaviour.

4. Comparitive creep test data obtained from unconfined in-isolation and confined in-soil tests on two non-woven geotextiles showed that a significant reduction in their long term strains occurred when they were confined in-soil.

5. To obtain load-extension data for design purposes, confined in-soil testing appears to be essential, particularly for non-woven and composite geotextiles.

ACKNOWLEDGEMENTS

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Effect of Confining Pressure on Performance of Geotextiles in Soils

L'effect de la pression de confinement sur la performance de géotextiles enterrés

The purpose of this research was to study the effect of confining stress and soil support and cover materials on tensile strength characteristics of geotextiles. This has direct application to the equivalent tensile stress and type of failure of the geotextiles when embedded in the ground.

The effect of varying the type of support and cover materials on the stress-strain relationships of two different geotextiles, Polyfilter X (woven) and Mirafi 140S (nonwoven), were investigated for normal stresses of 47.9 kPa (0.5 tsf), 95.8 kPa (1 tsf), 191.6 kPa (2 tsf) and 383.2 kPa (4 tsf). It was found that, in general, the "equivalent tensile stress" at failure increased with increasing confining stress.

Equations for the stress-strain curves and for the tangent moduli as a function of normal stress were derived for Polyfilter X and a dry sand/sand support/cover combination. The expressions developed may be used directly in finite element models of soil-fabric structure interactions. The procedure described may be used for other combinations of geotextile and support/cover materials to develop similar relationships.

Le but de cette recherche était d'étudier l'effet de la pression de confinement du support des sols et des matériaux de couverture sur les caractéristiques de tension de géotextiles. Une application directe de cette étude est la définition de la tension équivalente et du type de rupture des géotextiles enterrés.

L'effet de la variation du type de support et matériau de couverture sur la relation force-tension de deux géotextiles différents, Polyfilter X (tissé) et Mirafi 140S (non tissé) a été observé pour des tensions normales de 47.9 kPa (0.5 tsf), 95.8 kPa (1 tsf), 191.6 kPa (2 tsf), et 383.2 kPa (4 tsf). Un résultat général est que la "tension équivalente" de rupture s'accroit avec la tension de confinement.

Les équations donnant les courbes force-tension et les modules de tangente en fonction de la tension normale ont été etablies pour le cas de Polyfilter X couvert et supporté par un mélange de sable sec et sable ordinaire. Les formules ainsi trouvées peuvant s'employer directement dans un modéle à eléments finis de l'interaction sol-structure en géotextile. L'approche peut s'employer à l'étude d'autres combinaisons de géotextiles et matériau de support/couverture, conduisant à des formules semblables.

INTRODUCTION

Currently, geotextiles are mostly used as a reinforcement material to improve a weak soil's load-bearing capacity. The mechanism developed by the use of the geotextiles as a reinforcement material is a function of the stress distribution due to the interface friction between the geotextile and the soil. Theoretically, the geotextile, when embedded in soil, is considered to be a pinned beam so that horizontal and vertical movements at the extremities are prevented. Rotation is allowed so that tension in the geotextile is developed as loads are applied. Practically, in order to design geotextile-reinforced systems, the effect of soil confinement and other placement factors on the performance of the geotextile must be evaluated. This research provides an insight into the effect of soil confinement, cover and support materials and soil moisture content, on the tensile strength parameters of selected geotextiles.

TESTING EQUIPMENT, MATERIALS AND TESTING PROCEDURES

Testing Equipment and Materials

Sample Box

A specially designed sample box was used in conjunction with a direct shear device to perform geotextile tensile tests. The box was made up of two sections which can be connected together with screws similar to a conventional direct shear device shear box. (Fig. 1). The spacing between the two sections of the box can be adjusted to allow the placement of a geotextile specimen in the horizontal plane between the support material (at the bottom) and the cover material (at the top). When assembled, the box was a square with interior side dimensions of 6.35 cm (2.5 in.) and a height of 3.81 cm (1.5 in.). A square cover plate slightly less than 6.35 cm on a side, was

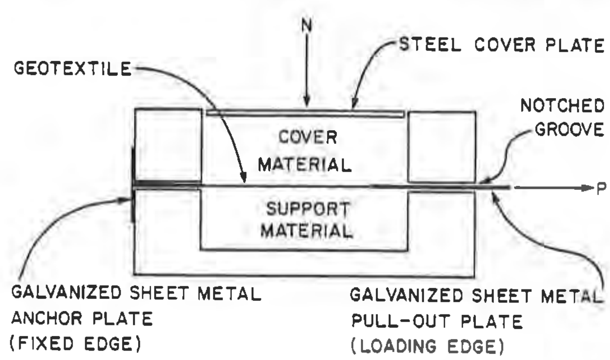

Fig. 1 Sample box for tensile tests.

placed on the top surface of the cover material. This cover plate distributed the normal load (N) from the test device uniformly over the soil material contained in the box.

Geotextile Specimen

To make sure the geotextile specimen received a uniformly distributed normal pressure from the vertical load head, it was cut in a 5.08 cm (2 in.) square. Thus the geotextile specimen could be placed in the box in such a way that it would be confined under the normal stress during the entire loading sequence up to 25% strain or until failure. The loading side of the specimen and its opposite extremity were glued and rivetted between two thin pieces of sheet metal that formed a type of clip (Fig. 1). The pull-out plate on the loading edge was connected to the horizontal loading arm of the direct shear device through which the tensile load (P) was applied. The opposite extremity was fixed to the far side of the box.

Materials Used

Two types of geotextiles were studied: woven and nonwoven. Three kinds of woven geotextile were tested: Polyfilter X, Mirafi 100X and Mirafi 500X. The nonwoven geotextiles tested were Mirafi 140S, Typar 3601, and Bidim C-34. Tests were performed with confinement oriented along grain and across the grain for each of the fabrics.

Sand and gravel were used as the confinement materials. The sand was #30 Ottawa sand having an approximate dry density, γ_d, of 14.13 kN/m^3 (90 pcf). The gravel was a fine river run gravel having uniform gradation and an approximate dry density, γ_d, of 16.96 kN/m^3 (108 pcf).

Testing Procedures

For "dry tests", the geotextile specimens were placed on top of the air dried support material which was placed into the bottom half of the sample box at a predetermined dry density (γ_d = 14.13 kN/m^3 for #30 Ottawa sand; γ_d = 16.96 kN/m^3 for fine gravel). The top half of the sample box was then positioned over the bottom half and fastened by hand-tightening the connecting screws. Care was exercised in placing the geotextile specimen so that the sheet metal pull-out plate and anchor plate fit in the grooves notched into the upper and lower portions of the box. The air dried cover material was then introduced into the top half of the sample box. If the support and cover materials were the same, the materials were placed at the same dry density. After the metal cover plate was placed on the top of the cover material, the loading head of the test device was lowered to make contact with it. Following application of the normal load, the sheet metal connector was attached to the horizontal loading arm of the direct shear device and the test begun.

For "wet tests", the soil and geotextile test specimens were prepared in the same way as for the "dry tests", except that after attachment of the pull-out plate to the direct shear device loading arm, the samples were soaked in water for 24 hours before testing.

All pull-out tests were performed at a horizontal displacement rate of 1.27 mm/min. (0.05 in./min.). The first reading of pull-out load was recorded for a horizontal displacement of 0.254 mm (0.01 in.). Thereafter, readings of pull-out load were recorded for 0.508 mm (0.02 in.) increments until 5.08 mm (0.2 in.) of deformation occurred; then readings were taken at 1.27 mm (0.05 in.) increments until 12.70 mm (0.5 in.) of total horizontal deformation occurred. By using this procedure, the low stress range of the stress-strain (load-deformation) curve was well defined. All specimen-soil combinations shown in Table 1, were tested at normal stresses of 47.9 kPa (0.5 tsf), 95.8 kPa (1 tsf), 191.6 kPa (2 tsf), and 383.2 kPa (4 tsf).

In this study, field conditions were simulated in the laboratory by keeping the entire geotextile confined under the normal pressure during horizontal loading until failure of the geotextile or 25% strain occurred. Also, the geotextile sample was free to deform transversely so that necking could occur in the portion of the sample that was under normal stress just as it could occur in the field.

Table 1: Summary of Testing Program

Each series included tests at normal stresses of 0 kPa, 47.9 kPa (0.5 tsf), 95.8 kPa (1 tsf), 191.6 kPa (2 tsf) and 383.2 kPa (4 tsf). All soils were air dry except where noted.

Geotextile Type	Sand-Sand Interface	Gravel-Sand Interface	Gravel-Gravel Interface
Woven			
Polyfilter X	X[a]	X	X
Mirafi 100X	X	-	-
Mirafi 500X	X	-	-
Nonwoven			
Mirafi 140S	X[a]	X	X
Typar 3601	X	-	-
Bidim C-34	X	-	-

[a] Tests were also performed for wet interfaces.

PRESENTATION AND DISCUSSION OF TEST RESULTS

Stress-strain curves were used to evaluate the tensile strength characteristics of the geotextiles tested. To develop such curves, plots of the variation of sample width versus loading deformation for each of the geotextiles were obtained under conditions of zero confining stress (Fig. 2). It was assumed that the thickness of the specimen did not change during loading, even in the reduced section. It was also assumed that the amount of necking of the confined geotextile at a given horizontal deformation was the same as that of the unconfined geotextile at the same horizontal deformation.

Under normal stress, σ_n, when the fabric was pulled out, shear stresses were mobilized along the surface area of the geotextile sample. The magnitude of the interface shear stress, τ, is equal to $c + \sigma_n \tan \phi$; where c and ϕ are the interface adhesion and the friction angle, respectively, between the confining material and the geotextile. However, because the displacements are variable along the length of the fabric, the shear stresses are not uniformly distributed.

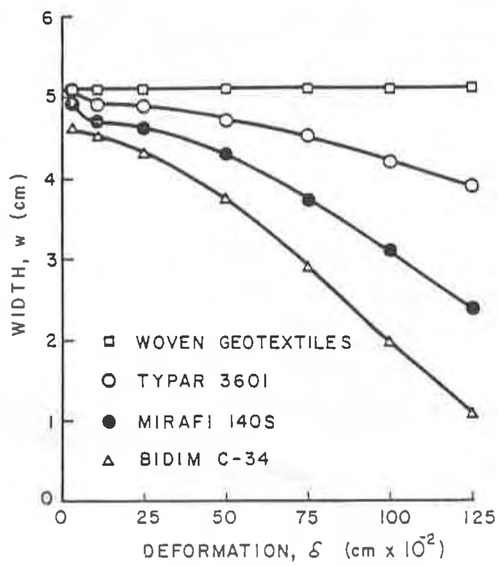

Fig. 2 Geotextile specimen widths versus horizontal deformation (unconfined loading).

Figure 3 shows a schematic of the way in which interface shear stresses acting on the geotextile specimen were assumed to develop. Immediately after application of the horizontal force P by the direct shear device, shear stresses began to be mobilized on the geotextile-soil interfaces. These shear stresses (τ_{st} on the top and τ_{sb} on the bottom) are due to the applied normal stress. The equilibrium length, L_e, is that portion of the geotextile where shear stresses become mobilized under a certain normal stress for a given horizontal load of magnitude P. Initially, there will be no movement and no tensile stress in the geotextile in length L which is beyond L_e (Fig. 3). The distribution of τ_{st} and τ_{sb} is not uniform even within L_e. Summation of the horizontal forces shown in Figure 3a yields $(\tau_{st} + \tau_{sb})A_{fe} = P$; where $A_{fe} = L_e$ times the average lateral width of the fabric (W_{avg}). If a cross-section A-A is cut at a length L_c less than L_e, the force equilibrium expression from Figure 3b is $(\tau_{st} + \tau_{sb})A_{fc} + T_G = P$: where $A_{fc} = L_c \times W_{avg}$, and T_G = geotextile tensile force. When the applied horizontal load P reaches its ultimate value, full interface shearing resistance becomes mobilized along the total surface area of the geotextile, and the geotextile sample moves as a whole. At this point it is reasonable to assume a uniform shear stress distribution along the interface due to the reorientation of the soil particles. If the support and cover materials are the same, then $\tau_{st} = \tau_{sb}$. The galvanized sheet metal surfaces were very smooth, and the shear stresses developed along them due to the normal stresses were assumed to be very small compared to the shear stresses developed on the geotextile-soil interface. Therefore, the mobilized shear stresses along the sheet metal-soil interface were neglected.

Equivalent tensile stresses (σ_T) under various normal stresses (σ_n) were obtained by dividing the geotextile stretch-out load (P) by the cross-sectional area of the geotextile. Equivalent tensile stresses are plotted in Figures 4 through 7 at 5% strain increments for the sake of clarity; but the stress-strain curves themselves were drawn using intermediate points as well.

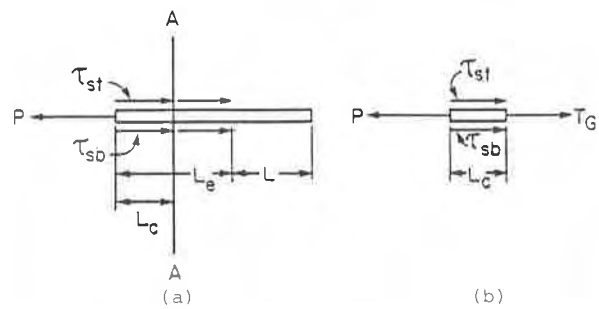

Fig. 3 Horizontal forces and interface shear stresses acting on geotextile specimen.
(a) Mobilization of the shear stresses
(b) Cross-section A-A

EQUIVALENT TENSILE STRESS OF GEOTEXTILES

The width of all woven geotextiles remained virtually constant during tensile testing without soil in the sample box (Fig. 2). Therefore, stress-strain curves were determined for constant cross-section at all stress levels. The width of all nonwoven geotextiles, however, diminished during tensile testing and subsequently failed by necking (Fig. 2). Therefore, stress-strain curves were determined for reduced cross sections depending upon the amount of displacement (strain) attained during the test. The equivalent tensile stress (σ_T) was obtained by dividing the geotextile stretch-out load by the cross sectional area of geotextile.

RESULTS SHOWING EFFECT OF COVER AND SUPPORT MATERIALS ON EQUIVALENT TENSILE STRESS

When both cover and support materials were dry #30 Ottawa sand, high shear stresses were mobilized along the geotextile-soil interface due to the application of normal stress. For the case of the geotextile confinement under dry #30 Ottawa sand as cover material and dry fine gravel as support material, there was less surface contact on the geotextile-gravel interface than on the geotextile-sand interface. Thus, there was less shear stress mobilization along the geotextile-gravel interface than along the geotextile-sand interface. The net result was a reduction in the equivalent tensile stresses as shown in Figures 4 and 5 for one type of woven fabric and one type of nonwoven fabric, respectively, under 383.2 kPa (4 tsf) normal stress. Also, it is noticed from these two figures that the geotextile gravel-gravel interface exhibited the lowest equivalent tensile stresses as compared to the other geotextile-material combinations due to the lowest surface contact at the interface and thus less mobilization of shear stress.

RESULTS SHOWING EFFECT OF MOISTURE CONTENT

The fibers used to make fabrics are generally hydrophobic. Therefore, they are relatively insensitive to moisture regain. The tensile strength for the woven and nonwoven geotextiles tested with wet #30 Ottawa sand as cover and support materials was the same as for the dry case under zero normal stress. However, when woven geotextiles were tested with normal stresses applied to wet cover and support materials, it appeared that some water was retained on the geotextile-soil interface during testing, probably due to the rough surface of the woven fabric. This wet geotextile-soil interface caused the geotextile to slip slightly. Thus shear stresses mobilized on the geotextile interface decreased with a corresponding reduction in the equivalent tensile stresses (Fig. 4). The equivalent tensile stress reduction, at 25% ε, was approximately 30%.

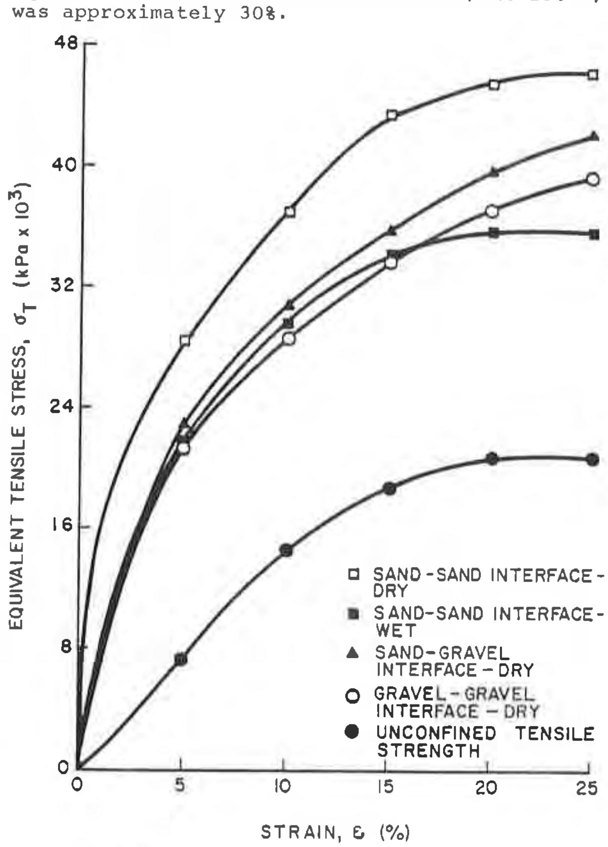

Fig. 4 Stress-strain curves for Polyfilter X (woven fabric) under 383.2 kPa (4 tsf) normal stress for various combinations of cover and support materials.

Additionally, nonwoven geotextiles tested under wet conditions sometimes displayed slightly higher equivalent tensile stresses under increased normal stresses than when they were tested under dry conditions (Fig. 5). This may be due to the "wick" action of the geotextile in draining moisture from the soil near the interface. The resulting increase in effective stresses in the soil could cause the cover and support materials to densify in the area of the geotextile and lead to the observed almost 5% increase in the equivalent tensile stress.

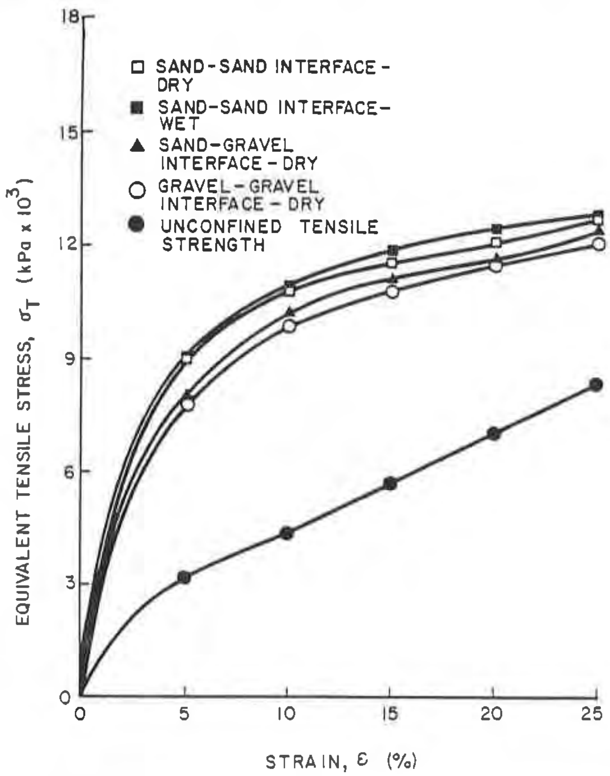

Fig. 5 Stress-strain curves for Mirafi 140S (nonwoven fabric) under 383.2 kPa (4 tsf) normal stress for various combinations of cover and support materials.

RESULTS SHOWING EFFECT OF NORMAL STRESS ON EQUIVALENT TENSILE STRESS

Stress-strain curves for Polyfilter X woven fabric and Mirafi 140S nonwoven fabric (support and cover material = dry #30 Ottawa sand) are shown in Figures 6 and 7 respectively, for various normal stresses. High shear stresses were mobilized along the geotextile-soil interface due to the application of normal stresses. This is clearly noticed in the two stress-strain figures especially in Figure 6 where a significant increase in the equivalent tensile stress can be noted for even a relatively small normal stress of 47.9 kPa (0.5 tsf) as compared to the unconfined specimen. The higher the normal stress, the higher the equivalent tensile stress at the same strain level. The results for all the other geotextile-sand combinations followed the same pattern.

Almost all the stress-strain curves of the geotextiles exhibited an initial linear portion until approximately 0.5% strain level. In addition, all curves have identical tangent moduli after about 20% strain regardless of the type of geotextile or the magnitude of normal stresses. As indicated previously, it is felt that this is due to the full and even mobilization of all the shearing stresses on the geotextile-soil interfaces.

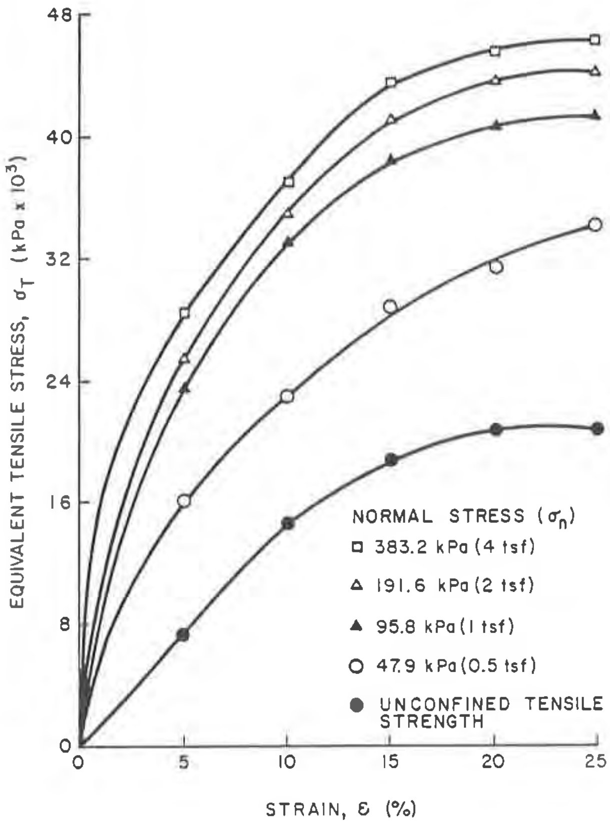

Fig. 6 Stress-strain curves for Polyfilter X (woven fabric); support and cover material = dry #30 Ottawa sand.

RELATIONSHIP BETWEEN TANGENT MODULUS (E_t) AND NORMAL STRESS (σ_n)

Because problems involving geotextile-soil interactions are often modelled and solved by the finite element method (FEM), an equation was developed for the stress-strain curves and for the tangent modulus (E_t) as a function of the normal stress (σ_n), following a procedure from Duncan and Chang (1). The initial tangent moduli (E_i), of the stress-strain curves under the various normal stresses, were computed and plotted versus their corresponding normal stresses (σ_n), as shown in Figure 8 for

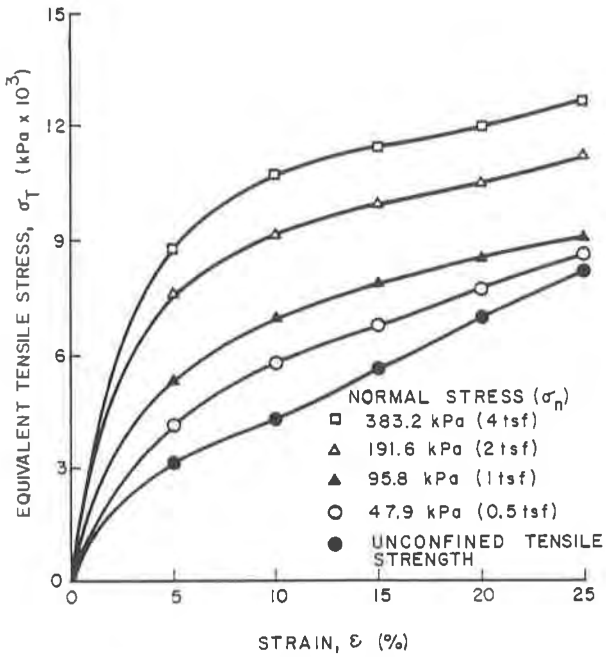

Fig. 7 Stress-strain curves for Mirafi 140S (nonwoven fabric); support and cover material = dry #30 Ottawa sand.

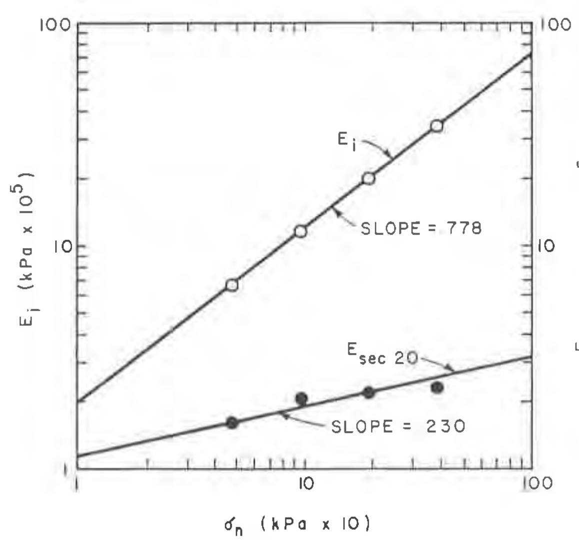

Fig. 8 Log E_i and log E_{sec20} versus log σ_n for Polyfilter X (woven fabric); cover and support material = dry #30 Ottawa sand.

Polyfilter X fabric. In addition, the secant moduli (E_{sec}), of the stress-strain curves at 20% strain were computed and plotted versus their corresponding normal stresses. Two straight lines were obtained: one for E_i, and one for E_{sec20}. These straight lines indicate that E_i and E_{sec20} increase linearly on log-log paper with increasing normal stresses. If the slope of the E_i line and its y-intercept are called q and k, respectively, the following equation can be written:

$$\text{Log } E_i = q \log \sigma_n + \log k \qquad (1)$$

This equation is solved for initial tangent modulus to yield:

$$E_i = k(\sigma_n)^q \qquad (2)$$

To obtain a better definition of E_i, ε/σ_T was plotted versus ε as shown in Fig. 9 for Polyfilter X and for σ_n = 47.9 kPa (0.5 tsf) and σ_n = 383.2 kPa (4 tsf). The inverse of ε/σ_T at the intercept gives the initial tangent modulus for its corresponding normal stress. For normal stresses between 47.9 kPa and 383.2 kPa, linear interpolation between the straight lines shown in Fig. 9 is valid to find the E_i value. A break in the straight line curve exists at small strain levels for both normal stresses. Therefore, each curve is

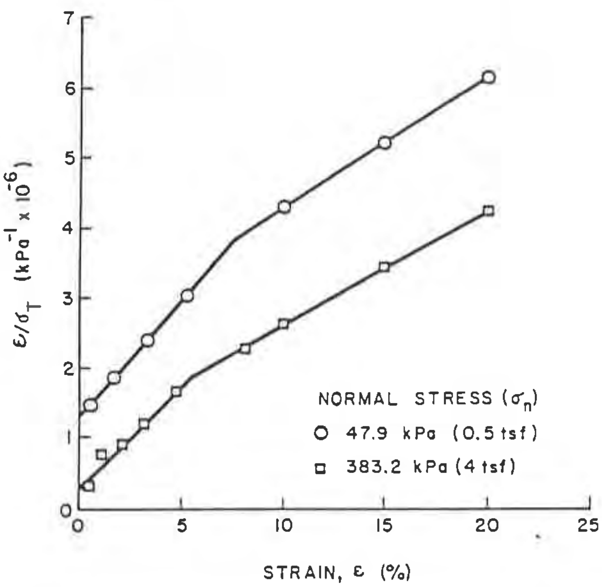

Fig. 9 ε/σ_T versus ε for Polyfilter X (woven fabric); support and cover material = dry #30 Ottawa sand.

approximated by two straight line segments with different slopes and y-intercepts. The slopes and y-intercepts of the straight lines are represented in Table 2 for Polyfilter X. Values for these parameters could be obtained for any combination of geotextile and cover/support materials by using the same procedure as outlined above. The general equation of any straight line on the axes shown in Fig. 9 is:

$$\frac{\varepsilon}{\sigma_T} = m\varepsilon + b \qquad (3)$$

where m and b are the slope and y-intercept, respectively.

Table 2

Slope (m) and y-intercept (b) values for the straight lines of Figure 9.

σ_n(kPa)	ε	m (kPa^{-1})	b (kPa^{-1})
47.9	$0 < \varepsilon < .075$	31.90×10^{-6}	1.31×10^{-6}
	$.075 < \varepsilon < .20$	18.13×10^{-6}	2.32×10^{-6}
383.2	$0 < \varepsilon < .05$	29.01×10^{-6}	$.29 \times 10^{-6}$
	$.05 < \varepsilon < .20$	15.95×10^{-6}	1.02×10^{-6}

Solving Equation 3 for σ_T, gives:

$$\sigma_T = \frac{\varepsilon}{m\varepsilon + b} \qquad (4)$$

which is the general equation for the stress-strain curves of Fig. 6. Since the tangent modulus (E_t) is the derivative of σ_T with respect to ε, Equation 4 may be expressed as follows:

$$E_t = \frac{d\sigma_T}{d\varepsilon} = \frac{m\varepsilon + b - m\varepsilon}{(m\varepsilon + b)^2} = \frac{b}{(m\varepsilon + b)^2} \qquad (5)$$

But at $\varepsilon = 0$, $E_t = E_i = \frac{1}{b}$

and from Equation 2: $E_i = k(\sigma_n)^q$

$$\text{Therefore } b = \frac{1}{E_i} = \frac{1}{k(\sigma_n)^q} = \frac{1}{k}(\sigma_n)^{-q} \qquad (6)$$

This expression for b, when substituted into Equation 5, yields the following general expression for E_t as a function of applied normal stress and strain level:

$$E_t = \frac{\frac{1}{k}(\sigma_n)^{-q}}{(\frac{1}{k}(\sigma_n)^{-q} + m\varepsilon)^2} \qquad (7)$$

Equation 7 relates the mechanical properties of the geotextile with the normal stress under which the geotextile is embedded; therefore, it is useful for design analysis in the soil-geotextile reinforcement systems in which the geotextile becomes an interactive stress-carrying component of the system. Simple laboratory tests such as those described previously can be used to determine the constants K and q for various combinations of cover and support soils-geotextile type and moisture conditions.

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Creep Characteristics and Stress-Strain Behavior of a Geotextile-Reinforced Sand

La fluage et le comportement contrainte-déformation de sable renforcé par des géotextiles

Laboratory investigations evaluated the creep behavior of woven and nonwoven geotextile-reinforced triaxial test samples. Short-term tests were performed on samples reinforced with evenly spaced horizontal circular disks of three common geotextiles to obtain stress-strain relationships. Long-term creep tests were conducted on similar specimens subjected to a constant load of 90% of the corresponding short-term ultimate strength. It was found that the inclusion of fabric reinforcement markedly increased both the maximum principal stress difference and the initial deformation modulus. Specimens reinforced with nonwoven geotextiles exhibit creep behavior similar to specimens reinforced with woven geotextiles.

Des essais de laboratoire ont été effectués pour évaluer la réponse au fluage de sable renforcés par des geotextiles. Des échantillons de sable dense renforcés par des disques horizontaux, également séparés, et faits de géotextiles communs, tissés et nontissés, ont été soumis à la compression dans l'appareil triaxial. Des échantillons similaires ont été soumis à des essais de fluage sous une charge égale à 90% de la resistance maximale immédiate. L'insertion de l'armature de renforcement a accru singulièrement l'écart maximum entre les contraintes principales et le module initial de deformation. Les échantillons avec armature nontissée ont montré une résponse au fluage similaire à celle des échantillons renforcés avec une armature tissée.

INTRODUCTION

The application of geotextiles to permanent construction requires that fabric properties be sufficiently stable to permit acceptable performance of the structure throughout its design life. However, there are many uncertainties concerning the long-term reliability of geotextiles, particularly their resistance to sustained loading. Knowledge of the long-term behavior of geotextiles is essential for the safe and economic design of fabric-reinforced retaining structures and embankments as well as fabric containment systems.

There is little published information on the creep characteristics of geotextiles in typical geotechnical environments. Studies have been conducted to determine the creep properties of both fibers (1) and fabrics (2, 3), but they have not included the effects of soil-reinforcement interaction. It is recognized that geotextiles tested in isolation may exhibit different behavior than geotextiles tested in soils (4). Koerner et al. (5) proposed a tentative design procedure to consider the creep of fabric-reinforced cohesive soils.

The objective of this study was to examine the response of woven and nonwoven geotextiles confined in dense sand when subjected to short-term and long-term loadings. A conventional triaxial test configuration was chosen for simplicity. Therefore, the tests provide only a qualitative indicator of in-situ geotextile stress-strain-time behavior.

TEST PROCEDURES

Laboratory investigations were conducted in two phases. An initial series of short-term triaxial tests was followed by a second series of long-term or creep tests. Samples were typically 36 mm in diameter and 73 mm in height. They were composed of a dry sand compacted to approximately 90% relative density (D_r) by tamping. Unreinforced control samples were used in both test series.

The reinforced specimens were constructed with circular fabric disks placed horizontally at the upper and lower third points and on the top and bottom platens. The diameter of the disks was the same as the triaxial samples. Similar tests have been conducted by Broms (6) and Schlosser and Long (7).

Sand

The soil was an oven dried, fine to medium, poorly-graded, angular sand with a trace of fines. This material is locally known as Lafayette Concrete Sand and is classified SP. Typical properties are shown in Table I.

Table I. Properties of Lafayette Concrete Sand

D_{10}	0.25 mm
C_u	2.36
ρ_s	2.70 Mg/m^3
e_{max}	0.70
e_{min}	0.37
ϕ_{triax}	46^o at D_r = 90%

Geotextiles

Three common geotextiles, one woven and two nonwovens, were selected for testing. A comparison of fabric properties is shown in Table II.

Table II. Properties of Geotextiles Tested*

Item	Supac 5-P	Mirafi 140S	Mirafi 500X
Manufacturer	Phillips	Celanese	Celanese
Composition	Nonwoven	Nonwoven	Woven
Fiber Type	Polypropylene	Polypropylene, Polyethylene	Polypropylene
Process Type	Needle-punched, heat fused	Melt Bonded	Slit film
Unit Weight (g/m^2) ASTM D-1910	180	140	136
Thickness (mm) ASTM D-1777	1.3	0.8	0.6
Grab Tensile Strength (N) ASTM D-1682	667	556	890
Grab Tensile Elongation (%) ASTM D-1682	80	65	24

*Source: Manufacturer's Literature

Short Term Tests

These tests were conducted to determine the short term stress-strain relationships and the strengths to be used for the subsequent creep tests. All tests were consolidated-drained (CD) on dry samples using air as the cell fluid. Confining pressures of 35 kPa, 69 kPa, and 276 kPa were chosen to simulate loadings in small to moderate embankments and walls.

A conventional loading press with proving ring was used to apply the axial stress at a constant rate of strain. Failure was defined as the maximum principal stress difference. Since volumetric strain of the dry specimens could not be measured, the principal stress difference was calculated using the initial specimen cross sectional area.

Long Term Tests

The long-term triaxial tests were conducted to determine the creep behavior of various geotextile-reinforced samples. The long-term test samples were constructed in the same manner as the short-term test samples.

A sustained, axial compressive stress was applied by a hangar and weight system on a loading frame. Confining pressures of 35 kPa and 69 kPa were maintained by air pressure regulators arranged in parallel. The maximum capacity of the hangar system did not permit creep tests to be conducted at higher confining pressures. The magnitude of the applied principal stress difference was 90% of the maximum principal stress difference determined from corresponding short-term tests. After creep loading of about 35 days, the samples were loaded rapidly to failure.

RESULTS

Short Term Tests

The reinforcing effect of inclusions on the short-term behavior of sands has been well documented (6,7,8, 9). The introduction of extensible inclusions (geotextiles) into a geotechnical environment will inhibit the development of soil tensile strains, thus producing a composite material of greater strength and modulus than an unreinforced soil at the same deformation.

The results of the short-term testing program are shown in Table III. The stress-strain relationships for fabric-reinforced and unreinforced specimens are shown by Figs. 1, 2, and 3. It can be seen that the geotextile inclusions markedly increased the maximum principal stress difference and the initial deformation modulus (E_i). However, at higher confining pressures the initial tangent modulus was found to decrease. The average increase in maximum principal stress difference due to reinforcing also decreased with higher confining pressures (211% increase at 35 kPa, 100% increase at 69 kPa, and 63% increase at 276 kPa). In addition, the fabric-reinforced samples exhibited a larger axial strain at failure (ε_f) than corresponding unreinforced samples. Consistent with the increase in maximum principal stress difference due to reinforcing, the Mohr-Coulomb relationships show a corresponding increase in the angle of internal friction (ϕ) from 46^o to 54^o, at a zero intercept (c' = 0).

Table III. Summary of Short-Term Triaxial Test Results for Various Reinforcement Materials (D_r = 90%)

Reinforcement	Type	σ_3 (kPa)	$(\sigma_1-\sigma_3)_f$ (kPa)	ε_f (%)	E_i (kPa)
Unreinforced	--	35	183	3.4	11,000
Supac 5-P	Nonwoven	35	464	4.6	16,500
Mirafi 140S	Nonwoven	35	646	4.8	23,500
Mirafi 500X	Woven	35	597	2.9	33,500
Unreinforced	--	69	376	3.8	18,000
Supac 5-P	Nonwoven	69	733	5.0	21,500
Mirafi 140S	Nonwoven	69	754	7.4	18,000
Mirafi 500X	Woven	69	762	7.4	19,000
Unreinforced	--	276	1401	6.6	92,000
Supac 5-P	Nonwoven	276	2180	8.4	64,000
Mirafi 140S	Nonwoven	276	2183	18	42,000
Mirafi 500X	Woven	276	2503	16	50,000

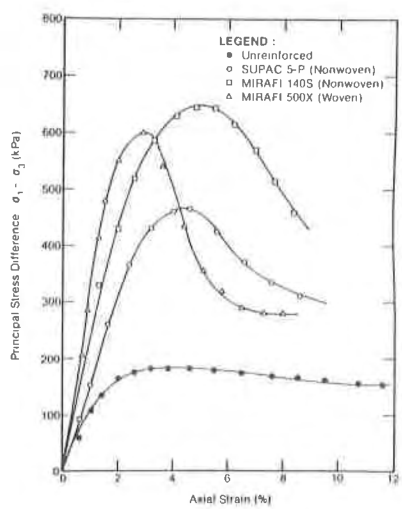

Figure 1. Stress-Strain Relationships for Various Reinforcement Materials (σ_3 = 35 kPa, D_r = 90%)

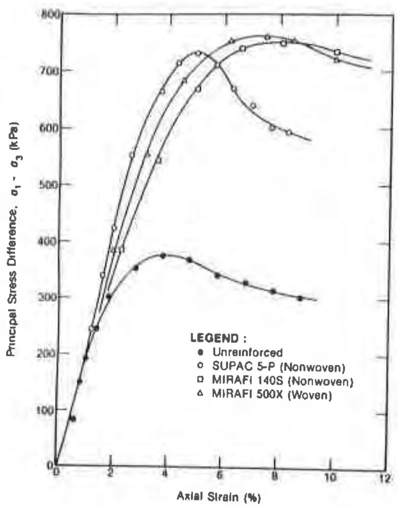

Figure 2. Stress-Strain Relationships for Various Reinforcement Materials (σ_3 = 69 kPa, D_r = 90%)

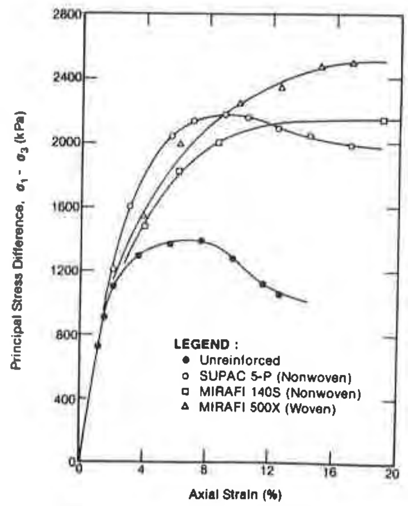

Figure 3. Stress-Strain Relationships for Various Reinforcement Materials (σ_3 = 276 kPa, D_r = 90%)

Long Term Tests

The results of the long-term testing program are shown in Table IV and Figs. 4 and 5. Consistent with the short-term testing behavior, it was found that at high relative densities (e.g., D_r = 90%), differences in geotextile properties did not greatly affect creep behavior.

Table IV. Summary of Long-Term Triaxial Creep Test Results for Various Reinforcement Materials (D_r = 90%)

Reinforcement		σ_3 (kPa)	$(\sigma_1-\sigma_3)_f$ (kPa)	Initial Strain (%)	Creep Strain (%)	Failure Strain (%)	Creep Duration (Days)
Unreinforced	--	35	231	0.7	0.1	1.1	35
Supac 5-P	Nonwoven	35	580	2.2	0.8	4.9	35
Unreinforced	--	69	535	1.0	0.05	2.1	35
Supac 5-P	Nonwoven	69	958	1.9	0.8	7.2	35
Mirafi 140S	Nonwoven	69	>1000	5.0	1.8	>10	42
Mirafi 500X	Woven	69	996	2.5	0.85	6.7	32

When fabric-reinforced samples are subjected to sustained loading conditions, the strain response consists of an immediate or "elastic" strain followed by a time-dependent creep strain. Figs. 4 and 5 show the creep strain versus time for reinforced and unreinforced specimens at two different confining pressures. Approximately 70% of the total creep strain occurred in less than three days of sustained loading. In all cases the observed creep response resulted in a nonlinear creep curve when plotted as a function of logarithmic time. Finnigan (1) and Van Leeuwen (2) reported a linear creep response when fabrics were tested in isolation.

Although the geotextiles tested had different properties, the stress-strain behavior (Figs. 1-3) for the majority of the specimens was very similar. In fact, it appears that at high relative densities (e.g., D_r = 90%), fabric properties do not greatly influence the behavior of reinforced sands. McGown and Andrawes (10) reported that at low initial placement densities, the improvement derived from fabric inclusions was greater than observed at high densities.

The effect of soil density influencing sample behavior becomes further apparent when considering the mode of sample failure. Hausmann and Vagneron (11) stated that an increase in the angle of internal friction, as a result of the addition of reinforcement to a sample, would indicate a failure controlled by slippage along the fabric surface. Conversely, failure controlled by rupture of the fabric material would result in an apparent cohesion (c'). Results of the short-term testing program indicate that failure of the reinforced samples was caused by particle slippage at the sand fabric interface. A slight bulging of the specimens occurred between reinforcing layers. Examination of the fabric disks after failure showed no evidence of ruptured filaments. It is believed that at high relative densities, soil-fabric frictional properties have the greatest influence on reinforced soil systems.

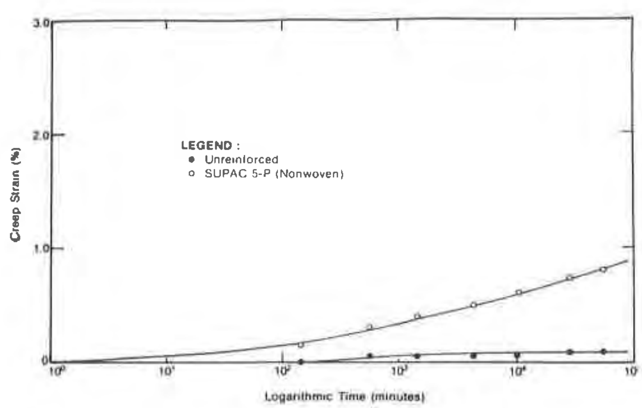

Figure 4. Creep Strain Relationships for Fabric-Reinforced and Unreinforced Triaxial Specimens (σ_3 = 35 kPa, D_r = 90%)

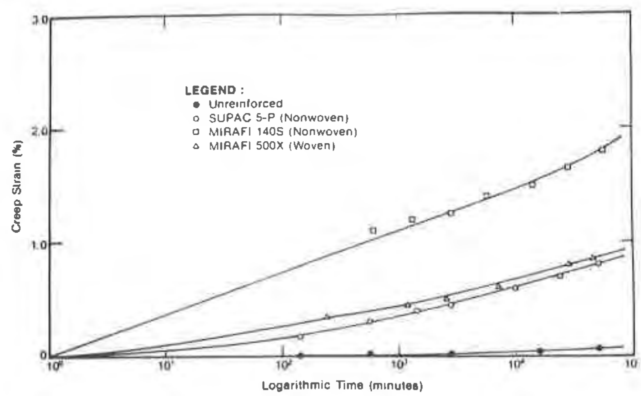

Figure 5. Creep Strain Relationships for Fabric-Reinforced and Unreinforced Triaxial Specimens (σ_3 = 69 kPa, D_r = 90%)

The long-term test results showed no significant differences in creep behavior due to confining pressure or geotextile properties. As expected, the lighter weight nonwoven (Mirafi 140S) exhibited a somewhat greater creep response than the heavier nonwoven (Supac 5-P). However, the woven geotextile (Mirafi 500X) showed a greater creep response than the heavier nonwoven (Supac 5-P). This result verifies the importance of soil-fabric frictional properties. It is possible that quite different behavior would be observed for fabric-reinforced samples at lower relative densities where fabric properties are likely to have a greater influence (8).

At the conclusion of creep testing, the samples were rapidly loaded to failure to evaluate stress-strain characteristics after creep loading. It was observed that the long-term stress-strain behavior resulted in an increase in strength (about 30%) and tangent modulus when compared to corresponding short-term tests. Similar results are reported by Haliburton, et al. (3). This behavior was exhibited by both unreinforced and reinforced samples.

Consistent with the short-term tests, all of the samples tended to bulge slightly between reinforcing layers. After testing, the fabric disks were examined and no evidence of ruptured filaments was observed. However, the nonwoven fabrics appeared to be "dished" in the center area of the disks. No "dishing" was noted with any of the woven fabrics although some scratches and striations were evident.

CONCLUSIONS

1. The inclusion of geotextile reinforcement increases the ultimate strength, deformation modulus, and the angle of internal friction of triaxial samples composed of dense, angular sand.
2. The increase in short-term maximum principal stress difference for geotextile-reinforced triaxial test samples of a dense, angular sand decreases with increasing confining pressure.
3. Geotextile-reinforced sands exhibit a larger axial strain at failure and more creep than corresponding unreinforced samples.
4. Approximately 70% of the total creep strain occurs within 3 days of sustained loading.
5. At high relative densities (D_r = 90%) geotextile properties do not appear to greatly influence the stress-strain or creep behavior of reinforced samples.
6. At confining pressures of 35 and 69 kPa, failure for both long-term and short-term samples is apparently controlled by slippage along the soil-geotextile interface.

ACKNOWLEDGEMENTS

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An Evaluation of Abrasion Tests for Geotextiles

Une évaluation de tests d'abrasion de géotextiles

Resistance to impact and wear abrasion are important properties of geotextiles used in railroad bed construction. A program was designed to compare the standard textile abrasion test (ASTM D1175) with two new abrasion tests designed to simulate impact and wear abrasion of geotextiles in railbeds. Eleven different geotextiles were tested. A scanning electron microscope was used to assess the abrasion processes and severity caused by the three tests. The ASTM D1175 test was found not to simulate abrasion of geotextiles in the railbed. This test did not perform well for non-woven, non-resin dipped fabrics. The Geotextile - Ballast Impact Abrasion Test and the Geotextile - Aggregate Repeated Loading Test did simulate railbed abrasion and performed well on both woven and non-woven geotextiles. In the former test, non-woven geotextiles were more resistant than woven fabrics. In the latter test, woven and non-woven geotextiles abraded approximately the same amount, albeit by different processes. The thicker and heavier fabrics were more abrasion resistant.

La résistance aux impacts et à l'abrasion sont deux propriétés importantes des géotextiles utilisés dans les ballastes ferroviaires. Des essais ont e'té entrepris pour comparer le test ordinaire d'abrasion (ASTM D1175) à deux nouveaux tests d'abrasion permettant la simulation d'impacts et d'abrasion de géotextiles dans les ballastes ferroviaires. Onze géotextiles différents ont e'té soumis aux tests. Un examen, au microscope électronique à balayage, du degré d'abrasion causé par chacun des tests révèle que le test ASTM D1175 ne reproduit pas les conditions d'abrasion dans les ballastes ferroviaires. Ce test n'est pas satisfaisant pour les textiles non tissés et non enduits de résine. Le test d'abrasion due aux impacts et le test de charge répétée reproduisent les conditions d'abrasion dans un ballaste, et donnent de bons résultats sur les géotextiles tissés et non tissés. Le premier test re'vèle que les géotextiles non tissés sont plus résistants que les géotextiles tissés. Dans le second test, les géotextiles tissés et non tissés souffrent d'abrasions égales, mais issues de processus différents. Les textiles épais et lourds résistent mieux à l'abrasion.

INTRODUCTION

Two properties of a geotextile, which are very important to its successful application, are resistance to impact abrasion and resistance to wear abrasion. Nowhere are these properties as critical as when the geotextile is used in the construction or rehabilitation of a railroad bed. During construction or rehabilitation, crushed rock (ballast) is literally dropped onto the geotextile. During normal operation of the railroad, repetitive wheel loads impart a rubbing motion between the ballast and the geotextile and, between the geotextile and the subballast below the geotextile. Excessive abrasion caused by either impact or wear reduces the filter, separator and drainage capabilities of the geotextile. This condition can jeopardize the integrity of the entire railroad structure.

In testing the abrasion resistance of geotextiles it is important to simulate the actual type of abrasion. It was felt that the "Standard Methods of Test for Abrasion Resistance of Textile Fabrics" (ASTM D1175) may not simulate the in-field impact or wear abrasion that geotextiles might undergo in the railbed situation. Therefore, a study program was designed to evaluate a test procedure of ASTM D1175 against two abrasion tests specifically developed to simulate impact and wear abrasion in railbed construction and rehabilitation.

The study was supported in part by a Natural Sciences and Engineering Research Council of Canada Grant, and in part by the Canadian Institute of Guided Ground Transport. The study was carried out at Queen's University at Kingston, Ontario, Canada. The writers acknowledge the assitance of Mr. D. Kempson, Mr. H. Verstappen, Mr. C. Peck, and the geotextile manufacturers who supplied samples of their products.

TESTING PROGRAM

The three abrasion tests used to test the geotextiles, were:

(i) the Rotary Platform, Double Head Procedure of ASTM Test Method D1175: Standard Methods of Test for Abrasion Resistance of Textile Fabric (ASTM, 1971) (the Rotary Abrasion Test);
(ii) the Geotextile-Ballast Impact Abrasion Test (the Ballast Impact Test); and
(iii) the Geotextile-Aggregate Repeated Loading Abrasion Test (The Repeated Loading Test).

The Rotary Platform, Double Head Procedure of ASTM D1175 is the standard method to evaluate the wear abrasion resistance of any textile subject to "rotary rubbing action under controlled conditions of pressure and abrasion action" (ASTM, 1971). Figure 1 shows the standard rotary abrasion tester during an actual test. For this test program two rubber based CS-17 Calibrase abrasive wheels, and a load of 1000 gms were used to abrade the geotextile samples for 10, 100, and 1000 cycles. The test is fully described in the reference and therefore is not described further.

A Geotextile Ballast Impact Abrasion Test was developed to simulate the impact abrasion caused by railway ballast placement on a geotextile. The apparatus for this test is shown in Figure 2. A graded material, passing the 38 mm sieve and retained on the 19 mm sieve, and representing AREA ballast grading No. 4 (American Railway Engineering Association, 1981) was placed and compacted in a wooden container 250 mm square by 75 mm high. A sample of the geotextile to be tested, large enough to cover the surface of the wooden container, was clamped to the sides of the container. An aluminum tube, 150 mm in diameter and 1 metre in length, was placed vertically over the container and geotextile. Approximately 5.0 kg of AREA ballast grading No. 4 was placed within another shorter aluminum tube above a removable trap door which was located exactly 1 metre above the geotextile. The trap door was quickly removed to allow the ballast to impact onto the geotextile. Samples were tested at 5 drops and 10 drops of ballast.

A Geotextile-Aggregate Repeated Loading Abrasion Test was also developed. This test was designed to simulate the wear abrasion caused by aggregate (ballast or subballast) on a geotextile used as a separation layer in the railbed. The apparatus for this test is shown in Figure 3. An aluminum cylinder, 250 mm in diameter, 300 mm high and closed at one end, was used. Standard Ottawa Sand, passing a 0.84 mm sieve and retained on a 0.59 mm sieve, was used to fill the bottom 50 mm of the cylinder.

Two tests were conducted simultaneously. One disc of the geotextile to be tested was placed on top of the sand and covered with 65 mm of ballast. This represented wear abrasion of the geotextile placed under ballast and sitting on a sand subballast or subgrade. A second disc of the same geotextile was placed on the previously mentioned lift of ballast and covered by a second 65 mm lift of ballast. This simulated the wear abrasion of the geotextile between two ballast lifts, or between ballast and subballast. The ballast used for both lifts was AREA grading No. 4. A repeated loading, rigid footing device, 100 mm in diameter, was clamped in place with a size 12 diaphragm air cylinder (Bellofram Products Company). The footing was then repeatedly loaded to 290 kPa and then fully unloaded. Three series of tests were performed at 1000, 10,000 and 100,000 cycles.

Eleven geotextiles were tested in this study. The selection of these samples was based upon the method of manufacture (woven vs. non-woven), type of polymer, filament characteristic, mass and thickness. It was attempted to study as broad a cross section of geotextiles types as possible. A complete description of the geotextiles tested is given in Table 1a.

A scanning electron microscope (SEM) was used to assess the abrasion of the geotextiles by comparing the abraded material with an unabraded control sample of each geotextile. Standard SEM procedures were followed (Hearle et al., 1972) to prepare a 10 mm^2 sample of each of the unabraded and abraded geotextiles. The samples were studied at 20x and 100x magnification, and photomicrographs were taken. All photomicrographs were taken at a stage angle of 45° to the scanning electron beam, since the best resolution was achieved at this angle. Although 20x and 100x magnification is very low for the SEM some experimental work by the writers showed that

Fig. 1 ASTM Rotary Platform, Double Head Abrasion Tester.

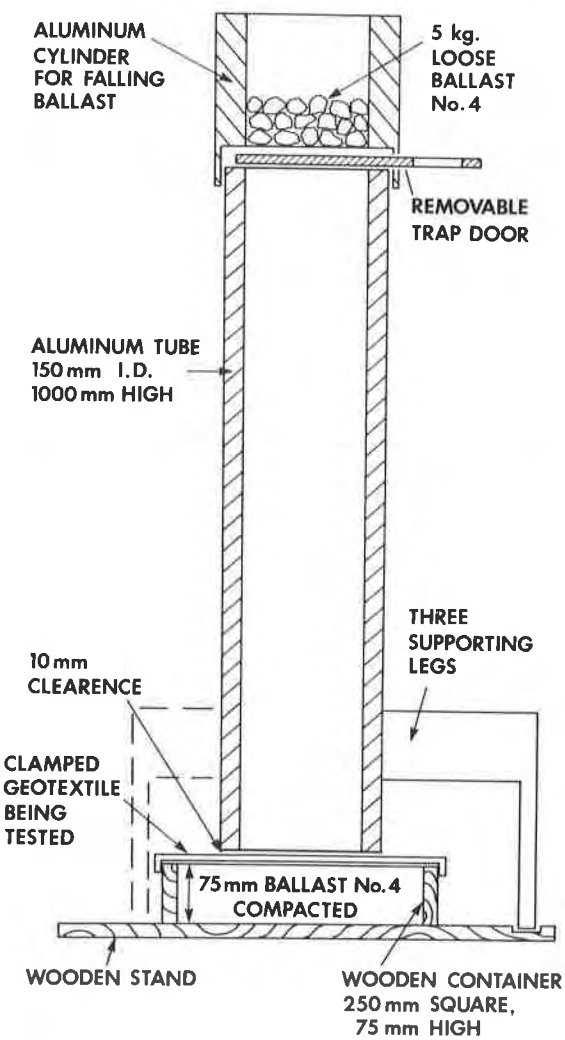

Fig. 2 Sketch of Geotextile-Ballast Impact Abrasion Test Apparatus.

resolution, depth of field and overall clarity was much greater with the SEM than with conventional microscopes.

RESULTS

From the testing program, a very large amount of qualitative and subjective information was obtained. The writers have attempted to remove some of the subjectivity of assessing the abrasion, and to simplify the results by only using certain modifiers to describe the processes of abrasion, and ranking those modifiers as minor or major. A listing and description of those modifiers used is given in Table 2. Table 1b describes the processes of abrasion that the eleven geotextiles underwent during the three abrasion tests. Figures 4 to 7 are photomicrographs that show the construction and quality of typical unabraded geotextiles, and various types and degrees of severity of abrasion.

Comparison of abrasion tests

The Rotary Platform, Double Head Procedure of ASTM D1175 was found to wear abrade the woven geotextiles and the resin dipped non-woven geotextiles (geotextiles I and K) well, but in a fashion foreign to geotextiles in a railbed. When the Rotary Abrasion Test was applied to the other non-resin dipped, non-woven geotextiles, the filaments tended to be pushed out of the way of the abrading wheels rather than undergoing abrasion. In many instances, the abrading wheels became clogged with the polymer of the geotextile being abraded, after more than 1000 cycles.

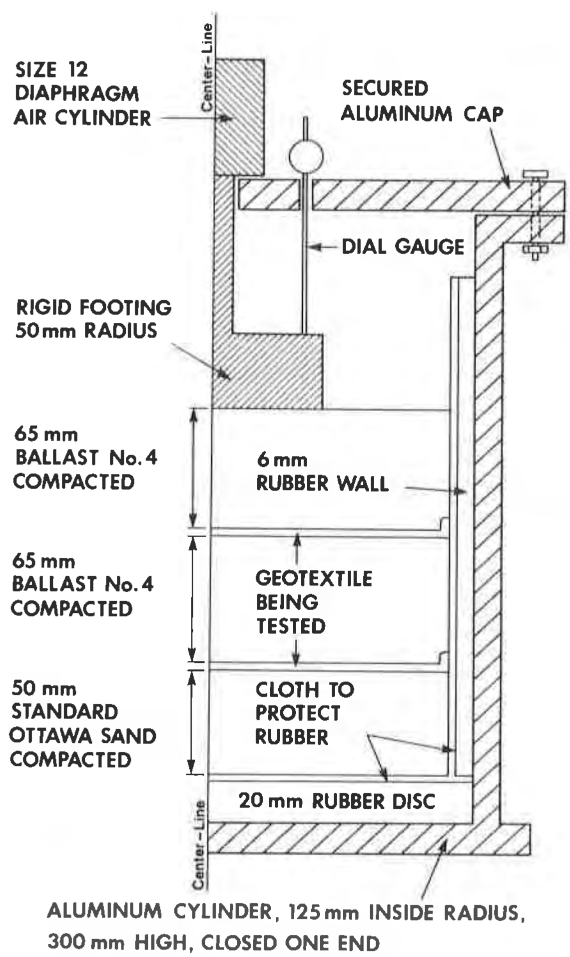

Fig. 3 Sketch of Geotextile-Aggregate Repeated Loading Abrasion Test Apparatus.

The Geotextile-Ballast Impact Abrasion Test was found to perform satisfactorily and simulate the infield condition of ballast emplacement. In most cases, ten drops of ballast were sufficient to cause substantial impact abrasion, and in some cases caused puncturing of the geotextile.

The Geotextile-Aggregate Repeated Loading Abrasion Test also performed very well and simulated in-track repeated loading very well. The ballast-geotextile-ballast test provided substantial wear abrasion to the geotextiles after 100,000 cycles. The ballast-geotextile-sand test seldom abraded the geotextile to any degree and therefore the results are not discussed further. Minor problems were encountered with the cycle counter and the loading cell, however these were minor technical flaws which posed no major problems to the testing program.

Comparison of geotextiles

Woven - The woven geotextiles were found to wear abrade by the process of peeling when subjected to the Rotary Abrasion Test. The amount of abrasion after 1000 cycles was approximately equal for all four woven samples (Table 1b).

After undergoing 10 ballast drops of the Ballast Impact Test, the woven geotextiles frequently showed evidence of peeling, splitting, and being cut. A minor amount of slippage of the filaments was also noted. Geotextile B proved to be the most resistant to impact abrasion while Geotextile D was the least resistant; the multifilaments having been cut all the way through. Geotextiles A and C were abraded to approximately the same degree and to an extent between the Geotextiles B and D.

The most frequent processes involved with wear abrasion of woven geotextiles by the Repeated Loading Test were also peeling, splitting, and being cut. Slipping and flattening of the filaments occurred on two of the samples. Three of the woven geotextile samples were punctured after 100,000 cycles of loading. Geotextile B was not punctured and sustained less abrasion than the other three.

Non-woven - The most frequent processes of wear abrasion, of the non-woven geotextiles having undergone the Rotary Abrasion Test, were peeling and alignment of filaments. Geotextile H was cut and punctured after 1000 cycles. Geotextiles I and K showed evidence of major peeling while the remaining samples showed only minor amounts of wear abrasion.

The common impact abrasion processes seen after 10 ballast drops of the Ballast Impact Test were peeling and being cut. Less frequently the filaments were flattened and fused. Geotextiles G and H were found to abrade the most, and Geotextile G was the only sample punctured by this test. The remaining 5 samples showed approximately the same amount of impact abrasion.

Peeling, flattening, clumping and being cut were the most common abrasion processes noticed after the Repeated Loading Test. Fusing occurred in three of the seven samples. Geotextile H was the least resistant to wear abrasion. Geotextile I was punctured after only 10,000 cycles. Geotextiles F and G were in the mid-

TABLE 1a: PROPERTIES OF GEOTEXTILES — TABLE 1b: TEST RESULTS

GEO-TEXTILE	DESCRIPTION	MASS g/m^2	THICK-NESS mm	ROTARY ABRASION TEST (1000 cycles)	BALLAST IMPACT TEST (10 drops)	REPEATED LOADING TEST (ballast-geotextile-ballast) (100K cycles - except where noted)
A	Woven, polypropylene, flat monofilaments, black	203	0.7	Pe^1	Pe^2, Sp^1, Cu^1	Pe^1, Sp^1, Cu^2, Pu^2
B	Woven, polypropylene, flat multifilaments, black	730	2.3	Pe^1	Pe^1	Pe^2, Cu^2
C	Woven, polypropylene, flat monofilaments woven with round monofilaments, black	271	0.5	Pe^1, Fl^1	Pe^2, Sp^1, Cu^1, Sl^1	Pe^1, Fl^1, Cu^2, Pu^2
D	Woven, rounded polyethylene monofilaments woven with flat polypropylene multifilaments, black	250	0.6	rounded Pe^2 flat Fe^1	Pe^1, Sl^1, Cu^2, Pu^2 Pe^1, Cu^2, Pu^2	Pe^1, Sl^1, Cu^2, Pu^2, Pe^1, Cu^2, Pu^2
E	Non-woven, needle punched polyester mat with polypropylene scrim, fibre length perpendicular to plane of fabric, white	475	4.0	Al^1, Se^1, Pe^1	Se^1	Pe^1, Fl^1, Cl^1, Cu^1 (10K cycles)
F	Non-woven, continuous spun bonded polypropylene monofilaments, heat set finish, grey	203	0.6	Pe^1, Cu^2	Pe^1, Cu^1	Pe^1, Fl^1, Cl^1, Cu^2, Pu^1
G	Non-woven, continuous needle punched polyester monofilaments, grey	198	1.0	Pe^1	Pe^1, Fl^1, Cu^2, Pu^1	Pe^1, Fl^1, Cl^1, Cu^2, Pu^1
H	Non-woven, continuous spun bonded polypropylene monofilaments encased in nylon sheath and heat bonded, white	137	0.8	Pe^1, Cu^2, Pu^2	Pe^1, Fl^1, Cu^2	Pe^1, Fl^2, Fu^1, Cu^2, Pu^2
I	Non-woven, continuous needle punched, double bonded polyester monofilaments, partially resin treated, blue and white	455	3.3	Al^1, Pe^2	Pe^1	Pe^1, Fl^1, Fu^1, Cu^2, Pu^2 (10K cycles)
J	Non-woven, continuous needle punched polyester monofilaments, grey	415	2.9	Al^1, Pe^1	Pe^1	Pe^1, Fl^1, Fu^1, Cu^2, Pu^1
K	Non-woven, continuous needle punched nylon with polypropylene scrim, resin dipped, black	550	4.5	Al^1, Pe^2	Pe^1, Fl^1, Fu^1	Pe^1, Fl^1, Cl^1

TABLE 2 : DESCRIPTION OF ABRASION PROCESSES

PROCESS	SYMBOL	DESCRIPTION
Aligned	Al	Previously non-aligned filaments become somewhat aligned in a preferred direction. Limited to non-woven geotextiles.
Clumped	Cl	Individual filaments form a clump. Limited to non-woven geotextiles.
Cut	Cu	Individual filaments are first cracked and then cut in the transverse direction of the filament.
Flattened	Fl	The thickness of the individual filaments is reduced while the width is increased to produce flattening.
Fused	Fu	Individual filaments are fused together; usually accompanied by flattening.
Peeled	Pe	Small slivers of individual filaments are partially or totally peeled from parent material.
Punctured	Pu	Individual filaments are abraded by various processes and the geotextile develops a hole, thus becomes punctured.
Separated	Se	Individual filaments become separated. Limited to non-woven geotextiles.
Slipped	Sl	Individual filaments of the warp or weft slip along the other and move either closer together or further apart. Limited to woven geotextiles.
Split	Sp	Individual filaments are first cracked and then split in the longitudinal direction of the filament.

Superscript 1 : minor amount of abrasion
Superscript 2 : major amount of abrasion

Fig. 4 GEOTEXTILE A (scale bar is 500 microns)
(a) Unabraded
(b) Rotary Abraded (1000 cycles)
(c) Ballast Impact Abraded (10 drops)
(d) Repeated Loading Abraded (100,000 cycles)

Fig. 5 GEOTEXTILE D (scale bar is 500 microns)
(a) Unabraded
(b) Rotary Abraded (1000 cycles)
(c) Ballast-Impact Abraded (10 drops)
(d) Repeated Loading Abraded (100,000 cycles)

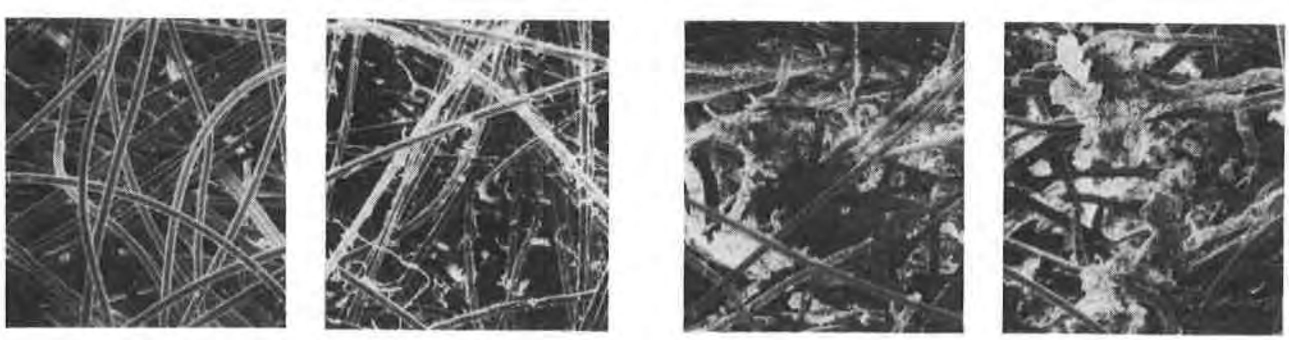

Fig. 6 GEOTEXTILE G (scale bar is 100 microns)
(a) Unabraded
(b) Rotary Abraded (1000 cycles)
(c) Ballast Impact Abraded (10 drops)
(d) Repeated Loading Abraded (100,000 cycles)

Fig. 7 GEOTEXTILE K (scale bar is 500 microns)
(a) Unabraded
(b) Rotary Abraded (1000 cycles)
(c) Ballast Impact Abraded (10 drops)
(d) Repeated Loading Abraded (100,000 cycles)

range of wear resistance. Geotextile J was slightly more resistant than Geotextile G. Geotextiles E and K showed least abrasion and were the only two which were not punctured. (The test on Geotextile E was only taken to 10,000 cycles before a failure in the apparatus stopped the test.)

Woven vs. Non-woven - Some general comments can be made with regards to the abrasion resistance of woven vs. non-woven geotextiles by these three tests. In the Ballast Impact Test the non-woven geotextiles appeared much more resistant to impact than the woven materials. The Repeated Loading Test was found to give the most consistent amount of abrasion to both types of materials. Both woven and non-woven geotextiles were found to abrade about the same amount, albeit by slightly different processses. As mentioned previously, because of the test technique associated with the Rotary Abrasion Test, no comparisons could be made between the woven and non-woven geotextiles for this test procedure.

CONCLUSIONS

A testing program involving three abrasion tests and eleven geotextiles (four woven and seven non-woven) has been carried out. Based upon the results of this study, the following conclusions can be made.

The Rotary Platform, Double Head Procedure of ASTM D1175 was found to wear abrade the woven geotextiles primarily by the process of peeling. However, the test did not simulate impact abrasion caused by ballast emplacement, or wear abrasion of geotextiles in ballast under repeated wheel loadings. This test did not perform well for non-resin dipped, non-woven fabrics.

The Geotextile - Ballast Impact Abrasion Test was found to simulate impact abrasion by the processes of peeling, splitting and being cut for woven material, and peeling and being cut for non-woven material. Ten ballast drops were found to be sufficient to cause substantial impact abrasion. In general, non-woven geotextiles were found to be much more resistant to impact abrasion than woven geotextiles. The thicker woven and non-woven geotextiles were generally more impact abrasion resistant.

The Geotextile - Aggregate Repeated Loading Abrasion Test, at the ballast-geotextile-ballast interface, simulated wear abrasion well. The common processes of abrasion shown by the woven geotextiles were peeling, splitting and being cut. Those for the non-woven geotextiles included peeling, flattening, clumping and being cut. One hundred thousand cycles was found optimum to provide substantial abrasion. For both the woven and non-woven materials, the thinner and lighter weight geotextiles showed less resistance to wear abrasion than the thicker and heavier geotextiles. The woven and non-woven materials were found to abrade approximately the same amount. The ballast-geotextile-sand test seldom abraded the geotextile to any significant degree for either the woven or non-woven geotextiles.

RECOMMENDATIONS

From the results of this study, the writers make the following recommendations:

(1) The Geotextile - Ballast Impact Abrasion Test and the Geotextile-Aggregate Repeating Loading Abrasion Test should be used to test the impact and wear abrasion, respectively, of geotextiles for railroad bed use.

(2) Further testing should be carried out so that the abrasion resistance of individual geotextiles can be assessed.

LIMITATIONS

This study was designed to evaluate three abrasion tests, and assess how they simulate impact and wear abrasion of geotextiles during construction and rehabilitation of railroad beds. The impact and wear abrasion of the eleven geotextiles were successfully assessed using qualitative and subjective evaluations of scanning electron microscope photomicrographs. This study should not be construed as a quantitative evaluation of the abrasion resistance of the individual geotextiles tested.

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Abrasion Resistance of Geotextile Fabrics
La résistance à l'abrasion des géotextiles

The resistance to abrasion of various Geotextile fabrics was measured on a purpose-built apparatus where the fabric was subjected to a vibrating ram the end of which consisted of aggregate set in resin. The resulting damage to the fabric was assessed first visually and then quantitatively using the California Bearing Ratio test. Repeat samples were assessed using the 50 mm strip tensile test. A wide range of resistance to abrasion was shown by tbe different fabrics. By modifying the apparatus other fabrics were subjected to vibrating steel cones, the object being to pierce the fabric. When this happened the cones came into contact with a bed of finely-divided metal thereby completing an electric circuit and causing a lamp to light. The times taken to do this for various fabrics were measures of their resistance to the vibrating cones. Finally a modified Martindale abrasion tester, with a roughened steel surface as abradant, was used to compare woven and non-woven fabrics.

La résistance à l'abrasion de plusieurs géotextiles fut mesurée sur un appareil spécialement developpé dans lequel le textile est sollicité par un plongeur vibrant dont la face se compose de pierres concasses fixées en résine. Le dommage au géotextile fut mesuré visuellement, dans l'essai CBR et dans l'essai a bande de 50 mm. La resistance à l'abrasion varie de geotextile en geotextile. Dans une modification de l'essai, des cônes en acier remplacent les pierres avec le but de percer le géotextile et de faire contact avec une couche de granules en acier située en dessous, ce qui allume une lampe par moyen d'un circuit électrique. Les délais des différents géotextiles sont une mesure de leur résistance à la pénétrations par les cônes vibrants. Enfin, avec une modification de l'essai d'abrasion Martindale utilisant une surface abradante en acier rugueuse, des comparaisons sont faites entre les geotextiles tissés et non-tissés.

1. Introduction

With the increasing variety of types of fabrics being used for Geotextile purposes it became important to develope tests which could assess the performance of these fabrics with regard to abrasion resistance, and in particular with the type of abrasion which the products would meet in the field. With this objective, three different abrasion tests were initiated, or modified, and used to test a range of fabrics designated for different end-uses.

2. Experimental

2.1 Simulated Vibrating Road

An apparatus was set up which could subject a fabric sample to the type of abrasion produced by aggregate when used on railway tracks or during the construction of roads. Using an epoxy resin, the appropriate aggregate was set into a metal cup which could be mounted on the end of a ram. The ram was vibrated in three different directions simultaneously, the directions being mutually at right angles. One direction was vertical, the other two horizontal. A circle of fabric 250 mm in diameter was cut out, clamped into the apparatus and subjected to the abrasion. The photographs, Figures (1) and (2), show the apparatus used.

Figure (1). Aggregate fixed in cup.

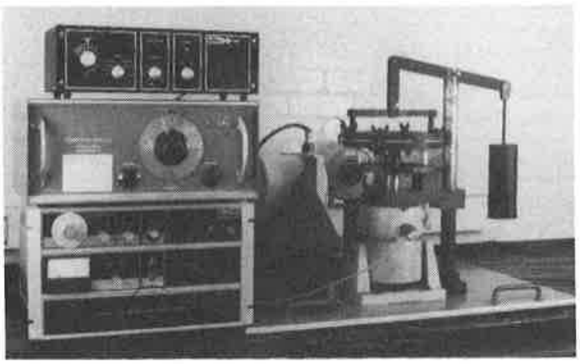

Figure (2). General View of simulated Road Abrasion apparatus.

Eight different fabrics were tested as follows:-

A. Continuous filament spun bonded Polypropylene.
B. Needle punched Staple Polypropylene.
C. Composite fabric, a needled Staple with a woven split-tape, all Polypropylene.
D. Continuous filament needle punched Polyester.
E. Needled and Thermic bonded Staple 95/5 Polypropylene/Polyester.
F. Continuous filament heat bonded 70/30 Polypropylene/Polyethylene.
G. Needled Staple Polyester.
H. Continuous filament spun bonded Polyester.

After preliminary trials to establish the correct degree of abrasion, the conditions of test used to give a reasonable comparison of these fabrics were as follows:-

Three planes of vibrations, each at 10 Hertz, Amplitude setting 2 (approximately 4 mm) vertical load 2.7 kg, time 1 hour. This equates to 36,000 vibrations. The Photograph shown in Figure 3 illustrates the abrasion produced on a sample of each of the eight different fabrics.

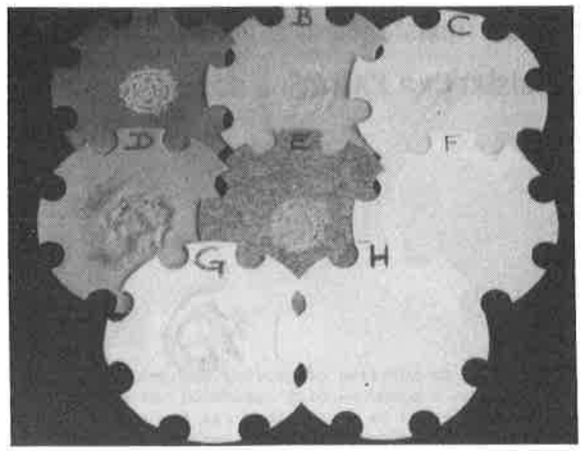

Figure (3). Abraded fabric samples.

To assess the strength of the fabrics after abrasion a bursting test was carried out on the abraded portion of the fabrics. The test used was the California Bearing Ration (CBR) test as used in the laboratory with a Tensile test instrument. In this test a steel ram of circular cross-section, diameter 50 mm, is forced against a fabric sample which is securely clamped. The force required to break the fabric is measured. With the circle of abrasion on the fabrics being 80 mm, the 50 mm diameter ram on the CBR test should give a measure of strength of the abraded portion of the fabric. The CBR breaking loads of non-abraded fabric samples were tested as controls. The results are shown in the following table (Figure 4), each result being the mean of three samples tested.

Figure (4). Table of Mean CBR Results.

(Each result is the mean of 3 samples tested and the means are normalized to a fabric weight of 200 g/m^2 so that they can be compared more easily.)

Fabric	Fabric Weight (g/m^2)	Normalized CBR (N/200 g/m^2)		% Loss
		Before Abrasion	After Abrasion	
A	200	1918	1084	43
B	355	1066	1010	5
C	315	1078	1034	4
D	425	1828	1640	10
E	430	1776	1310	26
F	270	2532	2420	4
G	395	830	412	50
H	216	1584	1460	8

An alternative way of assessing the strength of the fabrics before and after abrasion is to use the 50 mm wide Strip Tensile test. This is a standard test used frequently in Europe, mainly because it is used extensively for woven apparel fabrics. Normally it does not have much relevance to Geotextile fabrics because they are used in a much larger mass and hence localized tensions are supported by the surrounding mass of fabric. In this abrasion treatment, however, there is an abraded area of fabric of 80 mm diameter, hence a strip 50 mm wide cut through the centre of the abraded area and tested for tensile strength should be indicative of the extent of abrasion. Samples from each of the previously described eight Geotextile fabrics were abraded using the vibrating "aggregate" ram, and then a 50 mm wide strip was cut out from each sample. Each strip was tested for breaking load on a tensile tester using a gauge length of 180 mm and a cross-head speed of 200 mm per minute. Four samples from each of the eight fabrics were abraded and tested in this manner and the mean results are given in the following table (figure 5). Control tests were carried out on unabraded samples and the percentage loss in strength produced by the abrasion was calculated and is given in the table.

Figure (5). Table of Mean 50 mm wide Strip Tensile Results.

(Each result is the mean of 4 tests.)

Fabric	Fabric Weight (g/m^2)	Normalized 50 mm wide Strip Tensile Strength ($N/200\ g/m^2$)		% Loss
		Before Abrasion	After Abrasion	
A	200	581	283	51
B	355	383	308	20
C	315	298	114	62
D	240	500	500	0
E	430	415	410	1
F	270	511	481	6
G	395	259	221	19
H	216	546	409	25

2.1.1 Discussion on and Conclusions from 2.1.

The objective in this experiment was to simulate the abrasion produced on Geotextile fabrics by aggregate used for railways and during the laying of roads. The severity of the abrasion force applied by the equipment used, ie extent of amplitude of vibration, frequency and mean pressure applied, was selected so that the fabrics being assessed showed as wide a range of abrasion as possible and this is illustrated by the photograph of the eight fabric samples in figure 3. The aggregate used was set in the epoxy resin in such a way that a convex surface presented itself to the fabric to give as large an area of abrasion as possible.

Visual examination of the abraded fabrics showed substantial differences, with some severely abraded while others were affected very little. Visually, the fabrics most resistant to abrasion were F and H, both continuous filament products, the former heat bonded polypropylene/polyethylene, the latter spun bonded polyester. The CBR results shown in figure 4 indicate fabrics G, A and E to have suffered the greatest loss in strength the others relatively little loss. The strip tensile results given in figure 5 show five of the fabrics giving a much reduced strength, C, A, H, B and G. The other fabrics, D, E and F were affected very little. Overall, the continuous filament heat bonded polypropylene/polyethylene fabric (F) appeared to show most resistance to this type of abrasion, with fabrics A, C and G being most affected by the abrasion.

2.2 Vibrating Steel Cones

The reason for this type of abrasion test was that certain customers wanted to use Geotextile fabrics as protection layers for impermeable liners against the abrasion and puncturing by stone or rough concrete. A laboratory test was needed to determine which would be the best fabrics for their particular use. An apparatus similar to that used in experiment 3.1 was set up, but with the following differences:- the aggregate ram was replaced by a metal dish filled with "shattered" metal (small particles of steel, see figure 6), the fabric sample under test was clamped over the divided metal, and a circle of small steel cones was pressed against the fabric from above. The ram was vibrated vertically at a frequency of 10 Hertz with an amplitude of 5 mm and a load of 1 kilogram. An electrical circuit was made connecting the steel cones via a battery - and - lamp combination to the steel dish containing the divided metal. When the cones pierced the fabric they came into contact with the divided metal and completed the electrical circuit causing the lamp to light. The whole apparatus is shown in figure 7.

Figure (6). Metal Dish containing "shattered" metal.

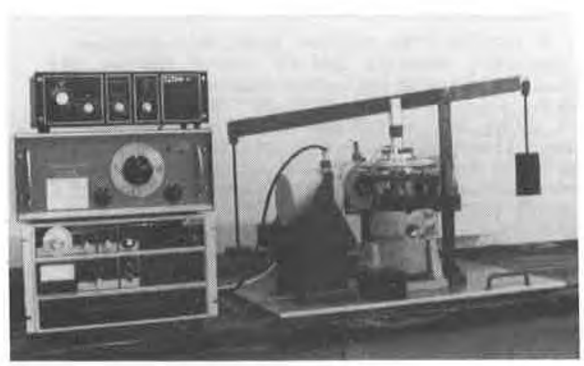

Figure (7). Vibrating Cones Apparatus.

For the initial experiments the times taken for the lamp to light for different fabrics were used as a measure of their resistance to this type of abrasion. However, some fabrics took a long time, hence to speed up the test each fabric was subjected to the above conditions of vibrating cones for two hours, after which the static load required for the fabric to be pierced was measured; the higher the load required, the higher the resistance of the fabric to this abrasion. Seven different fabrics were compared, five non-woven and two impermeable sheets. There were three polypropylene/polyethylene continuous filament melded fabrics of different weights, 100, 140 and 230 g/m^2, a high bulk c.f. polypropylene/polyethylene, a c.f. polyester and two polyvinyl chloride sheets of thickness 1.0 and 1.2 mm. The results are given in figure 8.

Figure (8). Table of Results from Vibrating Cones Apparatus.

Fabric	Static load required for Cones to pierce abraded fabric (kg)
c.f. polypropylene/polyethylene 100 g/m^2	0.25
c.f. polypropylene/polyethylene 140 g/m^2	6.0
c.f. polypropylene/polyethylene 230 g/m^2	8.0
high bulk c.f. pp/pe 230 g/m^2	10.0
c.f. polyester 250 g/m^2	8.0
Polyvinyl chloride 1.0 mm thick	8.0
Polyvinyl chloride 1.2 mm thick	10.0

2.2.1 Discussion on and Conclusions from 2.2.

The object of this test was to show the relative resistance of fabrics to sharp metal surfaces. The apparatus worked very well, although it should be noted that the divided metal must not be too fine otherwise the electrical contact with the cones is insufficient to light the lamp. The results show the non-woven thicker fabrics equate with the polyvinylchloride sheeting, and the high bulk product is particularly good..

2.3 Modified Martindale Abrasion Tester

In this test the standard Martindale abrasion test equipment, normally used for apparel fabrics, was modified for use with Geotextile fabrics because the abradant needed to be much stronger. The weighted sample holder was removed and its end modified by placing a ridged weld across it as shown in figure 9. The modified Martindale abrasion equipment is shown in figure 10. The Martindale tester works by moving in a horizontal plane first in one direction then gradually changing to a direction perpendicular to the first, thus giving a circular abrasion patch.

Figure (9). Modified weighted sample holder for use as Abradant.

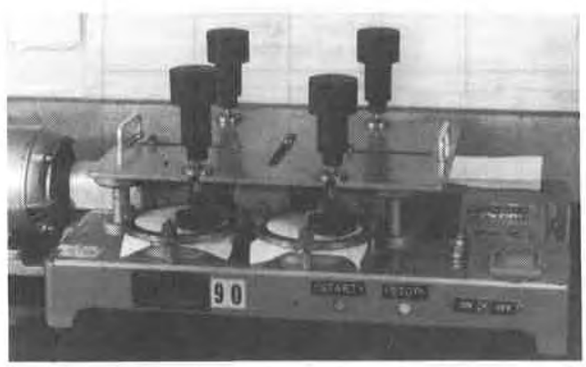

Figure (10). Modified Martindale Abrasion Equipment.

The Geotextile fabrics compared in this experiment were two non-wovens and a woven split tape. The non-wovens were a continuous filament polypropylene/polyethylene melded fabric and a needle punched c.f. polyester; the woven fabric was of polyropylene. Two pairs of samples were abraded on the equipment at the same time until one pair was worn through. On each of 4 trials the woven split-tape was worn through, with the others showing visible signs of abrasion but not being destroyed. A visual assessment of the two non-wovens showed the needle punched fabric to be the more abraded.

2.3.1 Discussion on and Conclusion from 2.3

The purpose of this trial was to try to use a standard equipment for abrasion. The Martindale equipment which was chosen needed its abradant to be changed from a fabric to a solid metal. The weld modification was chosen because in one end-use there was frequent contact with welded materials. The results showed that this test too could differentiate between fabrics, but again only comparatively, not in an absolute sense. Further test development will be needed to standardize the equipment, in particular the abradant.

3. Recommendation

Experience has shown the relevance and validity of all three test methods for the evaluation of the abrasion resistance of Geotextile fabrics, therefore it is recommended that these tests be considered by the Committees on Abrasion Testing of the British Standards Institute and the American Society for Testing and Materials for inclusion in their list of tests for Geotextile fabrics.

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Experimental and Theoretical Study of Tensile Behavior of Nonwoven Geotextiles

Etude expérimentale et théorique du comportement en traction des géotextiles nontissés

The tensile behavior of a rectangular specimen of a nonwoven geotextile depends on several parameters. Tests allow to evaluate the influence of the following parameters: polymer (polyester, polypropylene), structure of the nonwoven (needlepunched, heatbonded), continuity (continuous filaments, short fibers), and specimen geometry. One of the findings is that the behavior of narrow specimens (where filaments can move rather freely) is basically different from the behavior of geotextiles in the ground. Consequently, it is recommended to use specimens with a width/height ratio of at least 5. A theoretical relationship between the strength of a nonwoven (isotropic or anisotropic) and the strength of its filaments is presented. Theoretical values of the strength of isotropic nonwovens are in close agreement with the test results (the few differences can be explained by the anisotropy induced during the test by filament reorientation). Also, according to the theoretical analysis, a width/height ratio of 5 is an optimal value beyond which the behavior of the specimen does not vary significantly.

Le comportement en traction d'une éprouvette rectangulaire de géotextile nontissé dépend de plusieurs paramètres. Des essais permettent d'analyser l'influence des paramètres suivants: polymère (polyester, polypropylène), structure du nontissé (aiguilleté, thermolié), continuité (filaments continus, fibres courtes), géométrie de l'éprouvette. Une des conclusions des essais est que les éprouvettes étroites ont un comportement très différent de celui du géotextile in situ du fait de l'excessive mobilité des filaments. On recommande donc l'emploi d' éprouvettes dont le rapport largeur/hauteur est au moins 5. Une étude théorique fournit une relation entre la résistance d'un nontissé le plus général (isotrope ou non) et celle de ses filaments. Les résultats théoriques relatifs au nontissé isotrope sont en bon accord avec les résultats expérimentaux (quelques différences étant expliquées par l'anisotropie induite lors de l'essai par la réorientation des filaments). Enfin, l'étude théorique montre que le rapport largeur/hauteur de 5 est un optimum au dela duquel le comportement de l'éprouvette change peu.

INTRODUCTION

Cette communication traite du comportement d'une éprouvette rectangulaire de géotextile nontissé de largeur B, soumise à une traction dans la direction de sa hauteur H. L'allongement est ΔH, d'où ε_1 la déformation moyenne axiale. Le rétrécissement est ΔB, d'où ε_2 la déformation moyenne latérale (Fig. 1).

1. ETUDE EXPERIMENTALE

1.1 Présentation des essais

Les essais ont été faits avec des mors de 500 mm fabriqués pour les géotextiles. La géométrie des éprouvettes a été: B de 50 a 500 mm, H de 30 a 500 mm, B/H de 0.1 a 16.

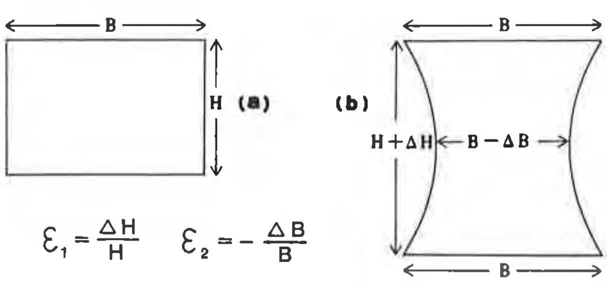

Fig. 1. Eprouvette: (a) avant traction; (b) tendue.

La vitesse de déformation axiale a été de 50% par minute sauf dans une série d'essais, faite avec des vitesses de 12.5 a 100% par minute, qui a montré que la vitesse avait une influence négligeable. Tous les essais ont été faits dans le sens machine après avoir vérifié, par quelques essais dans le sens travers, que les géotextiles testés étaient quasi-isotropes.

Chaque résultat présenté provient d'un seul essai. Compte tenu de la dispersion inévitable, il aurait été plus exact de répéter chaque essai et de faire une moyenne. Pour un nombre total d'essais donné (plus de 200), il a été jugé préférable de sacrifier la précision et d'examiner un grand nombre de paramètres.

1.2 Géotextiles testés

Pour chacun des cinq géotextiles nontissés testés nous donnons: le nom, la masse surfacique μ, la porosité n, le type, le polymère constituant les filaments ou les fibres, la masse linéique λ des filaments ou des fibres, et un symbole qui sera utilisé dans la suite: (1) Bidim U44, μ = 340 g/m², n = 0.90, aiguilleté filaments continus, polyester, λ = 7.35 dtex, BD; (2) Sodoca AS400, μ = 370 g/m², n = 0.86, aiguilleté filaments continus, polypropylène, λ = 7.1 dtex, SD; (3) Lutravil LDH, μ = 195 g/m², n = 0.70, thermolié filaments continus, polyester, λ = 9.45 dtex, LT; (4) Typar, μ = 270 g/m², n = 0.54, thermolié filaments continus, polypropylène, λ = 12.6 dtex, TP; (5) Sommer 400, μ = 400 g/m², n = 0.93, aiguilleté fibres courtes, polyester λ = 17 dtex (50% en poids) et polypropylène λ = 6.9 dtex (50% en poids), SM.

Ce choix de géotextiles a permis d'étudier l'influence des paramètres suivants: (i) structure (aiguilletés, thermoliés); (ii) continuité (filaments continus, fibres courtes); (iii) polymère (polyester, polypropylène). Noter que la comparaison polyester-polypropylène est plus aisée entre BD et SD, de porosités voisines, donc ayant des arrangements semblables de filaments, qu'entre LT et TP. Le paramètre masse surfacique a été éliminé en divisant toutes les forces par la masse surfacique.

1.3 Comportement des filaments et fibres

Les essais de traction sur les filaments des géotextiles BD, SD, LT et TP donnent deux types de courbes (Fig. 2): (i) la courbe des filaments de polypropylène croît d'abord rapidement jusqu'à ε_1 = 40% environ puis croît lentement jusqu'à la rupture qui survient pour ε_1 très grand (159% pour les filaments de SD et 238% pour les filaments de TP); (ii) la courbe des filaments de polyester croît de façon continue jusqu'à la rupture qui survient pour ε_1 = 71% (filaments de LT) et ε_1 = 78% (filaments de BD). Jusqu'à ε_1 = 30% toutes les courbes sont voisines mais, au-delà, les filaments de polyester sont moins déformables.

Les fibres du géotextile SM se rompent pour: ε_1 = 47.5% (polyester) et ε_1 = 105% (polypropylène). Ayant subi un fort étirage préalable, elles sont moins déformables que les filaments de la Fig. 2.

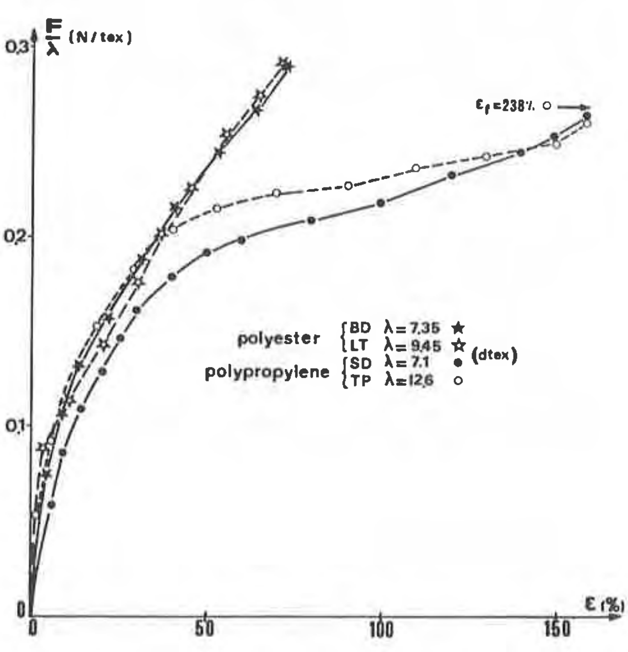

Fig. 2. Essais de traction sur filaments: force F par unité de masse linéique λ en fonction de la déformation ε. La tenacité ζ_f du filament est la valeur de F/λ a la rupture (ζ_f = 0.295 N/tex pour les filaments de polyester et 0.265 N/tex pour les filaments de polypropylène).

1.4 Résultats d'essais sur éprouvettes de géotextiles

Influence du polymère. Sur l'ensemble des essais sur nontissés à filaments continus, la déformation axiale à la rupture ε_{1f} a été de 36% à 78% (géotextile BD), 25% à 64% (LT), 65% à 160% (SD), et 25% à 85% (TP). Pour un géotextile donné, la plage des valeurs de ε_{1f} est large du fait de la variété de géométries des éprouvettes testées (voir Fig. 3a et 4a). Pour un même type de structure (aiguilletés d'une part, thermoliés d'autre part), les nontissés polypropylène sont plus déformables que les nontissés polyester, ce qui s'explique par la plus grande déformabilité des filaments polypropylène (Fig. 2). On constate également que la déformation axiale à la rupture des nontissés à filaments continus est inférieure à celle des filaments (entre la déformation axiale à la rupture des aiguilletés et celle de leurs filaments, le rapport est de 40 à 100%, selon la géométrie de l'éprouvette). Au contraire, les nontissés à fibres courtes peuvent avoir des déformations axiales très grandes, excédant celles des filaments (voir Fig. 6).

Ce qui précède concerne uniquement l'influence du polymère sur la déformation axiale à la rupture des géotextiles nontissés. L'influence du polymère sur la résistance sera discutée dans l'étude théorique.

Enfin, la déformation latérale, ε_2, d'un nontissé polyester est légèrement supérieure en valeur absolue à celle d'un nontissé polypropylène (Fig. 3c et 4c).

Influence de la structure du géotextile. D'après les Fig. 3a et 4a, et de nombreuses courbes analogues non reproduites: (i) les thermoliés ont un module initial plus grand que les aiguilletés et beaucoup moins sensible aux variations du rapport des côtés de l'éprouvette, B/H; (ii) pour un polymère et une géométrie d'éprouvette donnés, la déformation axiale à la rupture d'un thermolié est inférieure à celle d'un aiguilleté. Lors des essais, on constate que la rupture est brutale dans le cas d'un thermolié alors qu'elle se propage progressivement à travers l'éprouvette dans le cas d'un aiguilleté. Ces constatations sur l'allure des courbes et le mode de rupture suggèrent que: (i) les filaments des aiguilletés sont relativement libres de s'orienter dans le sens de l'effort de traction, réorientation qui se traduit par une grande déformation axiale, donc un module faible au début de l'effort, et une répartition de l'effort entre de nombreux filaments atténuant les concentrations de contraintes susceptibles de causer une rupture brutale; (ii) les filaments des thermoliés, n'étant pas libres de s'orienter dans le sens des efforts de traction, peuvent se rompre prématurément en cisaillement et être le siège de contraintes concentrées susceptibles de causer la propagation brutale de la rupture.

La déformation latérale d'un aiguilleté se produit dans la première partie de l'essai du fait de la mobilité de ses filaments et elle est supérieure en valeur absolue à celle d'un thermolié (Fig. 3c et 4c). La variation de surface des éprouvettes (Fig. 3b et 4b) sera discutée plus loin.

Influence de la continuité des filaments. Les essais sur un aiguilleté de fibres courtes SM (Fig. 6) montrent que: (i) sa tenacité (α/μ à la rupture) est de l'ordre du tiers de celle des aiguilletés à filaments continus; (ii) sa déformation axiale à la rupture est supérieure à celle des fibres qui le constituent. On peut penser que les fibres courtes glissent les unes par rapport aux autres au cours de la traction, d'où la faible tenacité et la grande déformation.

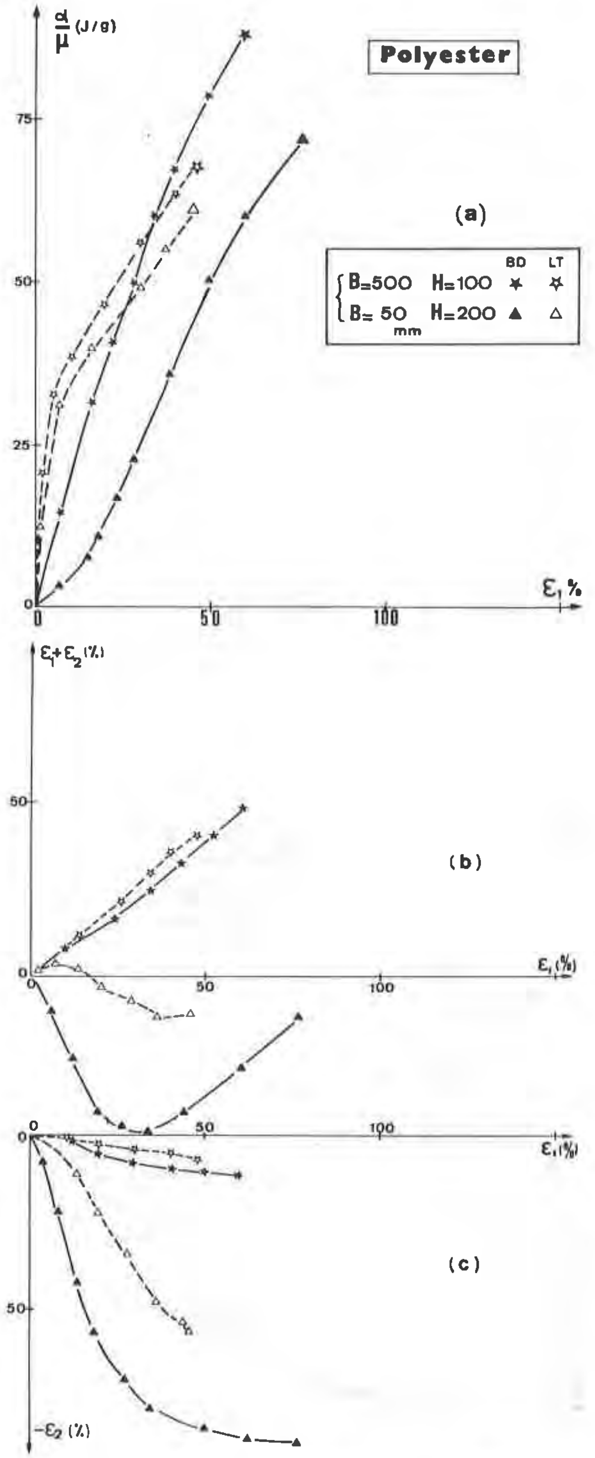

Fig. 3. Essais de traction sur géotextiles nontissés polyester: (a) force par unité de largeur α/masse surfacique μ; (b) variation de surface de l'éprouvette; (c) déformation latérale ε_2 (ε_1 est la déformation axiale).

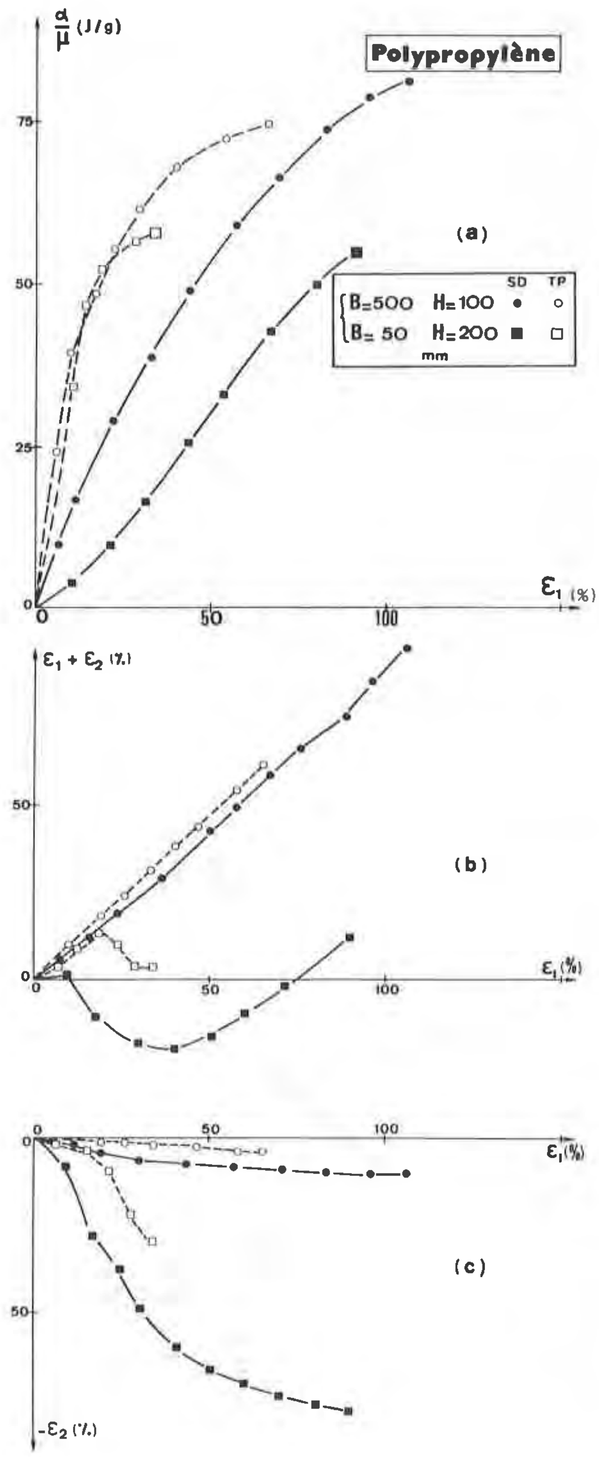

Fig. 4. Essais de traction sur géotextiles nontissés polypropylène: (a) force par unité de largeur α/masse surfacique μ; (b) variation de surface de l'éprouvette; (c) déformation latérale ε_2 (ε_1 est la déformation axiale).

Influence des dimensions de l'éprouvette. Les courbes de la Fig. 5 sont relatives à des éprouvettes ayant un même rapport des côtés, B/H=1, et des dimensions (B et H) différentes. Pour un géotextile donné, ces courbes sont identiques sauf à la rupture qui se produit pour des déformations décroissantes lorsque la surface (BH) de l'éprouvette croît. Nous avons obtenu des résultats identiques pour les autres valeurs de B/H considérées. Par conséquent: (i) tout se passe comme si, au voisinage de la rupture, les filaments étaient moins libres de se réorienter lorsque l'éprouvette est grande; (ii) pour un rapport des côtés B/H donné, les valeurs de module (relatives à des déformations axiales inférieures à environ la moitié de la déformation axiale à la rupture) sont significatives, alors que la résistance et la déformation à la rupture dépendent des dimensions B et H. Il faut donc toujours mesurer le module dans les essais de traction.

Influence du rapport des côtés de l'éprouvette. D'après les Fig. 3a, 4a et 6, et de nombreuses courbes analogues non reproduites, on peut dire, pour les nontissés à filaments continus, que: (i) lorsque B/H augmente, la tenacité (α/μ à la rupture) du géotextile augmente et sa déformation axiale à la rupture ε_{1f} diminue, ces deux effets étant beaucoup plus marqués pour les aiguilletés que pour les thermoliés; (ii) le module initial des aiguilletés augmente lorsque B/H augmente alors que celui des thermoliés varie peu. La Fig. 6 montre que le comportement en traction du nontissé à fibres courtes (SM) dépend très peu de B/H.

Lors des essais, on constate que le mode de rupture est très influencé par le rapport B/H. Pour les petites valeurs de B/H (0.25, par exemple, essai de traction traditionnel) la rupture a lieu près des mors, tandis que pour les grandes valeurs de B/H (5, par exemple, valeur retenue par le Comité Français des Géotextiles) (1) on observe fréquemment une rupture se propageant en travers de la partie centrale de l'échantillon, suivant une ligne légèrement inclinée. On peut donc en conclure que les éprouvettes avec grand B/H permettent des essais plus significatifs.

Le rétrécissement ε_2 de l'éprouvette au cours d'un essai de traction (Fig. 3c et 4c) est nettement plus faible en valeur absolue pour les grandes valeurs de B/H (5, par exemple) que pour les petites valeurs de B/H (0.25, par exemple). Les éprouvettes avec grand B/H simulent donc le comportement des géotextiles dans les ouvrages où les déformations latérales sont quasi-nulles.

La surface d'une éprouvette varie au cours de l'essai de traction (Fig. 3b et 4b). Il y a conflit entre deux tendances: augmentation de surface due à la déformation axiale et diminution de surface due au rétrécissement latéral. Lorsque B/H est élevé (5, par exemple), la surface de l'éprouvette augmente régulièrement au cours de l'essai, alors que pour B/H faible (0.25, par exemple), la surface de l'éprouvette commence par diminuer avant d'augmenter (sauf dans le cas du nontissé thermolié TP). Ceci s'explique par la réorientation des filaments, plus difficile lorsque B/H est grand et plus difficile pour les thermoliés que pour les aiguilletés. On peut conclure de ces remarques que les éprouvettes ayant une grande valeur de B/H fournissent des conditions d'essai plus régulières.

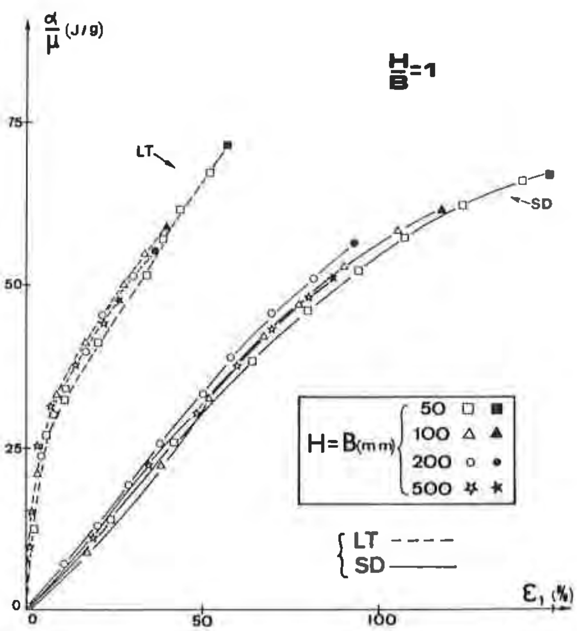

Fig. 5. Force par unité de largeur α/masse surfacique μ en fonction de la déformation axiale ε_1 pour éprouvettes de dimensions différentes mais de même rapport des côtés. Chaque courbe se termine au point noir.

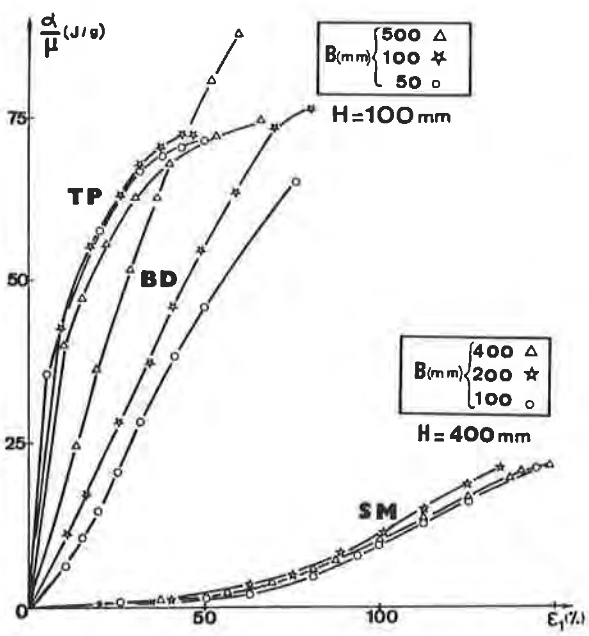

Fig. 6. Influence du rapport des côtés de l'éprouvette sur les résultats de l'essai de traction. Courbes donnant la force par unité de largeur α/masse surfacique μ en fonction de la déformation axiale.

2. ETUDE THEORIQUE

2.1 Présentation de l'étude

Cette étude conduit à une relation entre le comportement en traction d'un géotextile nontissé et celui de ses filaments. Les hypothèses suivantes sont faites: (i) le géotextile est homogène, c'est-à-dire statistiquement identique en tout point; (ii) la masse linéique et l'orientation des filaments sont distribuées de façon quelconque (l'étude s'applique aux nontissés anisotropes ou isotropes et, à la limite, aux tissés qui sont un cas extrême d'anisotropie); (iii) les filaments sont continus (l'étude ne s'applique pas aux géotextiles faits de fibres courtes); (iv) les filaments sont rectilignes (cette hypothèse n'est généralement pas vérifiée, d'où un écart entre résultats théoriques et expérimentaux); (v) la loi de comportement des filaments est linéaire ou linéaire avec palier (voir Fig. 9).

L'étude a nécessité de longs calculs dont seul le principe et les résultats sont présentés. La première partie consiste à décrire mathématiquement l'arrangement le plus général de filaments.

2.2 Arrangement des filaments

Distribution des filaments selon l'orientation. Nous avons mis au point la méthode suivante: (i) on considère un échantillon de géotextile circulaire de surface A; (ii) on délimite toutes les orientations possibles par n secteurs d'angle π/n (n très grand) (Fig. 7a); (iii) sur la bissectrice de chaque secteur on porte un rayon vecteur:

$$\rho_i = \sqrt{2n\ m_i/(\pi A)} \qquad (1)$$

avec: m_i = masse des filaments dont l'orientation est comprise dans le secteur i.

En joignant les extrémités des rayons vecteurs, on obtient une "courbe de distribution angulaire" dont la surface est égale à la masse surfacique μ du géotextile. En effet:

$$\sum (\rho_i/2)\ (\pi\rho_i/n) = \sum m_i/A = \mu \qquad (2)$$

Dans le cas d'un nontissé isotrope, la courbe est un demi-cercle de rayon $\sqrt{2\mu/\pi}$. Le cas le plus simple de nontissé anisotrope est celui où la courbe est une demi-ellipse (Fig. 7b):

$$\rho = \sqrt{2\ a\ \mu/(\pi(\cos^2\theta + a^2\sin^2\theta))} \qquad (3)$$

avec: a = coefficient d'anisotropie (supérieur ou inférieur à 1) égal au rapport ρ_m/ρ_x (rayon vecteur sens machine/rayon vecteur sens travers).

La méthode est générale. La courbe de distribution angulaire s'applique même aux tissés. Elle a alors la forme d'une croix (Fig. 7c) dont les branches ont pour surface μ_m et μ_x, masses surfaciques dans le sens machine et le sens travers respectivement (avec évidemment $\mu_m + \mu_x = \mu$).

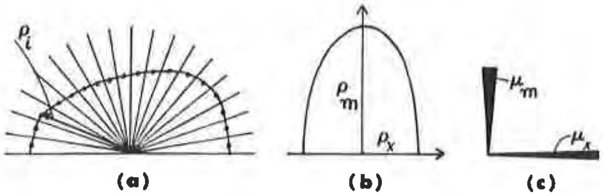

Fig. 7. Courbes de distribution angulaire des filaments: (a) principe; (b) nontissé anisotrope; (c) tissé.

Relation entre filaments et mors. Les seuls filaments contribuant à la résistance de l'éprouvette sont ceux qui relient les deux mors (Fig. 8a). La somme des masses linéiques des filaments reliant les deux mors de largeur B avec une inclinaison comprise entre δ et δ+dδ est:

$$\sum \lambda_{\beta\delta} = B\ d\Omega_{\beta\delta} \cos\delta\ (1 - H\ \mathrm{tg}\,\delta/B) \qquad (4)$$

avec: $d\Omega_{\beta\delta}$ = aire du secteur dδ de la courbe de distribution angulaire (valant $\mu d\delta/\Pi$ dans le cas d'un nontissé isotrope); β = angle entre la direction machine et la normale aux mors (Fig. 8b).

2.3 Contribution des filaments à la résistance de l'éprouvette

La contribution d'un filament à la résistance de l'éprouvette dépend de sa déformation. Par des considérations géométriques, on montre qu'un filament incliné de δ (Fig. 8a) a une déformation ε_δ plus faible que la déformation ε de l'éprouvette:

$$\varepsilon_\delta = \sqrt{1+\varepsilon \cos^2\delta\ (2+\varepsilon)}\ -1 \simeq \varepsilon \cos^2\delta \qquad (5)$$

Deux cas sont à considérer selon la forme de la courbe force/masse linéique en fonction de la déformation (Fig. 9). Si la courbe est linéaire (Fig. 9a), la force/masse linéique développée par un filament d'inclinaison δ (se déformant de $\varepsilon \cos^2\delta$) est plus faible que celle $(F/\lambda)_o$ développée par un filament perpendiculaire aux mors (se déformant de ε):

$$(F/\lambda)_\delta = (F/\lambda)_o \cos^2\delta \qquad (6)$$

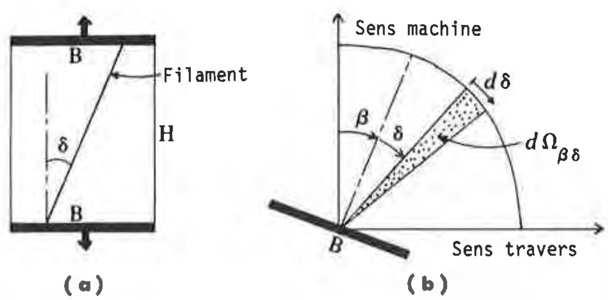

Fig. 8. Relation entre filaments et mors.

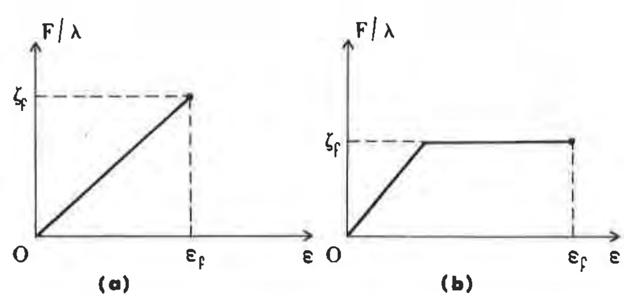

Fig. 9. Courbes de traction de filaments: (a) linéaire; (b) linéaire avec palier.

Sauf dans le cas de géotextiles anisotropes subissant une traction dans leur direction la plus faible, la résistance de l'éprouvette (somme de toutes les forces données par l'Eq. 6) atteint son maximum lorsque les filaments perpendiculaires aux mors se rompent (en effet, sitôt après, la résistance de l'éprouvette décroit car les filaments restants sont plus inclinés et ont une densité angulaire plus faible). On a alors, d'après la Fig. 9a, $(F/\lambda)_o = \zeta_f$, d'où:

$$(F/\lambda)_\delta = \zeta_f \cos^2\delta \tag{7}$$

La force développée par l'ensemble des filaments d'inclinaison comprise entre δ et $\delta+d\delta$ s'obtient en multipliant les Eq. 4 et 7:

$$dF_\delta = B\zeta_f \, d\Omega_{\beta\delta} \cos^3\delta \, (1 - H \, tg\delta/B) \tag{8}$$

La projection de cette force sur l'axe est $dF_\delta \cos\delta$ et l'intégration pour toutes les valeurs possibles de δ (de 0 a Arctg (B/H)) conduit à la relation entre la tenacité ζ_g d'un nontissé et celle ζ_f de ses filaments. Dans le cas d'un nontissé à anisotropie elliptique avec a > 1 subissant une traction dans le sens machine, on obtient:

$$\zeta_g = \{(a\zeta_f/\pi)/(a^2-1)^2\} \, \{2a^3 \text{ Arctg } (aB/H) - (3a^2-1) \text{ Arctg } (B/H) - (a^2/(B/H)) \ln((1+(aB/H)^2)/(1+(B/H)^2))\} \tag{9}$$

Dans le cas d'un nontissé isotrope (subissant une traction dans n'importe quelle direction) cette équation devient:

$$\zeta_g = (\zeta_f/4\pi) \, (3 \text{ Arctg } (B/H) + (B/H)/(1 + (B/H)^2)) \tag{10}$$

Si la courbe force/masse linéique en fonction de la déformation présente un palier assez long (Fig. 9b), au moment de la rupture de l'éprouvette tous les filaments travaillent à leur pleine tenacité, ζ_f. Le $\cos^2\delta$ disparait de l'Eq. 7 et $\cos^3\delta$ devient $\cos\delta$ dans l'Eq. 8. Dans le cas d'un nontissé isotrope, l'intégration conduit alors à:

$$\zeta_g = (\zeta_f/\pi) \, (\text{Arctg } (B/H)) \tag{11}$$

Les courbes des Eq. 10 et 11 (Fig. 10) atteignent 90% de la valeur asymptotique finale pour B/H = 4.3 et 6.3 respectivement, justifiant ainsi théoriquement la valeur 5 choisie par le Comité Français des Géotextiles pour le rapport B/H des éprouvettes (1). La valeur finale est $\zeta_g = 3\zeta_f/8$ pour l'Eq. 10 et $\zeta_g = \zeta_f/2$ pour l'Eq. 11. La moyenne de la tenacité d'un tissé dans les deux directions des fils est évidemment $\zeta_g = \zeta_f/2$. On en déduit que la tenacité d'un nontissé isotrope est théoriquement 75% a 100% de celle d'un tissé.

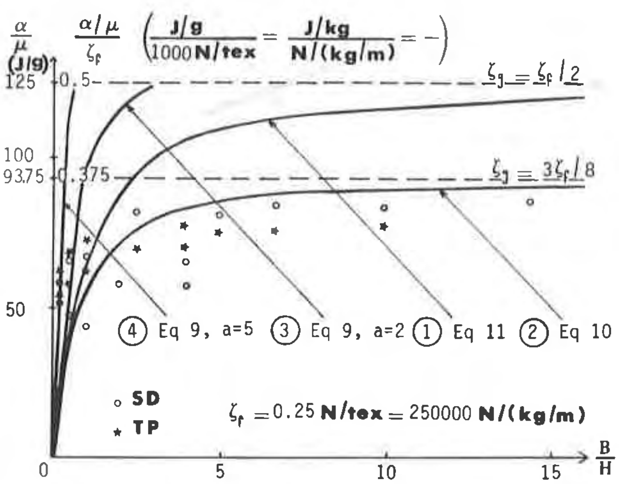

Fig. 10. Points expérimentaux et courbes théoriques pour le polypropylène (ζ_f = 0.25 N/tex, valeur moyenne du pseudo-palier de la Fig. 2).

2.4 Comparaison des résultats théoriques et expérimentaux

Les tenacités mesurées sur SD et TP, nontissés polypropylène quasi-isotropes devraient se situer entre les courbes 1 et 2 de la Fig. 10, car on peut estimer que le pseudo-palier des filaments de polypropylène (Fig. 2) constitue un cas intermédiaire entre la courbe linéaire et la courbe avec palier. En fait, les valeurs expérimentales sont environ 25% plus faibles pour B/H >3 et sont plus grandes pour B/H<1. On observe les mêmes écarts entre les points expérimentaux relatifs aux nontissés quasi-isotropes polyester, BD et LT, et les valeurs calculées par l'Eq. 10 (qui devrait convenir pour les nontissés polyester car la courbe de traction de leurs filaments n'a pas de palier) avec ζ_f = 0.295 N/tex.

Le déficit de 25% pour B/H>3 s'explique par le fait que les filaments réels ne travaillent pas dans des conditions idéales: ils ne sont pas rectilignes et leur réalignement dans une éprouvette relativement large n'est pas facile, ce qui conduit à une rupture pour une déformation inférieure a celle permettant aux filaments de développer leur force maximale. En conclusion, le rapport entre la tenacité d'un nontissé isotrope (dans toute direction) et celle d'un tissé (moyenne dans la direction des deux fils) est de 56% (0.75 x 75%) a 75% (0.75 x 100%).

Le fait que, pour les faibles valeurs de B/H, les points expérimentaux soient au dessus des courbes théoriques relatives aux géotextiles isotropes peut s'expliquer par l'anisotropie induite dans l'éprouvette par la réorientation des filaments, chose facile dans une éprouvette relativement étroite (voir courbes 3 et 4 tracées pour un coefficient d'anisotropie a = 2 et a = 5 respectivement, Fig. 10).

CONCLUSION

Les résultats des essais de traction dépendent, en autres paramètres, du rapport des côtés de l'éprouvette, notamment pour les nontissés aiguilletés dont la mobilité des filaments diminue lorsque le rapport largeur/hauteur de l'éprouvette augmente. La valeur optimale de ce rapport est de 5 d'après l'étude théorique.

REMERCIEMENTS

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A Study of the Flexibility of Geotextiles

Etude de la flexibilité des géotextiles

In order to achieve easiness of processing, the geotextiles should display, in addition of their end uses characteristics, an ability to handling compatible with the features of the ground zone where they have to be used. The "handle ability", i.e, the easiness of setting and the ability to fit to the morphology of the ground is in close relationship to the flexural characteristics of the material.
A comparaison between the methods of measurement of flexibility leads to the adaptation to the case geotextiles of the operating procedure described in ASTM D 1388-64 Textile Standard, which was later on recommanded by EDANA (50-1-80).
Measured Flexural modules are in good agreement with these obtained from the Initial Young Modulus. The determination of the flexibility is able to lead to these Initial Modulus which are generally difficult to obtain.

Pour une mise en oeuvre aisée des géotextiles, ceux-ci doivent avoir en plus des caractéristiques nécessaires à leur emploi, une maniabilité qui soit compatible avec l'état du terrain où ils vont être utilisés. La maniabilité, c'est-à-dire la facilité de pose et la faculté d'épouser les formes du sol en place, est directement liée à la flexibilité du matériau.
Une comparaison des méthodes de mesure de la flexibilité a conduit à adapter au cas des géotextiles le mode opératoire décrit dans la norme textile ASTM D 1388-64 et reprise par EDANA (50-1-80) pour les nontissés.
Les modules de flexion mesurés sont en bon accord avec ceux obtenus avec les modules initiaux de traction. La mesure de la flexibilité peut donc permettre d'obtenir ces modules initiaux qui sont généralement difficilement accessibles.

INTRODUCTION

Le choix d'un tissu d'habillement se fait souvent à partir de propriétés subjectives telles que le toucher, la main, le drapé qui sont plus ou moins liées à sa rigidité. Ce n'est généralement pas le cas pour des tissus industriels et plus particulièrement pour les géotextiles où l'on s'intéresse en premier aux caractéristiques mécaniques telles que résistance à la rupture ou résistance au déchirement et aux caractéristiques hydrauliques. Cependant, la souplesse est utile à connaître et cela pour plusieurs raisons. Elle intervient dans la facilité de pose où un matériau rigide suivra plus difficilement les lignes de courbure d'un ouvrage. Elle joue un rôle dans l'interpénétration du géotextile avec les matériaux granulaires, et aura par suite une incidence sur les caractéristiques de frottement. Elle est mise en cause dans tous les cas où le géotextile doit pouvoir épouser la surface du sol ou des parois comme dans les tranchées drainantes. Sa connaissance est importante pour obtenir une similitude satisfaisante dans les études hydrauliques sur modèles réduits. Ce dernier point conduit plus particulièrement à la nécessité d'avoir une méthode qui permette d'évaluer la flexibilité des géotextiles.

Après avoir donné les raisons du choix d'une méthode et son principe on présente les résultats obtenus sur une gamme variée de géotextiles.

1 CHOIX DE LA METHODE

La norme française NF G 37-108 définit une méthode d'essai qui permet de caractériser la souplesse des supports textiles revêtus d'élastomères ou de matières plastiques. Elle consite à poser sur un plan horizontal une bande rectangulaire du matériau et à relier une de ses extrémités à l'autre de manière à former une boucle. La souplesse est caractérisée par la hauteur de la boucle.

Cette méthode pourrait convenir à des matériaux assez rigides comme les nontissés thermosoudés, mais elle devient inopérante pour les matériaux relativement souples comme les nontissés aiguilletés. En effet, pour ces derniers, la hauteur de la boucle dépend principalement de la manière dont on manipule l'éprouvette pour faire la boucle. Etant donné qu'elle n'est pas applicable à tous les géotextiles elle n'a pas été retenue.

L. Kelley (1) a proposé une détermination de la rigidité à partir d'une pendule de torsion. Une extrémité d'une éprouvette est fixée dans une mâchoire fixe tandis que l'autre est pincée dans une mâchoire qui peut osciller. Le mouvement d'un échantillon non résilient est très vite amorti et une tension appliquée à l'éprouvette peut diminuer l'amortissement. La rigidité est calculée à partir de la formule :

où G est la rigidité
K un facteur constant
L la longueur de l'échantillon
I le moment d'inertie de torsion de la mâchoire
C la largeur de l'échantillon
D l'épaisseur de l'échantillon
T la période de l'oscillation en seconde
μ un facteur de forme égal au rapport de l'épaisseur à la largeur.

Avec des géotextiles relativement épais et peu résilient, il peut être nécessaire de surcharger d'une manière importante la mâchoire mobile afin que l'amortissement soit suffisamment faible pour que la période d'oscillation du pendule soit mesurable. Dans le cas des géotextiles nontissés et plus particulièrement aiguilletés qui ont une capacité d'allongement importante, cette surcharge peut conduire à une déformation de l'éprouvette qui ne permet pas une détermination de la rigidité dans l'état initial du matériau.

La norme ASTM D 1388-64, reprise par l'Edana sous le numéro 50-1-80, propose une méthode de détermination de la rigidité qui a été étudiée par Pierce (2)pour les articles destinés à l'habillement. Son principe est de poser une bande rectangulaire de tissu sur une plate-forme horizontale perpendiculairement à un des bords de la plate-forme et de la déplacer suivant l'axe de sa plus grande longueur de sorte qu'une partie croissante pende et se courbe sous son propre poids. Le déplacement de l'éprouvette est effectué par glissement en l'appuyant sur la plate-forme avec une règle plate. On note la longueur de la partie pendante de l'éprouvette lorsqu'une de ses extrémités a atteint un plan passant par le bord de l'éprouvette et faisant un angle de 41,5° avec l'horizontal (Fig. 1). Pour chaque éprouvette, on relève les valeurs de l pour chacune de ses extrémités et chacune de ces faces, c'est-à-dire 4 déterminations par éprouvette. La longueur de courbure est notée directement à partir de son observation et le module de rigidité peut être calculé à partir de cette longueur de courbure. Des essais préliminaires ont montré que cette méthode pouvait être utilisée avec quelques modifications pour quantifier la rigidité des géotextiles.

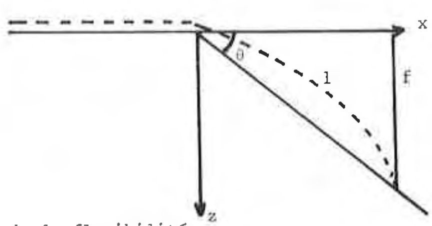

Fig. 1 Schéma de la flexibilité.

2 MESURE DE LA FLEXIBILITE ET RESULTATS

2.1 Dimensions des éprouvettes

Dans la méthode ASTM, les éprouvettes sont rectangulaires et ont pour dimensions 2,5 x 15 cm ; les dimensions des éprouvettes Edana sont : 2,5 x 20 cm. Avec les géotextiles, la longueur de l'éprouvette s'est avérée trop petite pour que l'extrémité de la partie pendante de l'éprouvette puisse aller toucher le plan incliné. Il a donc été nécessaire de prendre des éprouvettes plus longues. Une longueur de 50 cm a permis de tester tous les géotextiles étudiés.

L'influence de la largeur de l'éprouvette sur la longueur de la partie pendante a été examinée sur trois types de géotextile : un nontissé aiguilleté, un nontissé thermocollé et un tissé.

Dans le tableau 1 sont indiqués les moyennes des valeurs correspondant à 10 éprouvettes ce qui correspond en fait à 40 mesures. Il apparaît que les valeurs de l ainsi obtenues ne dépendent pas de la largeur des éprouvettes tout au moins dans la limite des largeurs expérimentées. Pour ne pas tomber dans des éprouvettes trop larges et pour faciliter la préparation des éprouvettes, la largeur retenue pour les éprouvettes est de 5 cm.

2.2 Paramètres mesurés

Si on considère une barre, l'équation fondamentale d'élasticité en flexion simple est :

$$\frac{I}{r} = \frac{M}{EI} \tag{2}$$

où r est la courbure, M est le moment fléchissant, E le module d'Young et I le moment d'inertie autour de l'axe oy.

La géométrique analytique donne :

$$\frac{I}{r} = \frac{\frac{d^2z}{dx^2}}{\left[1 + \left(\frac{dz}{dx}\right)^2\right]^{3/2}} \tag{3}$$

ce qui donne :

$$\frac{\frac{d^2z}{dx^2}}{\left[1 + \left(\frac{dz}{dx}\right)^2\right]^{3/2}} = \frac{M}{EI} \tag{4}$$

Si la barre est peu flexible, les déformations sont faibles et $\frac{dz}{dx}$ est négligeable ce qui conduit à

$$\frac{d^2z}{dx_2} = \frac{M}{EI} \tag{5}$$

Tableau 1. Influence de la largeur de l'éprouvette sur la courbure

Largeur de l'éprouvette	Nontissé aiguilleté l	CV %	Nontissé thermocollé l	CV %	Tissé l	CV %
2,5 cm	16	6,7	25,1	5,8	21,1	1,9
5 cm	17,2	7,5	25,8	4,6	20,9	1,5
7,5 cm	16,9	7,2	25,3	7,4	20,8	2

Dans le cas d'une barre console, de section constante, soumise à une charge P = pl uniformément répartie, le moment fléchissant dans une section quelconque est $M_x = + \frac{pl^2}{2}$ (6)

d'où $$\frac{d^2z}{dx^2} = \frac{pl^2}{2\,EI} \quad (7)$$

Par double intégration et en tenant compte des conditions initiales, on obtient :

$$f = \frac{pl^4}{8\,EI} \quad (8)$$

En posant f = p tg θ, on obtient :

$$G = EI = \frac{pl^3}{8\,\mathrm{tg}\,\theta} \quad (9)$$

G est le module de flexion.

Dans le cas général, où $\frac{dz}{dx}$ n'est pas négligeable, ce qui est le cas des éprouvettes de géotextiles, l'équation différentielle 4 n'est pas directement intégrale.

Pierce (2) dans son étude de la flexibilité des tissus d'habillement a proposé comme solution de cette équation la formule 10 :

$$G = \frac{pl^3}{8} \cdot \frac{\cos\theta/2}{\mathrm{tg}\,\theta} \quad (10)$$

Par construction de l'appareillage $\frac{\cos\theta/2}{\mathrm{tg}\,\theta} = 1$ ce qui conduit à :

$$G = \frac{pl^3}{8} \quad (11)$$

Les paramètres mesurés sont donc la longueur de courbure $c = \frac{1}{2}$ et le module de flexion $G = mc^3$.

Pour comparer la rigidité de géotextiles de différentes épaisseurs, on calcule également ce qu'on convient d'appeler un module de rigidité qui est donné par la relation :

$$Q = \frac{12\,G}{e^3} \quad (12)$$

2.3 Unités et équations dimensionnelles

La longueur de courbure c est exprimée en mètres.

Dans l'équation du module de flexion G = EI, on a :

$$E = \frac{F}{eb \times \frac{\Delta L}{L}} \quad (13)$$

avec F force par unité de largeur du géotextile
avec b largeur de l'éprouvette et e son épaisseur

$\frac{\Delta L}{L}$ allongement relatif correspondant à F

Tableau 2. Valeur des longueurs de courbure des modules de flexion G et des modules de rigidité q pour divers matériaux.

Echantillons	Masse surfacique en g/m2	Epaisseur en m x 10^{-3}	$C = \frac{1}{2}$ en m x 10^{-2}		$G = PC^3$ N x m x 10^{-4}		$q = \frac{12\,G}{e^3}$ en N/m2 x 10^6	
			sens production	sens travers	sens production	sens travers	sens production	sens travers
Nontissé thermosoudé PE - PP	112	0,5	15,2	12,9	39,6	24,3	380	230
	132	0,7	16,6	13	59,9	29,1	210	100
	173	0,8	19	15,8	118,1	68	280	160
	227	0,9	21,3	17,5	219,4	122,1	360	200
	255	1	22,1	21,8	275,2	262,4	330	310
Nontissé thermosoudé PP	133	0,45	11,6	8,7	20,9	8,6	270	110
	202	0,6	13,9	9,7	54	18,5	300	100
	274	0,7	16,2	11,6	117,4	43	410	150
Tissé de PP	100	0,38	5,5	5,9	1,6	2	36	44
	139	0,63	6,5	8,1	3,8	7,4	18	35
	198	0,74	6,6	8,2	5,8	10,9	17	32
	339	1,32	8,9	13,1	23,7	76	12	39
	547	1,78	10	9,3	54,7	44	11	9
Nontissé aiguilleté PET	156	1,5	6,8	5,1	4,9	2,1	1,74	0,73
	221	1,9	7,1	7	8	7,6	1,4	1,33
	276	2,3	8,1	8,6	14,8	17,5	1,46	1,72
	329	2,8	9,8	8,7	30,8	21,3	1,68	1,16
	538	4,4	11,5	11,8	81,8	86,8	1,15	1,22
Nontissé aiguilleté PP	204	2,5	5,3	5,5	3,1	3,3	0,24	0,25
	251	2,8	6,1	7,3	7,1	5,4	0,39	0,30
	274	3	6,6	6	6,2	10,4	0,27	0,46
	380	3,7	7,1	8,3	13,4	21,7	0,32	0,51
	600	4,8	9,7	11,8	54,8	98,6	0,59	1,07

PP = Polypropylène
PE = Polyéthylène
PET = Polyester

Tableau 3. Comparaisons des modules de flexion mesurés et calculés à partir des modules sécants G_1 et des modules initiaux G_2

Echantillons	Masse surfacique g/m2	$G = PC^3$ N x m x 10^{-4}		$G_1 = EI_{sécant}$ N x m x 10^{-4}		$G_2 = EI_{initial}$ N x m x 10^{-4}	
		sens production	sens travers	sens production	sens travers	sens production	sens travers
Nontissé thermosoudé PE - PP	112	39,6	24,3	15,5	12,7	14	10,8
	132	59,9	29,1	30,6	24	28	22
	173	118,1	68	46,7	40,8	54	52
	227	219,4	122,1	68,5	64	86	67,5
	255	275,2	262,4	100	83,6	107	127
Nontissé thermosoudé PP	133	20,9	8,6	-	-	14,7	8,6
	202	117,4	43	24,5	-	46,3	47,6
	274	54	18,5	-	-	35,7	27,9
Tissé de PP	100	1,63	2	11	9,9	3,3	3,7
	139	3,8	7,4	35	33,8	10	10,4
	198	5,8	10,9	63	57	10,7	12,9
	339	23,7	76	264	444	52	110
	547	54,7	44	1 110	1 170	95	66,4
Nontissé aiguilleté PET	156	4,9	2,1	63,5	29,5	3,2	2,1
	221	8	7,6	166	116	12	8,4
	276	14,8	17,5	251	199,6	17,2	18,5
	329	30,8	21,3	494	332	49	37,9
	538	81,8	86,8	1 865	1 265	96,8	113
Nontissé aiguilleté PP	204	3,12	3,3	46	52,4	12,5	14,6
	274	6,2	10,4	89,6	102	12,7	17,2
	380	13,4	21,7	186	220	31,9	37,6
	600	54,8	98,6	438	576	65,3	76,8

PP = Polypropylène
PE = Polyéthylène
PET = Polyester

et $I = \frac{be^3}{12}$

d'où $G = \frac{F e^2}{12 \frac{\Delta L}{L}}$

L'équation dimensionnelle de module de flexion

$G = ML^2 T^{-2}$

Il s'exprime en N x m.

Si on considère $G = p \times c^3$, p est le poids surfacique du géotextile

L'équation dimensionnelle du module de rigidité est $Q = ML^{-1} T^{-2}$

Il s'exprime en N/m2.

2.4 Résultats

Dans le tableau 2 sont rassemblés les valeurs de longueur de courbure, les modules de flexion et le module de rigidité correspondants, obtenus sur cinq séries de géotextiles de masses surfaciques différentes et qui peuvent se répartir en trois classes. La première comprend deux nontissés aiguilletés de filaments continus, l'un de polyester, l'autre de polypropylène. La seconde est composée de deux nontissés de filaments continus thermosoudés, l'un de polypropylène, l'autre d'un mélange de polypropylène et polyéthylène. La troisième est un tissé de propylène.

Pour chaque type de géotextile, la longueur de courbure et le module de courbure sont des fonctions croissantes de la masse surfacique des articles. Les valeurs de C et G correspondant au sens production et au sens travers sont assez voisines pour les nontissés aiguilletés et pour 4 des 5 tissés étudiés. Les différences entre les valeurs obtenues pour les sens production et travers des nontissés thermosoudés sont beaucoup plus marqués. Pour le tissé ce fait peut être attribué au nombre de fils différent dans les sens chaîne et trame. Pour les thermosoudés, la différence provient sans doute du fait que ces matériaux rigides sont stockés en rouleau. Des mesures faires sur les mêmes éprouvettes après relaxation à plat pendant un mois n'ont cependant pas apporté de modification.

Les valeurs du module de rigidité q se situent dans une étroite fourchette pour les produits d'un type donné et de différentes masses surfaciques : par contre, elles varient de 1 à 100 entre les aiguilletés et les thermosoudés.

Les valeurs de modules de flexion obtenues à partir de la mesure de longueur de courbure sont comparées dans le tableau 3 avec celles calculées à partir des modules sécants et initiaux et sont respectivement appelés G_1 et G_2. On peut constater qu'il existe une bonne concordance entre les valeurs de G mesurées et celles obtenues à partir des modules initiaux alors qu'il apparaît des divergences importantes pour les valeurs de G mesurées et celles obtenues à partir des modules sécants dans le cas des matériaux nontissés aiguilletés et dans le cas des tissés.

3 CONCLUSIONS

Les travaux menés par l'INSTITUT TEXTILE DE FRANCE ont permis de proposer une méthode de caractérisation de la flexibilité des géotextiles. La détermination des modules de flexion permet une approche des modules initiaux des géotextiles qui sont généralement difficiles à atteindre par des essais de traction.

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A Strain-Gauge Technique for Measuring Deformations in Geotextiles

Une technique à jauge d'allongement pour mesurer des déformations des membranes

A method has been developed to determine strains and stresses in membranes under field conditions. Specific problems had to be overcome in relation to the high strains to be expected (in the range of ca. 10-20 percent). Some of the aspects involved are the choice of a suitable adhesive for glueing the strain-gauges to non-woven as well as to woven geotextiles, the connection of the measuring cable with the strain-gauge and the interpretation of the signal measured. As the response of the strain-gauge mainly depends on the stiffness ratio between the membrane and the strain-gauge, this value has to be determined experimentally for each type of membrane. The behaviour of the geotextile often depends on its loading.
Strain-gauge reading in practice have been simulated in the laboratory in order to determine the response of the strain-gauges as well as the stress-strain relationship of the geotextile material. As a whole, the measuring system has proved to be very reliable under practical circumstances, even when heavy loads or large deformations are applied.

Une méthode a été développée pour déterminer des déformations en membranes en situ. Des problèmes spécifiques devaient être surmontés à cause des déformations élevées (10-20 pourcents). Quelques aspects a examiner sont le choix d'une colle convenable pour attacher les jauges d'allongement aux membranes tissées ou non tissées, la connection du cable à mesure à la jauge d'allongement et l'interpretation du signal électrique mesuré. La réponse électrique de la jauge d'allongement dépend principalement du rapport de la rigidité de la membrane et celle de la jauge d'allongement; cette valeur doit être déterminée expérimentalement pour chaque genre de membrane. Le comportement de la membrane dépend de la séquence des déformations subie dans le temps. La séquence selon le mesurage à jauge d'allongement en pratique a été simulée au laboratoire afin de déterminer la réponse des jauges autant que le rapport contrainte/déformation des membranes. En conclusion: le système à mesure a montré sa sûreté en pratique. même si les charges ou les déformations sont elevées.

INTRODUCTION

Very soon after the start of the working group G2 of SCW (the Netherlands Road Construction Research Centre based at Arnhem, Holland) at the end of 1976, investigating the feasibility of the application of geotextiles in road construction, it turned out that the main problem in studying this subject, would be the selection of a measuring device to be used in field experiments for monitoring strains. An inventory was made of measuring methods already in use, but most of them did not seem to suit our purpose. It was concluded, however, that the application of strain-gauges could be considered as useful. Enka-Research (Arnhem-Holland) had some experience with a measuring method, using a high performance strain-gauge (strain capacity up to 20%), type EP-08-40, CBY-120, supplied by Micro-Measurements. A number of model experiments had been performed, and it turned out that this particular type of strain-gauges was suitable for usage in combination with coated fabrics. Unfortunately, glueing of the strain-gauges to geotextiles was found to be a problem.
Experiments, carried out in the Materials Science Laboratory at the Department of Civil Engineering in Delft, showed that a certain type of silicone adhesive could be applied (Terostat-33, manufactured by Teroson GmbH, Heidelberg, W. Germany).
This type of cement offers an extra advantage because it is waterproof.
The behaviour of the Terostat is purely elastic and vertical loads do not effect the results of the measurements.
Summarizing, the use of an appropriate strain-gauge technique has the following advantages over other methods:

- registration of strains up to more then 10 percent is possible
- there is little effect on the behaviour of the geotextile
- reading-out at great distance is possible
- it is independent of vertical pressures
- reliability for long periods of service life
- the handling is easy

Firstly, the method of glueing strain-gauges to geotextiles is described, followed by a report on the feasibility tests.
Because of the very high possible strains, the Wheatstone-bridge, used to measure the strains, is no longer linear. The increase of electrical resistance of the strain-gauge, usually negligible if strains are small, has now to be taken into account; the formulae for this case will be derived.
A simple laboratory test, verifying the sensitivity of the strain-gauge glued to different types of geotextiles will be discussed.
Finally some results of field tests will be presented.

APPLICATION METHOD

The application takes three steps.
The first one is the glueing of the strain-gauge to the geotextile; the second is the cable connection between strain-gauge and measuring cable; the final one is the protection of the strain-gauge, including the connections, against external influences, especially against water.

The silicone glue is highly elastic, and it follows the elongation of the geotextile very well, without exerting any force on it. On the other hand, a small force is needed to extend the strain-gauge. This force has to be transferred across the adhesive layer, causing deformations due to shear stresses. In order to keep these deformations as small as possible it is necessary to apply but a very thin layer. The smaller the deformation the better is the response of the gauge. When the modulus of elasticity of the geotextile material is very low, the force needed to strain the gauge strip will locally influence the behaviour of the geotextile.
This also means an indesirably decrease of its response.

In conclusion: optimum results are produced with an adhesive layer of minumum thickness in combination with a high modulus geotextile.
In order to minimize the scatter of the results for different installation method, the method should be standardised and moreover the operation should preferably be carried out by one and the same individual.
In our laboratory the following procedure is adhered to. Firstly apply a thin layer of the silicone glue to the geotextile. Next scrape off as much as possible. After having pressed a strain-gauge into the glue and having covered it by a thin teflon strip, the assembly is compacted by a dead weight for appr. 15 minutes. After this operation final curing of the glue takes 24 hours. After curing, a connection is made between the gauge and a terminal by means of thin wires. The terminal is glued to the geotextile in the same way as the gauge. In order to get a flexible connection the connecting wires are curled into loops (see fig. 1).

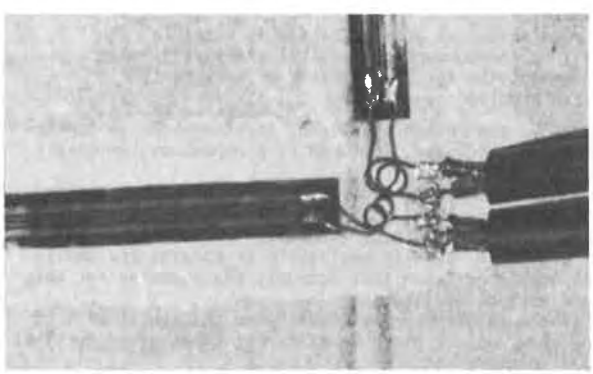

Fig. 1. Strain-gauges with connection cables. The actual gauge length is 10 cm.

Then to the other side of the terminal, wires with a length of appr. 10 cm are fixed. This complete assembly is then covered with silicone glue to prevent damage from its future environment.
It has to be stressed, however, that the silicone adhesive should be handled under perfectly dry conditions.

The handling in the field only consists of mechanically connecting the open ends between gauge wires and the cable to the measuring device.
A shrink lining is used to protect the connections from water.

FEASIBILITY TESTS

In order to test the effects of the hardening of the silicone glue, a number of strain-gauges were glued to a geotextile (stabilenka 200) and tested after different periods of time. As mentioned before, it should be borne in mind that due to the shear deformation of the glue and due to the interaction between geotextile and gauge the measured strain deviates from the deformation the geotextile is suffering. The rate of hardening of the adhesive is of importance with respect to these shear effects.
Table 1 gives the results of the tests performed. It can be concluded that the curing of the glue is complete within five days.

Table 1. Relation between strain-gauge value and actual strain of the geotextile, in dependency of the hardening time of the glue.

strain of geo-textile %	strain shown by strain-gauge, % — age of glue (days)							mean value	standard deviation
	13	12	11	10	9	6	5		
1		0.8	0.6	0.6	0.7	0.6	0.4	0.6	0.13
2	1.3	1.7	1.3	1.5	1.3	1.4	1.5	1.4	0.15
3	1.9	2.3	2.1	2.2	2.0	2.0	2.4	2.1	0.18
4	2.9	3.1	3.2	3.0	2.8	2.9	3.3	3.0	0.18
5	3.6	4.0	3.7	3.8	3.8	3.8	4.0	3.8	0.15
6	4.6	4.9	4.7	4.6	4.6	4.6	5.0	4.7	0.17

In order to investigate whether there is any influence of creep phenomena of the glue, a strip of a geotextile, instrumented as described was stretched to about 5%. After three months the strain-gauge reading had not changed.
In order to test the sensitivity for temperature changes, a test specimen was kept in an oven at about 100^{o}C, and in a refrigerator at about -20^{o}C. At the same time, the dummy strain-gauge was kept at a temperature of about 20^{o}C. During these tests no temperature influence was found.
To test the sensibility for vertical load, a strain-gauge has been loaded with 20 N/cm^2. The measured strain results were not affected.
We can conclude from all the performed tests, that the chosen method suits our purpose.
The outdoor performance had to be tested under field conditions. The method has now been improved up to a final state that allows of reliable strain measuring in practice.

THE WHEATSTONE-BRIDGE MEASURING HIGH STRAINS

The principle of strain measurement by means of a strain-gauge is well-known. The phenomenon can be described as follows. A relative change of strain of the strain-gauge material results in a relative change of electrical resistance. The ratio k, between deformation and change of resistance characterizes the gauge, namely:

$$\frac{\Delta R}{R} = k . \frac{\Delta l}{l}$$

$$\varepsilon = \frac{\Delta l}{l}$$

$$\frac{\Delta R}{R} = k . \varepsilon \qquad (1)$$

where R = electrical resistance
ΔR = change of resistance
ε_s = relative strain

To accurately measure the resistance changes in the gauge that are usually very small indeed, a Wheatstone bridge is applied.
Even a minimum change of resistance produces a reasonably measurable change of electrical voltage between the points C en D (see fig. 2).

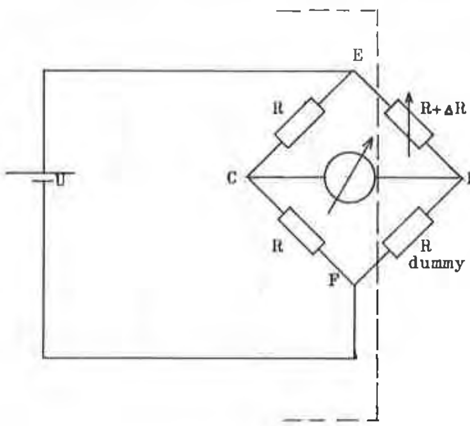

Fig. 2. Wheatstone bridge.

In fig. 2 the measuring device (Wheatstone bridge) is schematically depicted at the left of the depicted line. The dummy gauge is of exactly the same kind as the measuring one itself. It has been handled in exactly the same way and it is kept under the same environmental conditions of temperature and moisture.
At the start of the measurement, when the stain of the gauge equals zero, the measuring bridge is in equilibrium. That means that $U_c = U_d = \frac{1}{2}U$ and $U_{cd} = o$.

When strained to ε, R changes by ΔR
Then $R_{ED} = R + \Delta R$, $R_{DF} = R$

$$U_D = \frac{R_{ED}}{R_{ED}+R_{DF}} \cdot U \text{ and } U_c = \tfrac{1}{2}U$$

so,

$$U_{cd} = \left(\frac{R + \Delta R}{2R + \Delta R} - \tfrac{1}{2}\right) \cdot U$$

Substituting eq. 1 results in

$$U_{cd} = \left(\frac{1 + k.\varepsilon}{2 + k.\varepsilon} - \tfrac{1}{2}\right).U = \frac{k.\varepsilon}{4 + 2k\varepsilon} \cdot U$$

$$(2)\quad \varepsilon = \frac{4\,U_{cd}}{k(U - 2U_{cd})}$$

This means that in the case of very high strain the relationship between U_{cd} and ε_s is not linear anymore. Generally, however, if U_{cd} is negligible compared to U, eq. 2 transforms into:

$$(3)\quad \varepsilon = \frac{4\,U_{cd}}{k.U}$$

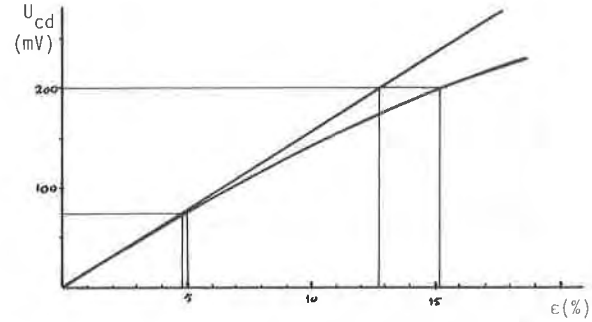

Fig. 3. Non-lineair relationship between U_{cd} and ε for high strains.

The approximation resulting in eq. 3 is only valid for strains less than ca. 5%. In the case of higher strains, eq. 2 has to be used to accurately calculate strain from measured voltage differences.

LABORATORY TESTS ON GEOTEXTILES

Because of the dependency of the strain-gauge response on the type of geotextile as well as on the bonding method of the strain-gauge to the geotextile, it is necessary to perform a gauging test for each type of geotextile.

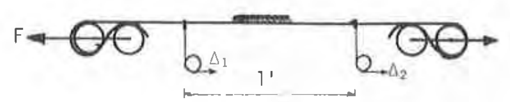

Fig. 4. Schematic set-up of a gauge test

During loading of the specimen, the strain is measured in two different ways.
The real strain of the geotextile is determined by means of two displacement transducers, placed at a distance l'. The difference between the displacements Δ_1 and Δ_2, divided by the distance l' give the real strain ε_i.

$$\varepsilon_i = \frac{\Delta_1 - \Delta_2}{l'}$$

This real strain value is compared with the strain determined by the strain-gauge, according to equation 2. Simultaneously the force F is being recorded.
Simultaneous determination of ε_i, ε_s and F produces the the stress-strain relationship, as well as the relation between ε_i and ε_s. The ratio $\varepsilon_s/\varepsilon_i$ yields the gauge response.
An example of the results of a test as described above for a strain-gauge bonded to a woven polyester fabric (stabilenka zoo) is given in fig. 5.

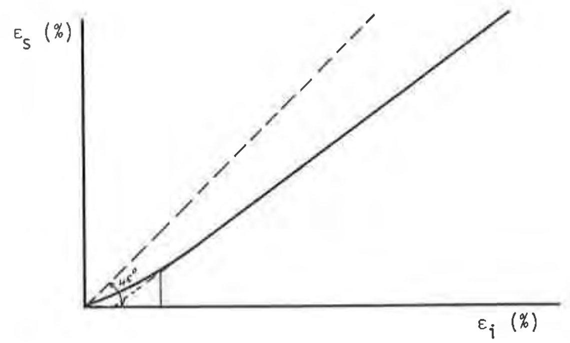

Fig. 5. Relationship between geotextile strain ε_i and strain-gauge value ε_s.

In this particular case the relation was found to be:
$\varepsilon_i = 0.32 + 1.17\ \varepsilon_s$.
The correlation coefficient between the experimental results and the calculated linear approximation is 0.9995. That means that the linearized measuring method produces satisfactory results.
Moreover, the response in this case amounts to more than 80%, what in our opinion is a very good result.
As to non-wovens, the results are less satisfactory, due to their low Young's modulus, as well as to their time dependent behaviour. Because of the non-elastic behaviour it is obvious for example that the relation between applied geotextile strain and measured strain is dependent on the strain rate. For a slow, continually rising strain, the ε_i - ε_s relation for a non-woven polyester (e.g. Colbond) is found to be:
$\varepsilon_i = 1.38 + 1.26\ \varepsilon_s$.
So for this material a strain-gauge response of appr. 70-75 percent has been achieved.
These are only a few examples. It is obvious that the results of the strain-gauge are very satisfactory in combination with a polyester membrane.
Whether the adhesive, i.e. the silicone type used, is also suitable for bonding of gauges to other materials, such as polypropylene and polyamide, has not been investigated up to now.
It has to be emphasized, that the ε_i - ε_s relationship gaven are only examples. The exact relationship has to be determined for each combination apart.

STRAIN-GAUGE TECHNIQUE IN FIELD EXPERIMENTS

Two types of field experiments have been conducted. The first one is the application of a woven geotextile between an embankment and a soft subsoil.
In Cuxhaven, West-Germany in 1978 40 gauges have been applied. Deformations in the fabric were very low but nevertheless it could be concluded that the developed strain-gauge technique was reliable under these field conditions. Even after a year 37 gauges were still in a good shape.
In Almere, The Netherlands in 1979 a similar field experiment was performed.
In this case the imposed deformation of the geotextile was so large that tearing occured at a strain value of about 10 percent. The registered maximum strain value however was approximately 8 percent, due to premature failure of the connections between measuring cable and strain-gauge.
In Cuxhaven as well as in Almere strain registration took place by means of a 40-channel scanning measuring bridge. The scanning method was adequate because of the very slow developments of deformation during the construction of the embankment.
An improved connecting method, by means of the earlier mentioned curled wires between the strain-gauge and a terminal, was for the first time applied in Nieuwerkerk a/d IJssel, The Netherlands in 1980 during second type of experiment, namely the application of non-wovens under a road foundation. Measuring of strains took place in several sections during the time that foundation material was transported to the site by motorlorries.
Hence the loading was dynamic and strains have been continuously registered by means of recorders.
Dynamic strains amounting to 13 percent have been recorded. An example of a recorder trace is given in fig. 6.

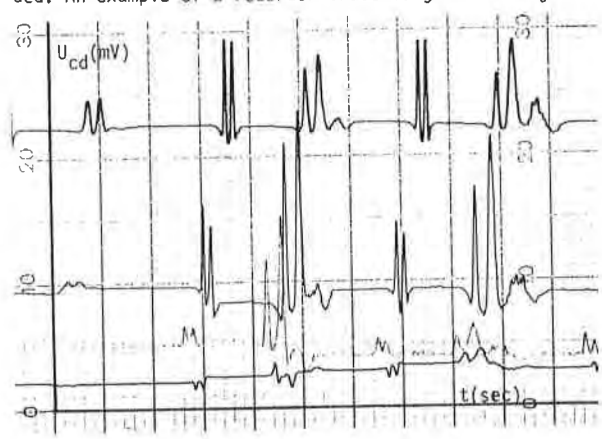

Fig. 6. Recorder trace of gauge value.

The value of U_{cd} changing with time, was afterwards simulated in a laboratory test, yielding the real geotextile strains as well as the corresponding forces.

It can be concluded that the developed measuring method is very well suited to the purpose of recording strains up to ca. 15% under field conditions.
The method can be used in cases where strain monitoring during construction is required to allow of assessing the permissible construction rate.

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Laboratory Testing of Geotextile Fabric Filters

Essai laboratoire des géotextiles filtrants

Geotextile filters are being utilized without a true evaluation of their flow capacity or filtration abilities for field application conditions. Laboratory testing should reflect properties of importance for field applications rather than those associated with theoretical analyses. At the same time, these properties must be consistent with the theoretical concepts involved. With this in mind, Caltrans has developed tests for evaluating Geotextiles for site specific applications. The tests evaluate: "Flow Capacity," a parameter similar to permeability that assumes each fabric has unit thickness; and "Plugging Flow Capacity," a parameter for evaluating filtration potential and soil fabric interaction flow capability. Testing utilizes 6 inch diameter samples with 5 inch diameter test areas and a horizontal parameter. Fabrics are evaluated directly with soil or by using glass beads to simulate representative soil sizes. The methods, their evolution, and some example data are presented.

On utilise des geotextiles filtrants sans evaluer leur capacites du flux avec les donnees issues des chantiers. Est plus utile que les essais du laboratoire mesurent les qualites utiles pour l'application en site que les proprietes associes avec l'analyse theoretique. A la meme les proprietes doit etre d' accord avec la theorie. Selon que CALTRANS a formule des essais pour evaluer les geotextiles avec les donnees issues des chantiers. Les essais mesurent capacite du flux, une parametre comme la permeabilite (par presomption que tous les textiles sont d'epaisseur uniform) et la capacite du flux tamponne la parametre pour evaluer la capacite filtrante potentielle et la capacite au flux du textile en contact du sol. Les essais utilisent des echantillons (au diametre du 15.24 cm) avec un espace du 12.6 cm pour mettre le textile a l'epreuve et un parametre horizontal. On analyse les textiles avec le sol ou des perles du verre pour simuler le granule normal du sol. On presentera ici la methode et son developpement et quelques donnees du cas.

Because of the tremendous potential for application of geotextiles as filter elements in drainage facilities, a method of evaluating them is necessary. Dissatisfaction with the available test procedures and their results led to the establishment of the test methods herein described. The factors of primary interest when using any filter system are its flow capabilities and its filtration characteristics. The filtration ability and the potential for plugging or clogging as well as the water flow capacity are indicators of the potential effectiveness.

Because fabrics are normally to be used in a single layer and the fact that they are thin and easily deformable, their true permeability is of less importance to their application than is their flow capacity per unit area, hereafter called Flow Capacity. In an effort to devise tests that would have direct applicability, the Flow Capacity and Plugging Flow Capacity tests were developed. The Plugging Flow Capacity measures the potential for filtration and reduced flow capabilities in conjunction with glass beads; for specific job applications the subject soils may be used in place of the beads.

The sample size and the initial testing techniques were selected to be compatible with existing soil testing criteria. Even with this rather large scale of testing, a five inch diameter sample, the nonwoven fabrics seem to exhibit a high degree of variability. Adjacent samples cut from a large fabric panel may exhibit Flow Capacities varying by hundreds of percent. The fabrics do, however, appear to vary within the same order of magnitude. While this kind of variation on an absolute materials comparison basis is normally unacceptable, when dealing with soils permeability, variations of an order of magnitude are commonplace. Thus, if the fabrics are dealt with on an order of magnitude basis, as are soils, the variations associated with them are acceptable. Using an order of magnitude acceptance criteria, the test for Flow Capacity and Plugging Flow Capacity are suitable measures of the potential of fabrics, both with regard to flow and filtration capabilities. The methods provide for relatively rapid evaluations of fabrics as well as offering a basis for comparing the types of fabrics available. The test methods were initially developed for use with nonwoven fabrics. They have, however, been used to compare some woven and some hybrid fabrics as well.

The test procedures are designed to evaluate the fabrics properties when subjected to the least desirable installation conditions, i.e., with the downstream side of the fabric exposed to open air rather than submerged or in contact with drainage aggregate. The application of results from these test methods, in conjunction with measured soil permeabilities, should enhance the objective design of fabric filtration applications. In general, the Flow Capacity and the Plugging Flow Capacity should both be one order of magnitude greater than the permeability of the soil to be drained. Additionally, the 85% size of the soil should be filtered and the 50% and 15% sizes should not produce greater head rise than the 85% size.

Application of these test methods and their results will require site specific design. The manufacturers should, however, be able to provide tables of information listing the Flow Capacity and Plugging Flow Capacities, by grain sizes, for their fabrics thus permitting application of the fabrics in many cases from manufacturers tables and the soil mechanical analysis and permeability. Additional experience, by others, with these test methods and their applications would be of great value in determining their value in the geotextile industry. To date, only one researcher has evaluated the system and the applicability of the results. In my opinion, the results and their application are superior to the EOS test method and its application.

GEOTEXTILE FABRIC FLOW CAPACITY

The following test method is for evaluating the flow characteristics of geotextile fabrics for use in subdrainage applications. The Flow Capacity is a measure of the water flow capability consistent with the application configuration. Where permeability measures an absolute flow potential, Flow Capacity evaluates flow capability under operational conditions.

Apparatus and Materials Required

1) Horizontal permeameter, with sample holder, see Figure A-1.
2) High volume, 25 gpm, and low constant pressure, 2 psi, water source.
3) Spirit level.
4) Constant elevation head reference.
5) Scale, 12 inch, to measure distance from reference to both the entrance head and exit head to nearest 0.1 inch.
6) Flow evaluation media, a stopwatch and calibrated catch bucket.
7) Record sheets.
8) Wrenches and screwdriver for assembly and dismantling.
9) Three fabric samples six inches in diameter.

Procedure

Install the first test sample in the concentric ring compression retainer as illustrated in Figure A-2. Assemble the permeameter and sample holder as shown in Figure A-1. Position the permeameter under the water source and above the drain facility, Figure A-3. Level the downstream channel, the portion through which the water leaves the fabric. Initiate water flow in sufficient quantity to completely submerge the inlet side of the fabric sample, leaving ample freeboard. Adjust flow to achieve a steady head.

When steady state flow is achieved, the Flow Capacity can be evaluated. Both the inlet head, H1, and the outlet head, H2, are measured as distances from the constant elevation reference. Record as H1(i) and H2(i) on the data sheet, where i is the sample number, i.e., H11, H12, etc.

Using the calibrated catch bucket, intercept the outlet flow and measure the time required to fill the bucket. Do this three times and record the average value as T(i).

Repeat the above procedure for the second and third samples.

Record the calibrated bucket volume, Q, in gallons and the sample test diameter, D, in inches, the permeameter inside diameter at the sample.

Calculations

The Flow Capacity is the average value obtained using the three test runs. First calculate the individual Flow Capacities, FC(i), then average these values to determine FC, the Flow Capacity.

Formulae:

$$H(i) = H1(i) - H2(i)$$
$$A = D^2/4$$
$$CF = 1.66 \times 10^6 \text{ for feet/day}$$
$$CF = 2.54 \text{ for centimeters/second}$$
$$FC(i) = \frac{Q(CF)}{T(i)\ H(i)A}$$
$$FC = \frac{FC(1) + FC(2) + FC(3)}{3}$$

i = the sample number

Reporting

Report all three FC(i) values and the FC value so the variation is apparent. Also list the respective H values and the cloth brand.

Example

Fabric Type: Example

H1(i)	H2(i)	T(i)	H(i)	FC(i)
10.7	0.75	29.0	9.95	1617 ft/day
10.55	0.80	28.1	9.75	1703
11.25	1.25	31.1	10.0	1500

Q = 5.41 gallons

D = 4.95 inches

$$FC = \frac{1617 + 1703 + 1500}{3} = 1607$$

PLUGGING FLOW CAPACITY

The following test method is for evaluating the filtration and plugging potential of geotextile fabrics for use in subdrainage systems. This test is an extension of the Flow Capacity test. This test utilizes glass beads to represent soils requiring drainage and filtration. If the D_{50} and/or D_{85} soil grain sizes are larger than a sieve size range of 60 to 100 and the soil is well graded, the filtration parameter need not be considered for the D_{50} size, only the fabric plugging by the D_{15} size and the filtration of the D_{85} size. The D_x grain size is the size D for which x% of the soil is finer than D. If all soil particles are greater than a 60 sieve size and the soil is well graded, then filtration is probable and plugging unlikely with the currently available fabrics. Particles finer than the 325 size, clays, normally do not readily dissociate and therefore the available nonwoven fabrics should also work with those soils that are essentially composed of these very fine clay particles. The Plugging Flow Capacity should be evaluated but will not reflect the cohesion and aggregation associated with these soils so may disallow a suitable fabric. The size ranges above and below the 100 to 400 sieve sizes can be evaluated when appropriate size beads are available.

Apparatus and Materials Required

1) Horizontal permeameter, with sample holder, see Figure A-1.
2) High volume, 25 gpm, and low constant pressure, 2 psi, water source.
3) Spirit level.
4) Constant elevation head reference.
5) Scale, 12 inch, to measure distance from reference to both the entrance head and exit head to the nearest 0.1 inch.
6) Flow evaluation media, a stopwatch and calibrated catch bucket.
7) Record sheets.
8) Wrenches and a screwdriver for assembly and dismantling.
9) Three samples of the fabric to be tested, six inches in diameter.
10) Glass beads, sieve size ranges: 60 x 100, 70 x 140, 100 x 200, 140 x 270, 200 x 325, 270 x 1000.

Procedure

Install the first sample in the concentric ring compression retainer as illustrated in Figure A-2. Assemble the permeameter and sample holder as shown in Figure A-1. Position the permeameter under the water source and above the drain facility, Figure A-3.

Level the downstream channel, the portion through which the water leaves the fabric. Initiate water flow in sufficient quantity to completely submerge the inlet side of the fabric sample, leaving ample freeboard. Adjust the flow to achieve a steady head.

When steady state flow is achieved, the Flow Capacity can be approximated. Both the inlet head, H1, and the outlet head, H2, are measured as distances from the constant elevation reference. Record as H1 and H2 on the data sheet.

Using the calibrated catch bucket, intercept the outlet flow and measure the time required to fill the bucket. Do this three times and record the average value as T.

After establishing the untreated Flow Capacity, glass beads are introduced into the flow stream. Wash five grams of the glass beads which span the D_{85} soil size into the flow stream as illustrated in Figure A-4. Adjust to steady state flow, if necessary, and remeasure and record H1(i), H2(i) and T(i), where (i) is the total weight of glass beads introduced; i=5, 10 or 15; i.e., H1(5), H2(5), T(5), H1(10), etc. Add a second five gram dose of D_{85} size beads. After steady state is again achieved, remeasure and record H1(i), H2(i) and T(i). Add another five grams of the D_{85} size beads and adjust to steady state flow remeasuring and recording H1(i), H2(i) and T(i). (Addition of more than 15 grams of beads becomes a test for bead permeability, not fabric Plugging Flow Capacity.) Repeat this entire procedure including the untreated Flow Capacity for the D_{50} and D_{15} glass bead sizes. Use separate record sheets and fabric samples for each size of glass beads.

Record the calibrated catch bucket volume, Q, in gallons, and the sample test diameter, D, in inches, the permeameter inside diameter at the sample.

Calculations

The Flow Capacity, FC, is approximated using H1, H2 and T in the PFC(i) equation in place of H1(i), H2(i) and T(i), respectively. The Plugging Flow Capacity, PFC, is calculated for each bead size and designated by the soil size represented. A PFC_{15}, a PFC_{50} and a PFC_{85} are calculated. The PFC value used is the minimum calculated for the given bead size. Three PFC values are calculated, one for each five gram addition of beads, for each grain size tested.

Formulae:

$$H(i) = [H1(i) - H2(i)]$$
$$A = D^2/4$$
$$CF = 1.66 \times 10^6 \text{ for feet/day}$$
$$CF = 2.54 \text{ for centimeters/second}$$
$$PFC(i) = \frac{Q(CF)}{T(i)\,H(i)A}$$

PFC_{15} = the minimum value of PFC(i) when using the D_{15} grain size equivalent glass beads.
PFC_{50} = the minimum value of PFC(i) when using the D_{50} grain size equivalent glass beads.
PFC_{85} = the minimum value of PFC(i) when using the D_{85} grain size equivalent glass beads.
i = the sample identification, the total weight of glass beads added to that time (5, 10 or 15).

Reporting

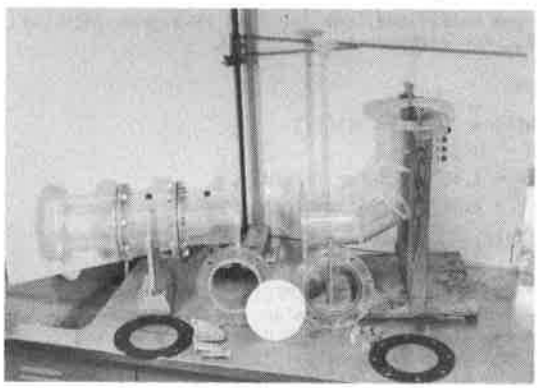
The PFC_{15}, its respective PFC(i) values and corresponding heads should be listed. A similar listing for the PFC_{50} and PFC_{85} should also be reported as well as the FC approximations. The D_{15}, D_{50} and D_{85} size glass beads used should be listed and the fabric type and brand should be presented.

Example

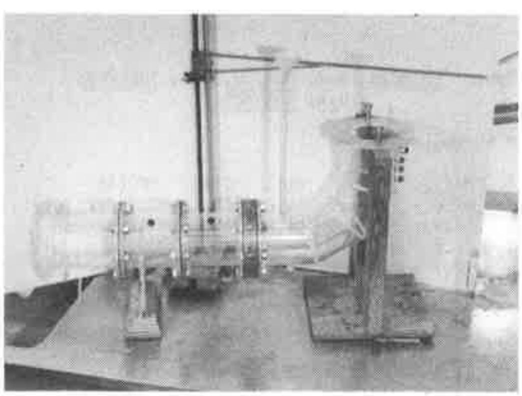
Fabric Type: Example, D_{85}, 60-100; D_{50}, 100-200; D_{15}, 200-325

PFC(o) = FC approximation

	i	H1(i)	H2(i)	H(i)	T(i)	PFC(i)
D_{85} Size	0	17.6	12.4	5.2	15.4	5827 ft/day
	5	17.6	10.6	7.0	15.4	4329
	10	17.7	8.2	9.5	15.7	3129
	15	17.7	6.1	11.6	16.3	2468
						PFC_{85} = 2468 ft/day
D_{50} Size	0	17.7	12.8	4.9	16.0	5952
	5	17.7	9.4	8.3	16.4	3428
	10	17.7	5.1	12.6	20.5	1807
	15	17.85	7.8	10.05	49.0	948
						PFC_{50} = 948 ft/day
D_{15} Size	0	17.7	12.4	5.3	15.5	5680
	5	17.7	9.1	8.6	16.0	3391
	10	17.6	5.7	11.9	20.9	1876
	15	17.8	6.1	11.7	43.8	910
						PFC_{15} = 910 ft/day



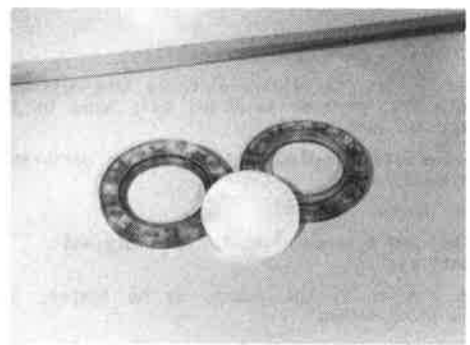
Permeameter with sample holder, gaskets and sample, the white disc.



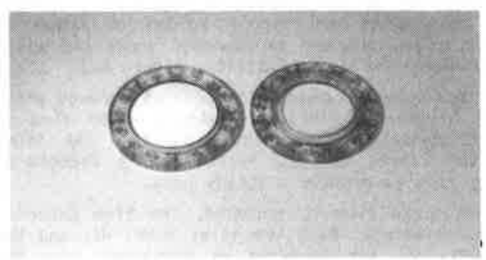
Fully assembled permeameter and sample holder.

PERMEAMETER WITH SAMPLE HOLDER

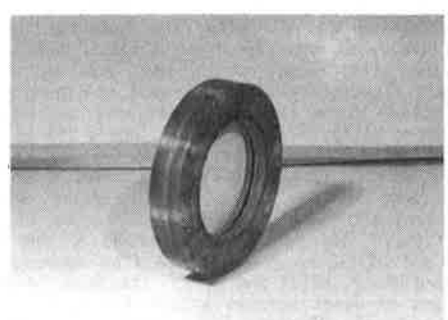
Figure A-1



Two piece sample holder with sample, white disc.



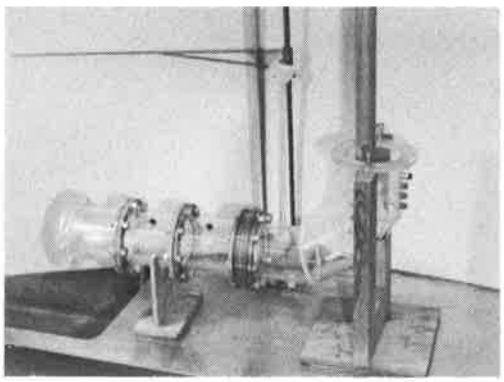
Sample holder with sample inserted into left hand section.



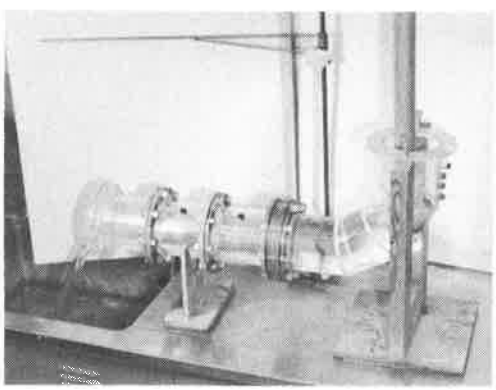
Fully assembled sample holder with sample.

SAMPLE HOLDER

Figure A-2



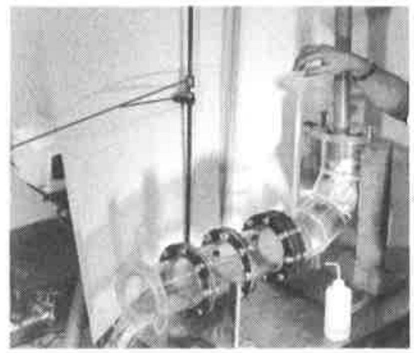
Permeameter positioned over drain and under water inflow pipe.



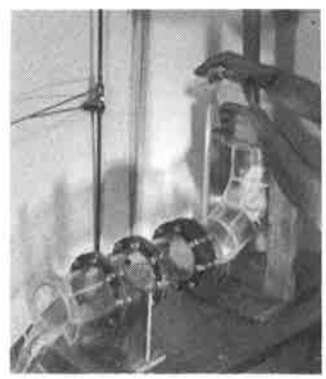
Permeameter with a steady state flow condition.

WATER INLET AND DRAINAGE FOR PERMEAMETER

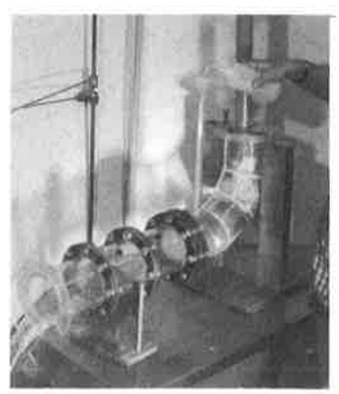
Figure A-3



Pouring in premeasured glass beads.



Washing beads out of vial.



Thoroughly washing funnel and introduction tube to assure total bead introduction.

GLASS BEAD INTRODUCTION

Figure A-4

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Results of Permeameter Tests on Statically Loaded Geotextiles

Résultats d'essais de perméabilité sur des géotextiles chargés statiquement

A special permeameter has been developed and constructed for the determination of the permeability both normal to the plane and in the plane of geotextiles. A description of the apparatus and the findings of an earlier test series is given in brief. The main part of the paper describes the results of permeameter tests performed on three geotextiles loaded up to 800 kN/m^2. The tests for flow normal to the fabric have been evaluated in terms of the permittivity. Similarly the tests for flow in the plane of the fabric are presented in terms of the transmissivity. In the examined stress range the permittivity varies considerably more for the needle-punched geotextiles than for the spun-bonded geotextile. It is also interesting to note that the coefficient of permeability normal to the plane and in the plane is the same for the needle-punched geotextiles, but that for the spun-bonded geotextile a significant difference exists according to the test results.

On a développé et construit un perméamètre spécial pour la détermination de la perméabilité normale au plan et dans le plan des géotextiles. La description de l'appareil et les résultats d'une première série d'essais sont donnés brièvement. La partie principale de l'article décrit les résultats d'essais de perméabilité sur 3 géotextiles chargés jusqu'à 800 kN/m^2. Les essais pour un débit normal au plan du géotextile ont été évalués en terme de permittivité. De manière similaire, les essais pour un débit dans le plan du géotextile sont présentés en terme de transmissivité. Dans le domaine des pressions examiné, la permittivité varie considérablement plus pour les géotextiles aiguilletés que pour le géotextile thermosoudé. Il est aussi intéressant de noter que le coefficient de perméabilité normal au plan et dans le plan est le même pour les géotextiles aiguilletés, alors qu'il existe une différence significative pour le géotextile thermosoudé comme le montrent les essais.

INTRODUCTION

A number of interesting contributions to the International Conference on the use of fabrics in geotechnics, Paris, 1977, dealt with the permeability of geotextiles and the development of testing procedures for its determination (e.g. 1, 2, 3, 4, 5). Meanwhile, a good deal of additional research has been done in several institutions and laboratories and much progress has been made in this specific area (6, 7, 8, 9).

At the Institute of Foundation Engineering and Soil Mechanics, ETH Zurich, preliminary investigations on the determination of the permeability of geotextiles started in 1978 (10, 11). These investigations were undertaken in cooperation with the Swiss Technical Commission on Geotextiles and arrived at formulating the requirements for a special permeameter. The performance of this special permeameter as an investigation tool should cater for the following requirements:

1. Reproducible test results.
2. Tests with flow of water normal to the plane and in the plane, the latter if possible in both directions, without removing the applied pressure on the sample or disturbing it.
3. To run tests both without and with soils.
4. To perform the test under static load, measuring the deformations.
5. Possibility of testing woven as well as nonwoven or compound geotextiles.
6. Size of the specimen of about 0.01 m^2 or larger.
7. Hydraulic gradients up to 50.

The apparatus was designed and constructed at the Institute of Foundation Engineering and Soil Mechanics, ETH Zurich and the first tests were performed in 1980.

APPARATUS AND PRELIMINARY TEST RESULTS

In figure 1 an overall view of the apparatus is given and in figure 2 a schematic diagramme is shown. A more detailed description of the apparatus is contained in (10, 11), and it is therefore limited to some of the essential features in this paper. The apparatus is based on a closed, temperature controlled circuit using demineralized deaired water which allows a flow both normal to the plane and in the plane of the geotextile. In the permeameter shown in figure 2 the rectangular geotextile sample is loaded between two pistons. The pistons contain filter plates made of quartz sand. The hydraulic head above and below the geotextile is measured by a total of eighteen pairs of piezometers.

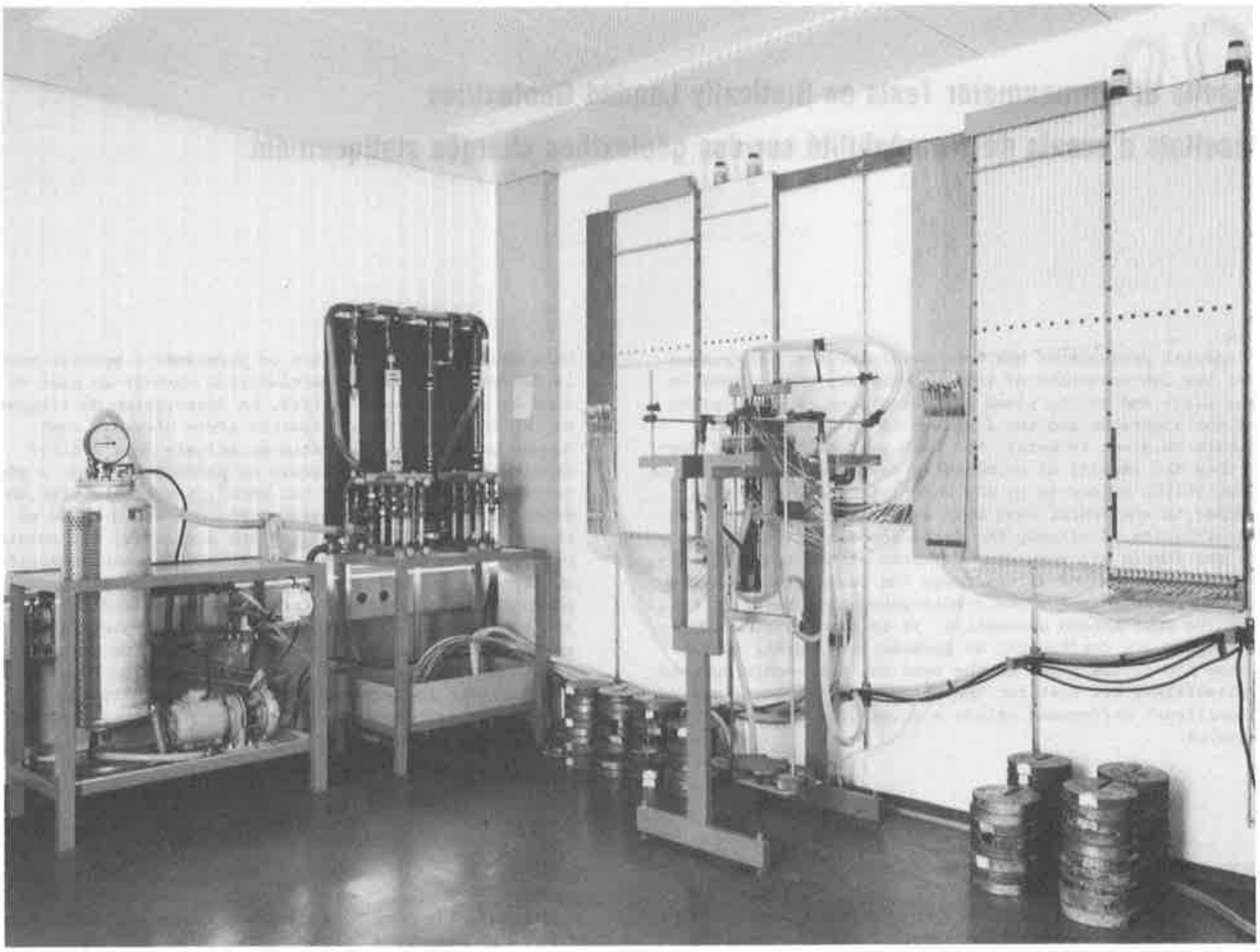

Fig. 1 Apparatus for the investigation of the hydraulic properties of geotextiles.

Fourteen pairs are located inside the area of 0.01 m^2 through which the vertical flow passes; four pairs are located outside this area and are used for the horizontal flow only.

Preliminary tests (10, 11) have shown that the coefficient of permeability is dependent on the hydraulic gradient. It is therefore important to perform tests and to compare test results at small hydraulic gradients as they are usually met in practical applications of geotextiles. Further, tests both for the permeability normal to the fabric and in the plane of the fabric have shown that the flow is significantly influenced by the normal stresses which are applied to the fabric.

PERMEAMETER TESTS NORMAL TO THE PLANE OF THE GEOTEXTILE

Three typical non-woven geotextiles have been selected for these tests. They may be characterized as follows:

geotextile 1: μ = 270 g/m^2, 100% polyester, needle-punched, unmodified, continuous filament.

geotextile 2: μ = 250 g/m^2, 100% polyester, needle-punched, staple fibre felt, resin bonded.

geotextile 3: μ = 200 g/m^2, 100% polypropylene, spun-bonded, continuous filament.

In order to assure small hydraulic gradients and laminar flow several sheets of geotextiles (up to 10) were placed in the permeameter. The tests were performed at effective normal stresses in the range of 20 kN/m^2 to 800 kN/m^2. At each pressure at least five determinations were made and the mean value was calculated. The results were obtained in terms of the permittivity according to

$$\Psi = \frac{q}{\Delta h \cdot A} \quad [s^{-1}] \qquad (1)$$

where: q = rate of discharge; Δh = loss of head per sheet of geotextile; A = area for normal flow (0.01 m^2).

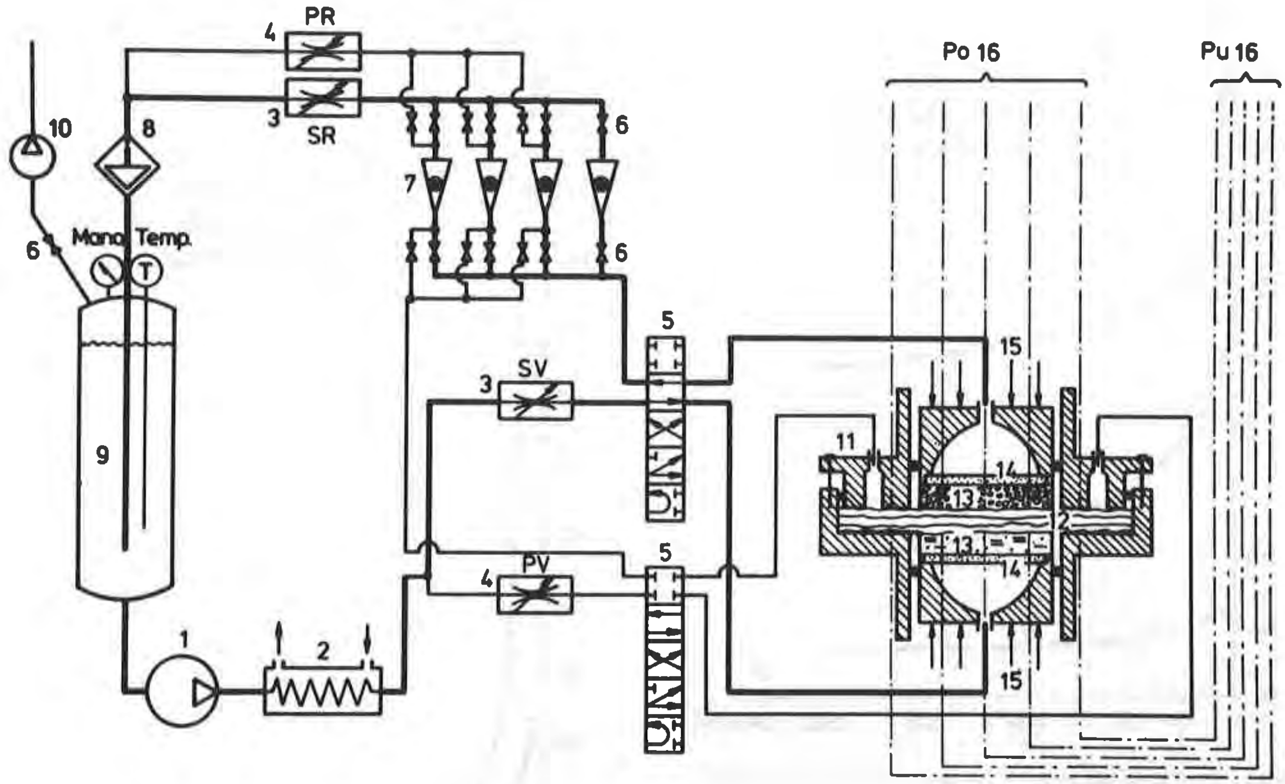

Fig. 2 Schematic diagramme of the apparatus. 1, water pump; 2, heat exchanger with thermostatic control; 3, throttle valve, flow normal to the plane of the fabric: SV, feed line; SR, return; 4, throttle valve, flow in the plane of the geotextile: PV, feed line: PR, return; 5, control valve; 6, shut-off valve; 7, flow meter; 8, filter for flushed out soil particles; 9, air chamber; 10, vacuum pump; 11, permeameter; 12, geotextile; 13, soil samples a and b; 14, filter plate; 15, applied static loading; 16, piezometer measuring tubes: Po, above the geotextile; Pu, below the geotextile.

From the permittivity the normal permeability k_{no} has been calculated:

$$k_{no} = \Psi \cdot T_g \qquad [ms^{-1}] \tag{2}$$

where: T_g = thickness of geotextile (one sheet).

The values of the permittivity Ψ are plotted as a function of the effective vertical stress σ' (Fig. 3). The corresponding k_{no} values are given in Figs. 5, 6 and 7. From the values given in the graphs the following variations in the permittivity and in the normal permeability may be calculated in the observed stress range (20 - 800 kN/m^2).

Table 1. Variations in the permittivity and in the coefficient of normal permeability.

geotextile	variation in Ψ	variation in k_{no}
1	2.4	6.0
2	1.9	6.1
3	1.7	2.7

Two observations can be made from table 1: The variations in Ψ and in k_{no} are larger for the needle-punched geotextiles 1 and 2 than for the spun-bonded geotextile 3. Secondly, the variation in Ψ is considerably smaller than the variation in k_{no}. Therefore, the permittivity can better be considered as a constant material property over a given range of normal stresses than the coefficient of normal permeability.

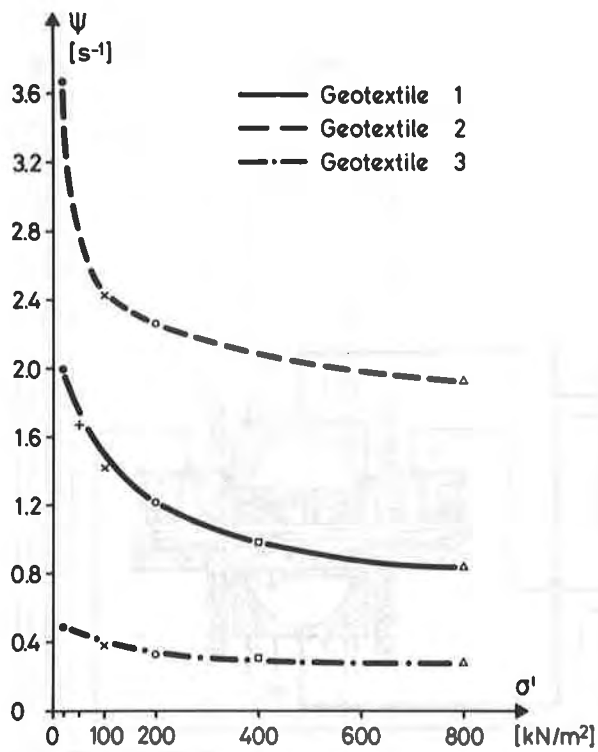

Fig. 3 Permittivity Ψ as a function of normal stress σ'.

PERMEAMETER TESTS IN THE PLANE OF THE GEOTEXTILE

These tests were performed on samples with a loaded breadth of about 230 mm and a length of 80 mm, and again on several sheets of geotextiles. For each of the investigated stresses in the range of 20 kN/m^2 to 800 kN/m^2 at least five determinations were made and the average values calculated. The test results are plotted in terms of the coefficient of permeability in the plane of the geotextile k_{po} in Figs. 5, 6 and 7.

The transmissivity has been evaluated according to

$$\theta = \frac{q \cdot L}{\Delta h \cdot B} \qquad [m^2\ s^{-1}] \qquad (3)$$

where: q = rate of discharge per sheet of geotextile; Δh = loss of hydraulic head; L = length of the flow path; B = breadth of the geotextile.

The coefficient of the permeability in the plane of the geotextile has been obtained from

$$k_{po} = \frac{\theta}{T_g} \qquad [m\ s^{-1}] \qquad (4)$$

where: T_g = thickness of geotextile (one sheet).

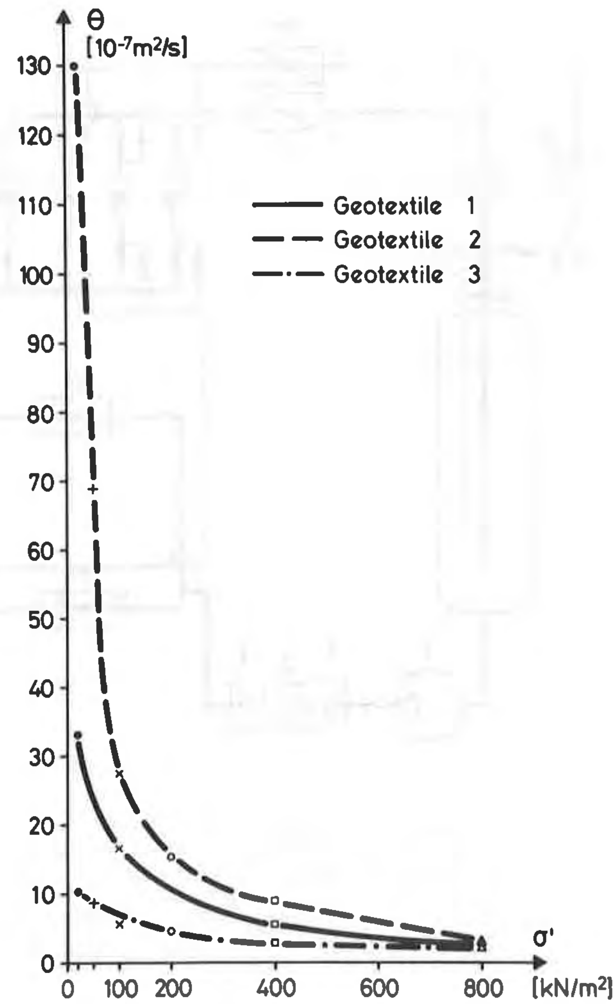

Fig. 4 Transmissivity θ as a function of normal stress σ'.

From the plotted values the following variations may be calculated:

Table 2. Variations in the transmissivity θ and in the coefficient of the permeability in the plane of the geotextile.

geotextile	variation in θ	variation in k_{po}
1	11.8	5.5
2	44.8	13.7
3	5.2	4.3

The influence of the normal stress acting on a geotextile is most significantly shown by the transmissivity, and it is quite obvious that the influence of the normal stress on the transmissivity may not be neglected. This influence is more pronounced for the needle-punched geotextiles 1 and 2 than for the spun-bonded geotextile 3.

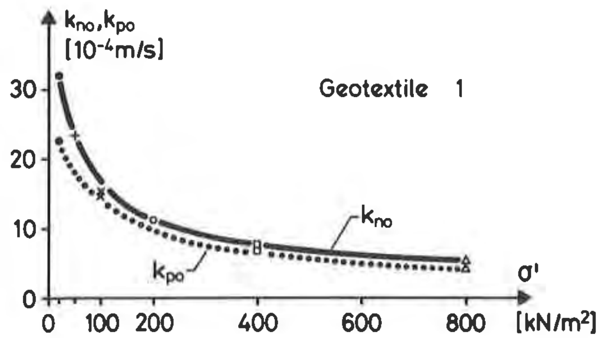

Fig. 5 Permeability k_{no} and k_{po} as a function of normal stress σ', geotextile 1.

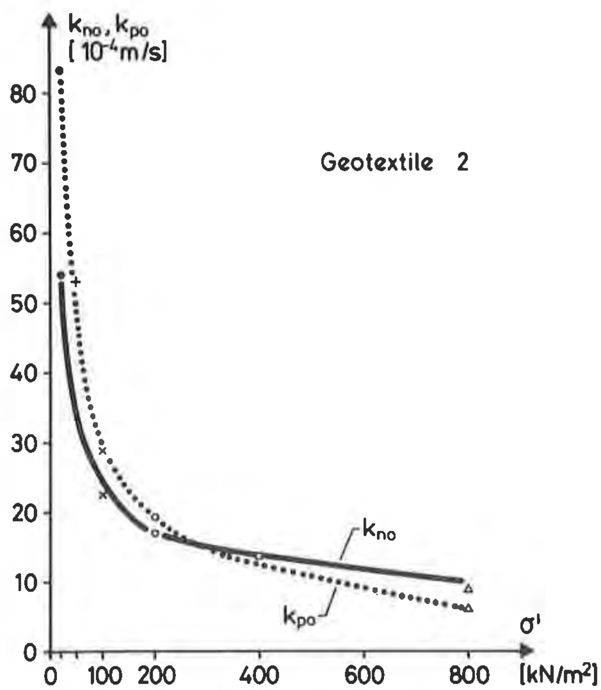

Fig. 6 Permeability k_{no} and k_{po} as a function of normal stress σ', geotextile 2.

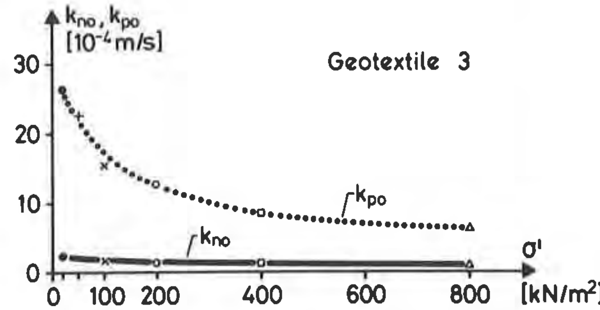

Fig. 7 Permeability k_{no} and k_{po} as a function of normal stress σ', geotextile 3.

CONCLUSIONS

From the performed permeameter tests normal to the plane and in the plane of the geotextile the influence of the normal stress acting on the geotextile has been shown to be important. The permittivity is less sensitive with respect to the normal stress than the transmissivity. Needle-punched geotextiles show greater variations than the spun-bonded geotextile. From the comparison of the coefficients of permeability normal to the plane and in the plane of the geotextile it may be said that needle-punching results in about the same coefficients, whereas spun-bonding appears to give a relatively higher permeability in the plane (factor 10) as shown in Fig. 7. This result will be checked by additional tests in a different permeameter which allows tests to be performed on a single geotextile sheet.

ACKNOWLEDGEMENTS

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Behavior of Geotextiles in the Case of Localized Stresses

Comportement des géotextiles aux sollicitations localisées

Localized breakings on geotextiles have to be taken into account. They can effectively constitute the start of tearings and in any case, they produce a discontinuity in the lap.
Two kinds of localized stresses can be differenciated those applied on the fibers and those bearing on the structure of the textile.
The first ones induce the breaking of geotextiles by tension, shearing and compression (up to melting). The simulation and laboratory tests show that needled and heat-bonded nonwovens have the same behaviour towards this kind of stress.
The second kind of stresses leads to the breaking by traction. The punching test showed that head-bonded nonwovens have a hingh sensitivity to the speed of the stress.

Les ruptures localisées sur les géotextiles doivent être considérées avec attention. Elles peuvent en effet constituer des amorces de déchirure et en tout cas, elles produisent une discontinuité dans la nappe.
Il faut distinguer deux types de contraintes localisées celles qui s'exercent au niveau des fibres et celles qui s'appliquent sur la structure des géotextiles.
Les premières entraînent la rupture des géotextiles, par traction, cisaillement et compression (jusqu'à la fusion). Les essais de simulation et de laboratoire montrent que les nontissés aiguilletés et thermoliés présentent la même sensibilité à ce type de sollicitations. Les deuxièmes entraînent la rupture par des contraintes de traction. L'essai de poinçonnement a montré la grande sensibilité des nontissés thermoliés à la vitesse de sollicitation.

INTRODUCTION

Il existe deux grands types de sollicitations localisées : celles qui s'exercent au niveau des fibres uniquement et celles qui s'exercent sur la structure du géotextile. Dans une première partie du texte, un essai de simulation est réalisé en appliquant une compression dynamique à un géotextile disposé sur un ballast. Deux essais de laboratoire sont comparés : un essai de cisaillement et un essai de traction à pinces jointives. Dans une deuxième partie, un essai de perforation sollicitant la structure du géotextile est effectué à deux vitesses très différentes.

1 CAS DES SOLLICITATIONS QUI S'EXERCENT AU NIVEAU DES FIBRES

1.1 Perforations observées sur chantiers

La figure 1 représente un géotextile extrait d'une piste de chantier renforcée.
Initialement le géotextile nontissé polyester obtenu par filage direct (spunbonded) était disposé entre deux couches de grave 0/100 mm de Seine (ROUEN) comportant une grande proportion de cailloux de silex. Leur épaisseur étant de 30 cm chacune.
Après le passage de 400 000 tonnes de matériaux transportés, l'épaisseur de la grave de recouvrement n'était que de 10 cm, et le géotextile présentait de nombreuses perforations provenant de l'action conjuguée de contraintes de traction, de cisaillement et de compression.

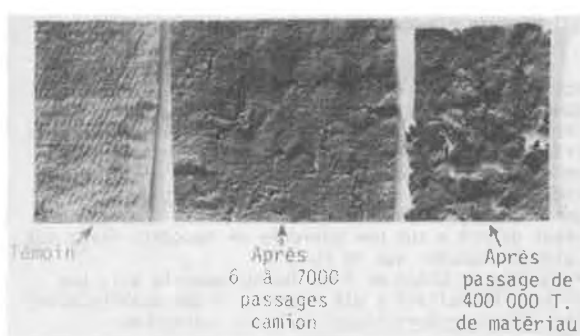

Fig. 1 - Géotextile nontissé aiguilleté de polyester provenant d'une piste de chantier.

1.2 Différents modes de rupture des fibres

La rupture par cisaillement donne une empreinte aux bords très nets comme celle qui est représentée figure 2 (coupure obtenue par un outil tranchant).
Un effort de compression lorsqu'il est suffisamment important peut se traduire par la fusion du géotextile.
On peut voir sur la figure 3 une rupture obtenue par un coup de marteau appliqué à un textile posé sur une surface dure.

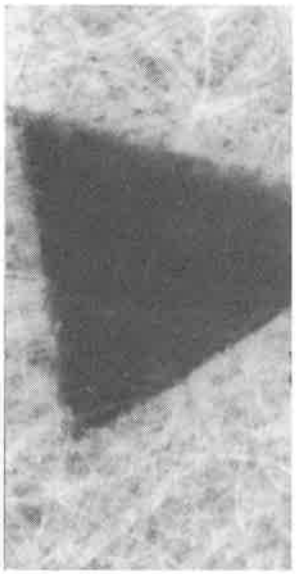

Fig. 2 - Perforation d'un géotextile par cisaillement

Fig. 3 - Rupture par compression jusqu'à la fusion

Fig. 4 - Mise en place du géotextile avant application de la charge

Fig. 5 - La compression dynamique est appliquée au au moyen de la Dynaplaque

Pour fondre 1 g de fibres de polyester, il faut environ 670 J, c'est-à-dire que pour fondre 1 cm2 d'un nontissé PES de 270 g/m2, il faut 18 J.
Toutes ces dégradations peuvent constituer des amorces de déchirure et elles produisent une discontinuité dans la nappe.

1.3 Essai de simulation

1.3.1 Méthode d'essai utilisée

Il a été réalisé par le Laboratoire Régional des Ponts et Chaussées de TOULOUSE au moyen de la Dynaplaque. Cet appareil permet d'appliquer sur le sol par l'intermédiaire d'une plaque de diamètre 60 cm une sollicitation dynamique analogue en intensité et en fréquence à celle provoquée par le passage d'un essieu chargé à 13 tonnes et roulant à 60 km/h et ceci au moyen d'une masse de 150 kg que l'on fait tomber d'une hauteur de 0,5 m sur une couronne de ressorts fixés sur la plaque reposant sur le sol.
Sur un bloc de béton de 8 m3 inclus dans le sol, une épaisseur de ballast a été répartie d'une manière aléatoire. Ses caractéristiques sont les suivantes :
25/50 mm en ophite γ = 2 900 kg/m3. Résistance au choc (LOS ANGELES) 13 à 18. Comme on peut le voir sur la figure 4, un géotextile est disposé sur ce ballast.
La Dynaplaque est ensuite disposée sur le géotextile et 10 - 50 et 100 coups sont appliqués (figure 5). Entre chacune de ces séries de compression, l'échantillon est déplacé de un quart de tour.
Les perforations observées semblent avoir été obtenues par fusion (figure 6).

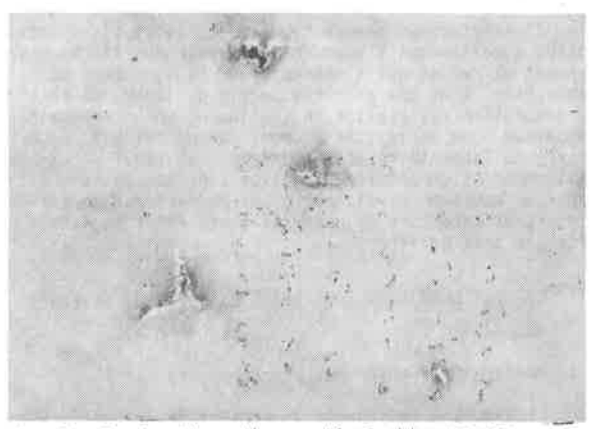

Fig. 6 - Perforations d'un nontissé obtenues par l'essai de simulation

1.3.2 Géotextiles testés

23 échantillons ont été testés. Dans la classe des nontissés obtenus par filage direct :
5 aiguilletés en polyester (BIDIM),
6 aiguilletés en polypropylène (SODOCA),
4 thermoliés 67 % polypropylène et 33 % polyéthylène (TERRAM),

3 thermoliés polypropylène (TYPAR).
Dans la classe des tissés : 5 tissés de bandelettes polypropylène (AMOCO).

1.3.3 Résultats

Après 10 - 50 et 100 coups, on a mesuré la "longueur de perforation" sur une table lumineuse.
La "surface de perforation" a été mesurée après 100 coups par transfert sur papier et pesée.
Les résultats qui sont dans le tableau I sont la moyenne de 2 essais (1 seul essai pour les tissés).

Tableau 1 - Longueurs et surfaces de perforation obtenues par l'essai de simulation

Type de géotextiles	Produit	Marque	Désignation	Masse surfacique (g/m2)	Volume de matière par unité de surf.	Longueur de perforation m: 10 Coups	50 Coups	100 Coups	Surface de perforation à 100 coups‰
Nontissés aiguilletés	PES	BIDIM	U14	183	1,10	133	345	487	6,15
			U24	263	1,50	118	324	460	5,78
			U34	307	1,95	104	354	503	6,44
			U44	389	2,46	100	222	229	2,60
			U64	646	3,98	43	43	74	0,52
	PP	POLYFELT	TS 300	250	2,89	140	300	441	5,12
		SODOCA	AS 200	211	2,20	161	295	441	5,53
			AS 250	292	2,60	145	309	309	4,25
			AS 300	292	3,00	133	248	284	3,47
			AS 400	462	3,70	106	149	264	2,80
			AS 420	409	4,10	74	137	245	2,58
Nontissés thermoliés	67% PP 33% PE	TERRAM	75g	79	0,85	125	343	525	11,85
			1000	154	1,67	121	309	569	7,82
			2000	260	2,82	90	316	371	4,76
			3000	310	3.36	87	152	234	2,76
	PP	TYPAR	136	157	1,74	72	168	477	6,38
			200	228	2,53	38	160	305	4,00
			270	302	3,35	40	121	214	2,65
Tissé de bandelettes		AMOCO	50/6050/72	103	1,14	69	153	207	2,20
			50/6060/72	139	1,34	70	196	201	2,64
			50/6062/72	196	2,15	18	56	93	1,11
			50/6064/73	340	3,77	47	75	95	1,07
			50/6066/73	549	6,10	0	0	40	-

1.3.4 Analyse des résultats

On a porté sur la figure 7 la longueur de perforation après 10 - 50 et 100 coups pour chaque échantillon.
Afin de tenter de relier les résultats entre eux, on a porté sur la figure 8 la longueur de perforation après 100 coups en fonction du volume de matière par m2 de géotextile. afin de s'affranchir du poids volumique des différents polymères (γ = 1 380 kg/m3 pour le PES et 900 pour le PP). On s'aperçoit que les points se trouvent à peu près linéairement répartis par grande famille de produits : les nontissés aiguilletés ou thermoliés et les tissés de bandelettes.
On a remarqué que les perforations obtenues sur les nontissés thermoliés occupaient une surface plus importante (à longueur de perforation égale) que celles occasionnées sur les aiguilletés.
Afin de tenir compte de cette observation, on a porté sur la figure 9 le pourcentage de surface perforée en fonction du volume de matière au m2.
Là encore il n'est pas possible de distinguer les aiguilletés des thermoliés.

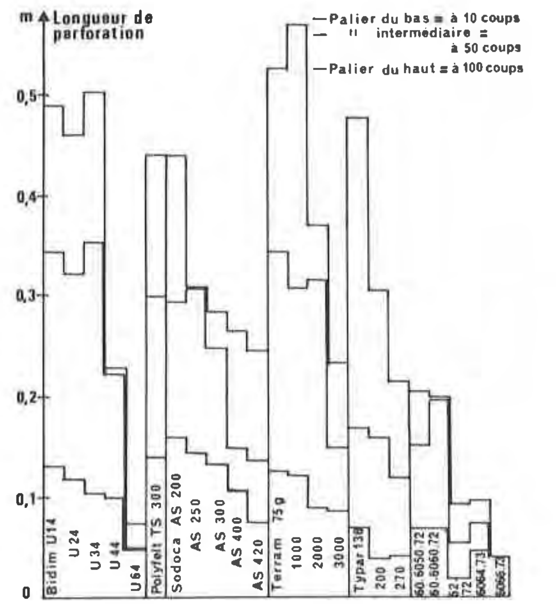

Fig. 7 - La longueur de perforation pour chaque produit apres 10 - 50 et 100 coups

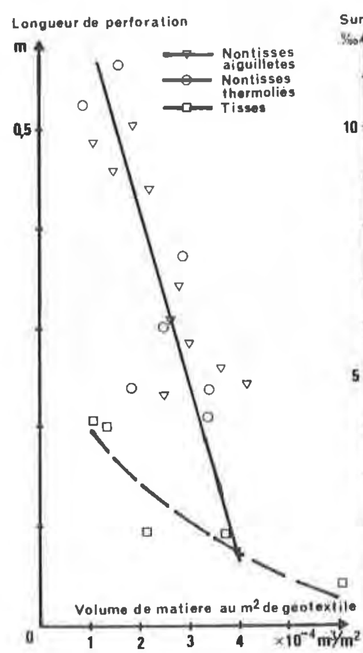

Fig. 8 - Longueur de perforation en fonction du volume de matière par m2

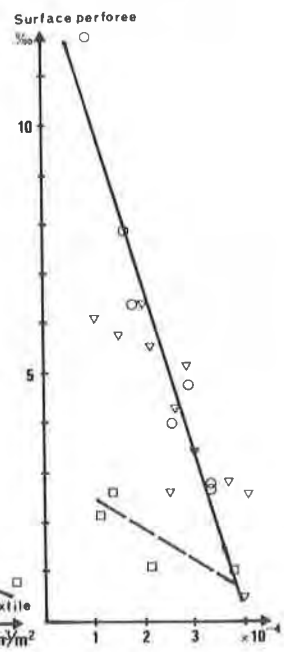

Fig. 9 - Fraction de surface perforée en fonction du volume de matière par m2

Les tissés de bandelettes semblent offrir une meilleure résistance à ce type de perforation. Il faut faire à ce stade, deux remarques importantes :
- on n'a testé qu'un seul type de tissé,
- ce type de matériau étant moins déformable, la surface de perforation est plus difficile à estimer car les bandelettes sectionnées ont tendance à se remettre en place sans laisser apparaître de trous.

En conclusion, on peut dire que cet essai de perforation sur surface dure au moyen d'une sollicitation dynamique permet après un nombre de coups suffisants (100) de caracteriser les géotextiles vis-à-vis de leur sensibilité à ce type d'agression. En fonction du type d'ouvrage, il est possible à l'ingénieur de prendre en compte ce type de risque à partir du moment où il aura défini le pourcentage de perforation acceptable. Enfin si l'on voulait proposer des classes, on pourrait les choisir de la manière suivante :

(I) surface de perforation (Sp) < 3 ‰ (nontissés épais et tissés de bandelettes),
(II) 3 < Sp < 5 ‰
(III) 5 < Sp < 8 ‰
(IV) Sp > 8 ‰

1.4 Essais de laboratoire

1.4.1 Essai de cisaillement transversal

A l'aide d'un dispositif réalisé à l'INSTITUT TEXTILE DE FRANCE (LYON) schématisé sur la figure 10 on applique un double cisaillement sur un échantillon de géotextile de 10 cm de largeur immobilisé par serrage. Deux types de géotextiles ont été testés :
- un nontissé thermolié de filage direct (TYPAR),
- un nontissé aiguilleté de filage direct (BIDIM).

Les résultats obtenus sont portés dans le tableau 2. Pour chaque géotextile, 5 essais dans chaque sens ont été réalisés.

Tableau 2 - Charge à la rupture en cisaillement (en kN/m)

Marque	BIDIM				TYPAR		
Type	Nontissé aiguilleté				N T thermolié		
Designation	U 14	U 34	U 44	U 64	3357	3607	3807
Sens Production	11,8	24,5	28,3	46,3	11,8	18,0	25,9
Sens Travers	15,1	26,1	29,8	54,8	11,3	17,6	26,7

1.4.2 Essai de traction pinces jointives

Sur les mêmes géotextiles, des essais de tractions pinces quasi-jointives ont été effectués à l'INSTITUT TEXTILE DE FRANCE (PARIS).

Les conditions expérimentales étaient les suivantes :
- disposition suivant figure 11,
- 2 essais dans chaque sens,
- espace initial entre les pinces : < 2 mm,
- vitesse d'essai 500 mm/mn (des essais effectués sur un BIDIM U44 ont montré qu'une vitesse de traction choisie entre 5 et 500 mm/mn n'avait pas d'influence sur la force de rupture).

Les résultats obtenus sont inscrits dans le tableau 3.

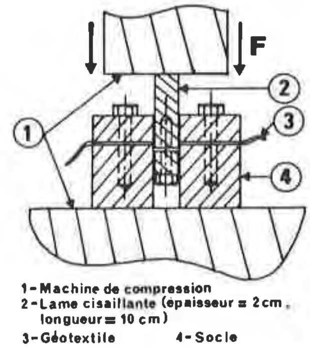

Fig. 10 - Dispositif de cisaillement transversal

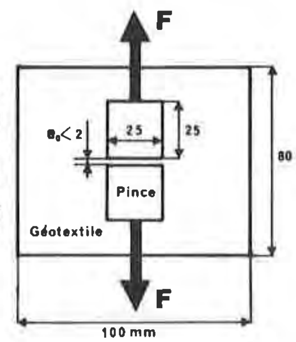

Fig. 11 - Essai de traction à pinces quasi-jointives

Tableau 3 - Charge à la rupture en traction pinces quasi-jointives (en kN/m)

Marque	BIDIM					TYPAR		
Designation	U 14	U 24	U 34	U 44	U 64	3407	3607	3807
Sens Production	18,8	24,8	27,4	34,4	68,2	15,4	24,8	26,6
Sens Travers	16,0	24,6	26,8	25,2	53,4	12,2	24,8	33,0

1.4.3 Comparaison des deux types d'essais

Pour le sens production comme pour le sens travers, si l'on porte la force à la rupture en fonction du poids surfacique des deux types de géotextiles (figures 12 et 13), on peut faire les constatations suivantes :
- les forces se répartissent à peu près linéairement,
- on ne distingue pas l'aiguilleté du thermolié,
- les valeurs obtenues par cisaillement et par traction sont très voisines.

Ce dernier point est valable pour ces matériaux constitués de polyester ou de polypropylène et dont les propriétés mécaniques transversales sont bonnes.

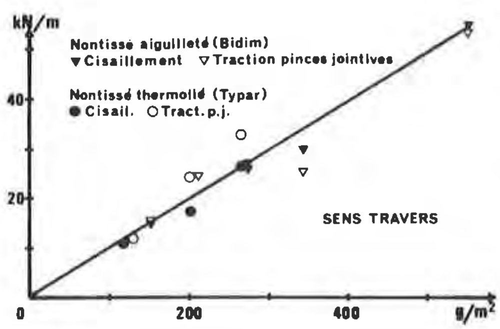

Fig. 12 - Force a la rupture en traction pinces jointives et en cisaillement en fonction du poids surfacique des nontissés - sens Travers -

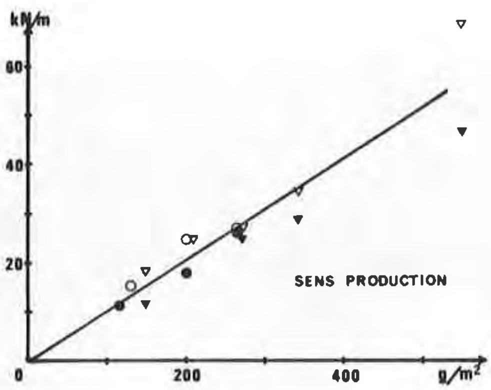

Fig. 13 - Force à la rupture en fonction du poids surfacique - sens Production -

Si l'on prend un tissu de verre ou de Kevlar, les résultats obtenus en cisaillement et en traction (pinces quasi-jointives) ne sont absolument pas comparables (tableau 4).

Tableau 4 - Charge à la rupture en cisaillement et en traction pinces jointives pour des produits à faibles propriétés transversales (en (en kN/m)

TISSUS	VERRE	KEVLAR
Cisaillement	1,9	34
Traction pinces jointives	13,7	85

En conclusion, les essais précédemment décrits ne permettent pas de mettre en évidence le mode de liage de deux types de nontissés car ils ne sollicitent que les fibres et non la structure. L'essai de cisaillement permet d'évaluer efficacement les produits constitués par des fibres à faible propriétés transversales.

2 CAS DES SOLLICITATIONS QUI S'EXERCENT AU NIVEAU DE LA STRUCTURE DU TEXTILE

2.1 Cas réel

C'est la perforation d'un géotextile sur support déformable par les dents du godet d'un engin ou par la chute de pierres lors d'un déversement. Dans chacun de ces cas, la vitesse de sollicitation est différente.

2.2 Essai de laboratoire

2.2.1 Méthode d'essai

L'essai mis au point par I T F - LYON consiste à perforer un géotextile de 18 cm de diamètre immobilisé par serrage au moyen d'un poinçon dont l'extrêmité est un ellipsoïde de révolution (diamètre = 4 cm ; demi-grand axe = 4 cm).

Un essai dynamique est réalisé à l'aide de l'appareillage schématisé sur la figure 14. Il consiste à laisser tomber le poinçon de masse M d'une hauteur h. L'énergie de rupture est donnée par le produit M x g x h. L'essai à vitesse lente est obtenu en montant le poinçon sur une machine de compression ; l'énergie de rupture est donnée dans ce cas par intégration du graphe F (Δe).

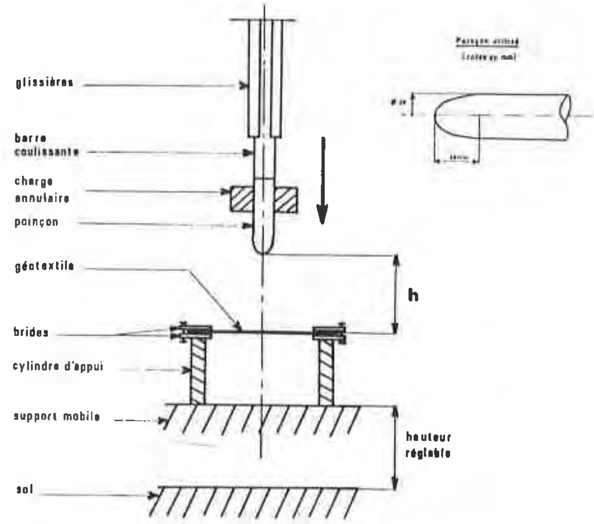

Fig. 14 - Dispositif de perforation dynamique.

Tableau 5 - Energie de rupture par perforation en statique et en dynamique

Type de géotextile	Produit	Marque	Désignation	Poids surfacique g/m2	Energie de perforation (J) Statique	Dynamique
Nontissés aiguilletés	PES	BIDIM	U 14	150	24,2	24,5
			U 24	210	35,6	36,4
			U 34	270	45	44,1
			U 64	550	74,7	85
	PP	SODOCA	AS 150	130	21	22
			AS 200	200	32,9	21,8
			AS 300	270	44,1	44,8
			AS 600	600	94	50,7
Nontissés thermoliés		TYPAR	3357	116	13,2	7,6
			3607	203	25,4	16
			3807	265	40	18,8
	PES	LUTRADUR	LDH 220	200	22,6	16,8
Tissé		TEXUNION	5747-71	250	23,5	31
			5749-71	245	19,1	28,7

2.2.2 Résultats

On a porté dans le tableau 5 les énergies de rupture en quasi-statique et en dynamique pour les géotextiles testés.
Si l'on porte ces énergies en fonction du poids surfacique des nontissés, on remarque que :
- à faible vitesse, les points représentatifs des aiguilletés et des thermoliés s'alignent à peu près sur une même droite (figure 15),
- en dynamique, les énergies de rupture pour les thermoliés sont plus faibles que celles que l'on obtient avec les aiguilletés (figure 16). Cela s'explique par la faible mobilité des fibres des thermoliés qui s'arrangent en partie à faible vitesse, mais qui sont sectionnées beaucoup plus nettement à grande vitesse (figure 17).

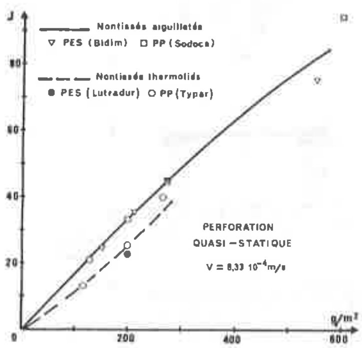

Fig. 15 - Energie de rupture en fonction du poids surfacique des nontissés - Vitesse lente -

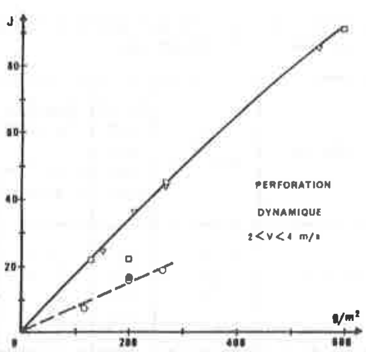

Fig. 16 - Energie de rupture en fonction du poids surfacique - Essai dynamique -

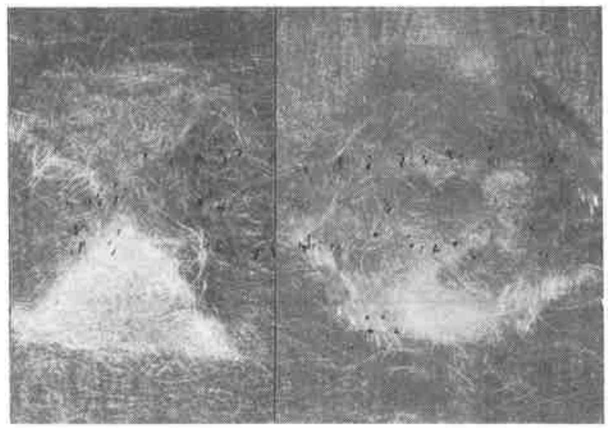

Fig. 17 - Faciès de rupture par poinçonnement d'un nontisse thermolié.
A droite, à vitesse lente,
A gauche, en dynamique.

CONCLUSION

Nous avons distingué deux types de sollicitations localisées. Celles qui s'exercent sur les fibres et celles qui intéressent la structure du géotextile.
Dans une première partie, un essai de simulation a permis de montrer que les tissés de bandelettes ainsi que les nontissés épais offraient une bonne résistance à des efforts de compression dynamique qui se traduisent par une fusion locale des fibres.
Deux essais de laboratoire ont été comparés : l'essai de cisaillement et l'essai de traction à pinces jointives. Les nontissés aiguilletés et thermoliés de polyester et polypropylène présentent les mêmes forces de rupture en cisaillement et en traction. Par contre, l'essai de cisaillement est à même de mettre en évidence les produits à faible propriété mécanique transversale.
Dans une deuxième partie, un essai de perforation quasi statique et dynamique a montré la sensibilité des nontissés thermoliés à la vitesse de sollicitation par rapport aux aiguilletés dont la mobilité des fibres est beaucoup plus grande.
Quel que soit le type de ruptures obtenues, leur présence sur un géotextile en place constitue des amorces de déchirure et produit une discontinuité dans la nappe.

Les auteurs remercient :

- M. BAUDONNEL J. (ITF-PARIS) pour la conduite des essais de traction à pinces jointives,
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Force Transfer at the Contact of Geotextiles and Soil Under Permanent and Cyclic Loads

Transfert des forces sur le contact des géotextiles et des sols auprès des charges constant et cyclique

Tests of sand - geotextile friction were made with the shear force either continuously variable, or cyclically transient. The paper analyzes the principal factors influencing the results of tests in a shear box apparatus, explains the differences in character of dilatancy changes in the shear of sand alone and the friction of sand with geotextile. The angle of friction between the soil and the fabric with static function is approximately the same as the residual angle of internal friction of the soil. The tests with cyclic changes of shear stresses were made with sand friction with duraluminium, wood, concrete and geotextile. The energy supplied to the soil during one cycle is markedly higher in the case of the geotextile base than in other cases. Since it has been already previously proved that the fatigue phenomena of the soil depend on the energy supplied to the soil, a high sensitivity of soil structures with geotextiles.

On a procédé aux expériences portant sur le frottement du sol pulvérulent et géotextiles. La force de cisaillement a été, d´une part, progressivement variable, et, de l´autre, cycliquement. L´étude analyse les facteurs principaux influençant les résultats des essais dans un appareil de cisaillement direct. On élucide les différences dans le caractère des changements de dilatation lors du glissement du sable meme et lors du glissement du sable sur les géotextiles. Les essais dans les cas cyclique ont été réalisées avec le sable sur la surface de duralumin, de bois, de béton et de géotextiles. L´énergie, inculquée au so durant un cycle est relativement plus grande dans les cas des géotextiles. Etant donnée le fait que déjà par le passé il a été prouvé que les manifestation de la fatigue du sol dépendent de l´énergie inculquée au sol, on suppose une sensibilité élevée des constructions du génie civil et géotextiles.

Laboratory tests of friction of Zbraslav sand were carried out with the purpose of specifying with greater accuracy the informative values of coefficients of friction of soils and type PAD TT4-181/77 geotextiles. The mechanical properties of the fabric were ascertained in detail as a separate problem /1/, /6/, /3/. The results of uniaxial tensile tests are shown in Fig.1 and Fig.2. The test of shear transfer from the soil to the fabric were carried out in shear box apparatus /Fig.3/. The tests were carried out under constant temperature for 15 minutes so that it can be assumed that the strain changes with temperature are negligible /2/, /3/. Since during the tests of the shear of soil along the fabric the latter slid along a duraluminium base, tests of friction of the fabric along a duraluminium plate with various surface treatment were carried out /Fig.4/. The angle of friction at rest δ_f was greater than the angle of friction in motion δ_r; however, the differences in the case of a dry,clean duraluminium plate were very small. In further tests, therefore, the angle of friction was considered as $\delta \doteq \delta_f \doteq \delta_r \doteq 13^{o}$. A comparison of standard shear tests with the tests of the shear of a soil along the fabric is shown in Fig.4. Up to the stress level of $\bar{\tau} = \sigma_n \operatorname{tg} \delta$ the effect of the fabric on the course of the test was non-existing. After the shear stress $\bar{\tau}$ has been attained, the fabric began sliding along the base and elongate. In the soil a failure zone at about 0,6 cm above the fabric arose /the overall test sample height above the fabric being 1,2 cm/. The described phenomenon was monitored by photographing the movement of the grains through the transparent side of the shear box by a camera connected with it. The sliding of the soil along the fabric is not identical along the whole box length. By the soil friction the fabric elongated and its movement was braked by its friction along the duraluminium base. The analysis of these displacements for one point of the working diagram of sand/fabric friction is shown in Fig.6. The differences of the sand displacement along the fabric in the individual points of the shear box length are considerable, even under the assumption that the vertical normal stresses are regularly distributed. Actually the local normal stresses σ_n vary, due to irregular porosity, so that the dashed curve in Fig.6. must be considered approximate only. The effect of the fabric influences substantially the effect of structure of adjacent soil under shear stress. The peak shear strength of soil is overcome only locally. In the majority of the shear surface there is $\tau < \tau_f$, if the peak shear strength τ_f has just been mobilized in some place. Let us introduce the notation δ_f for the peak and δ_r for the residual value of the angle of soil/fabric friction; in loose soils δ_f nears δ_r the more, the higher the porosity of the soil. /Analogy is known from the angle of internal friction of soil $\varnothing_f$ and $\varnothing_r$ /4/ /. We obtain $\delta_f \doteq 0.5\ /\varnothing_f + \varnothing_r/$

and $\delta_r \doteq \phi_r$ for $\bar{\delta} = 0.4\delta$ and $\bar{l} = 2\ d_{50}$, when $1/\bar{l} = 7$ threads/cm is the density of the fabric and d_{50} is the diameter of the grains /in cm/ representing 50% of the weight of the sample on the granulometric curve.

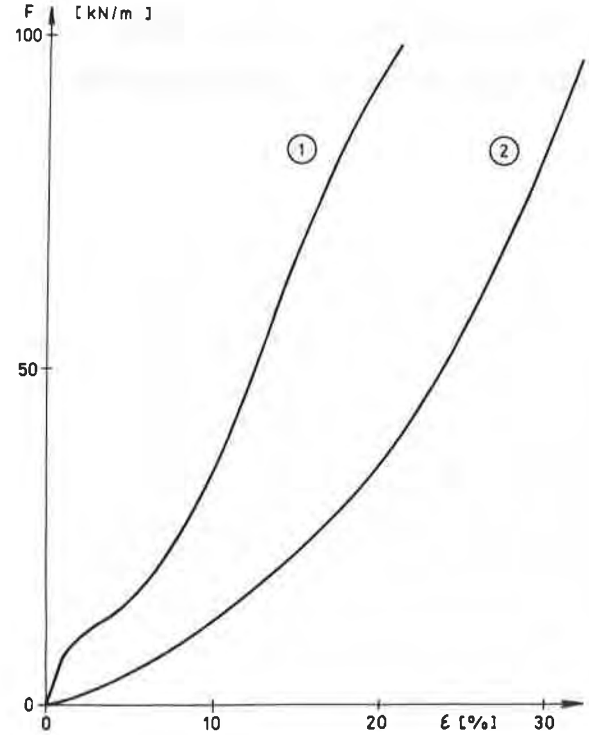

Fig.1 Working diagram of type PAD 12823 geotextile fabric in uniaxial tension. /After Minster 1974/
1 - tension in warp direction
2 - tension in weft direction.

The tests of the fabrics on a carborundum plate, with fine and coarse sand have ascertained that when $\bar{\delta} > \delta$ and $l \geqslant 2\ d_{50}$, the shear surface originates in the sand adjacent to the relatively coarse fabric. Then it is necessary to count with $\delta_f \rightarrow \phi_f$ and $\delta_r \rightarrow \phi_r$. If $\bar{l} \doteq d_{50}$, any movements of the fabric cause approximately identical movements of adjacent grains. If simultaneously $\bar{\delta} < \delta$, the elongation of the fabric causes major changes of porosity of adjacent soil; the porosity gest stabilized, when the critical value has been attained. Since the grains are well wedged in the fabric with approximately identical depth, the shear in the soil adjacent to the fabric occurs under approximately critical porosity in the whole failure zone.

It was also tested how the change /reduction/ of the angle $\bar{\delta}$ of friction of the fabric along its base will influence the measured value of the angle δ of friction of the fabric and the sand. The residual angle δ_r did not depend on $\bar{\delta}$, but it did influence the course of the test. Due to the reduction of $\bar{\delta}$ the displacements of the fabric increased and the places, where probably the peak value of friction was mobilized, were even more locally limited. The low value of $\bar{\delta}$ resulted in the test result recalling the tress-strain diagram of loose sand. In either case the angle of soil/fabric friction got stabilized at $\delta_r \doteq \phi_r$.

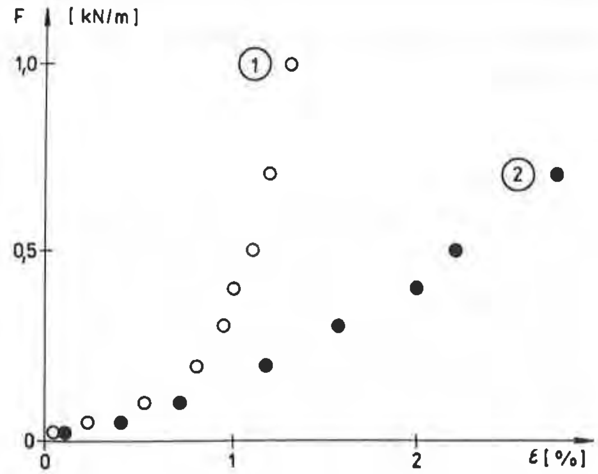

Fig.2 Detail of Fig.1 for small tensile forces F and small relative strain /After Minster 1974/.

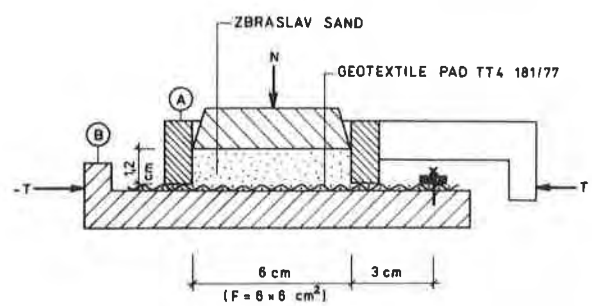

Fig.3 Diagram of friction tests of sand with fabric
A - shear box mobile in the direction of force T under vertical load N,
B - base

In the tests dry, clean fabrics were considered. In actual practice, however, the openings of the fabric may become clogged, after a short displacement, with a material with a low angle of internal friction. In further friction the expansive behaviour of adjacent soil will not manifest itself in its full extent; in an extreme case the friction of soil with the moist, clogged fabric must be characterized by the angle of $\delta \ll \phi_r$. Similar case may occur also, when dense or unwoven, precompressed fabrics are used.

It should be noted that the described tests do not characterize the spatial behaviour of fabrics in the conditions of actual construction. The fabrics are, as a rule, orthogonally anisotropic. In time their degradation and degeneration takes place, while the soils under variable load may become subject to fatigue /5/, /6/, but also to improvement by consolidation.

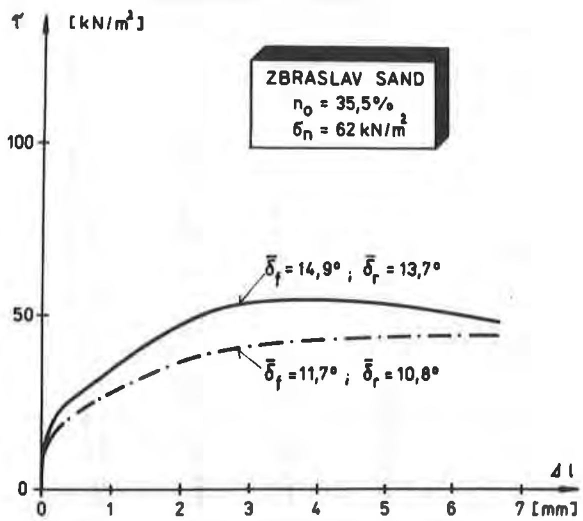

Fig.4 Shear tests of friction of sand with fabric under a vertical stress σ_n = 62 kN/m^2 for different peak / $\bar{\delta}_f$/ and residual / $\bar{\delta}_r$/ friction of fabric with its base.

So far we have considered only the case of a static shear test. The simple equipment available to authors made it possible to carry out a comparative study of the friction of soil with various materials under repeated transient shear loading. The test of this type differed from the standard test in that the attained shear stress was reduced to zero as soon as the displacement of the shear box attained one whole mm, and slowly increased again immediately afterwards. A typical course of the test is shown in Fig.7. During the tests no signs of soil fatigue were observed /5/, /6/, probably due to the small number of repeated cycles. The peak and residual stresses / τ_f, τ_r/ in a repeated shear loading of a sand sample correspond approximately with analogous values of the angles of friction δ_f, δ_r with different materials ascertained by standard test.

However, the transient repeated shear load tests have opened a new vista of structural soil changes occuring during soil friction with other materials. The diagram showing the friction of Zbraslav sand with concrete in Fig.7 comprizes a loop "C", shown in a thick line, which has the property of being the first loop after the value of τ_f has been attained. This loop is shown again in Fig.8 together with other loops obtained analogously during the friction tests with other materials/A - duraluminium, B - wood, C - concrete, D - type PAD TT4-181/77 fabric/ for σ_n = 212 kN/m^2 and σ_n = 62 kN/m^2. Apart from the initial sand porosity n_o every loop

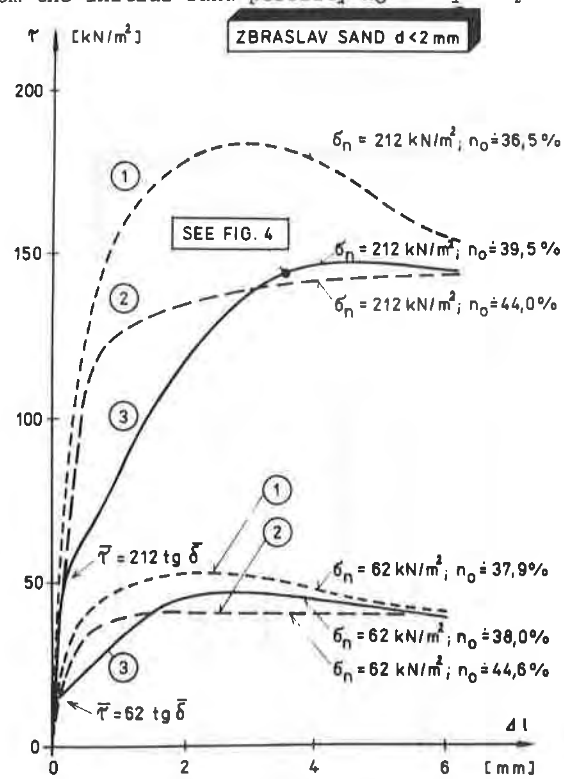

Fig.5 Comparison of standard shear tests of Zbraslav sand /dashed line/ with the friction tests of the same sand with geotextile /solid lines/.

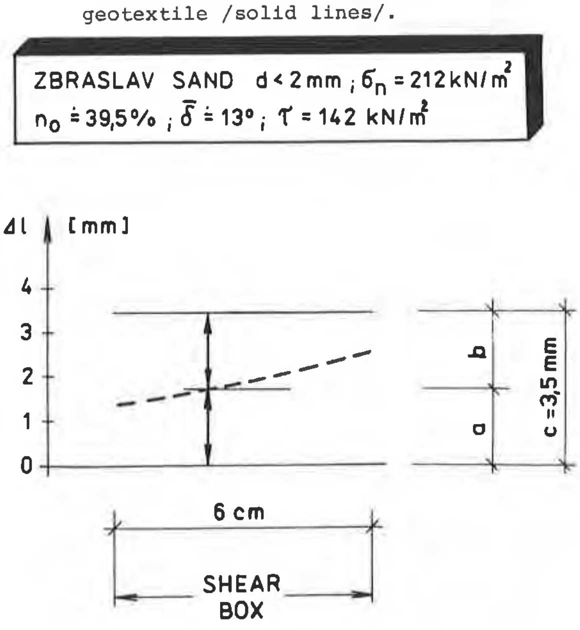

Fig.6 Analysis of displacements of a selected point /according to Fig.5/ during the

friction test of sand with fabric.
a - displacement of the shear box along the fabric,
b - displacement of the fabric along the base,
c - displacement of the shear box with regard to the base.

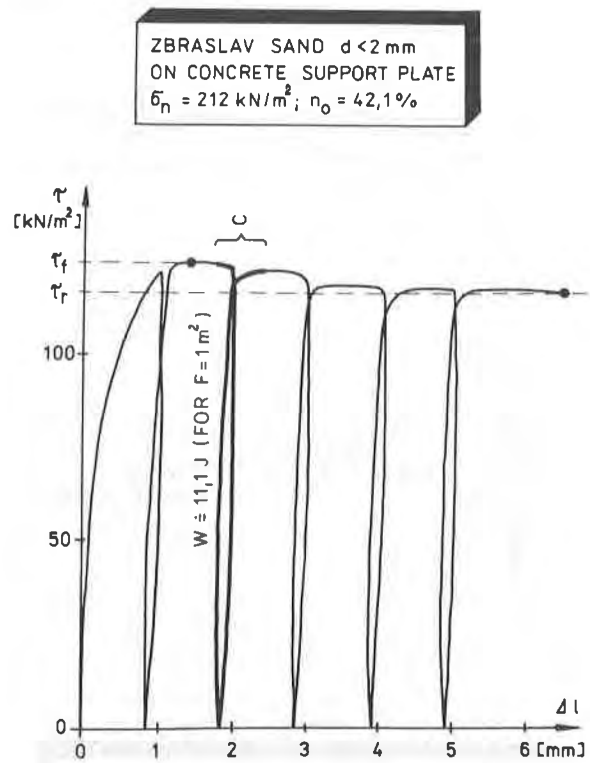

Fig.7 Working diagram of the friction test of soil with concrete under repeated shear loading. The first loop after the peak value of τ_f has been attained is marked in thick lines. W - energy corresponding with the area of the loop referred to 1 m^2 of the shear surface

shows also the energy W supplied to the soil during the cycle represented by the loop. /The energy in J is referred to 1 m^2 of the shear surface/. In /5/, /6/ it was shown that the structural changes in the soil due to the reduction and re-increase of τ were higher with higher W and vice versa. It was also shown that the higher the energy supplied to the soil during one cycle, the sooner the soil might fail. On the basis of these studies it is possible, for example, to assume a higher fatigue resistance to friction on the surface of metal piles than on the surface of timber or concrete. Let us note that the loops "D" are only slightly higher than the loops "C" in the tests of otherwise identical conditions, /for "D" there were $\bar{T} \doteq 2\ d_{50}$ and $\bar{\delta} = 0.4\ \delta$ /, but the energy supplied to the soil-fabric system was about four times as high as the energy supplied to the failure zone in the case of loop "C". We do not know what quota of energy corresponding with the loop "D" will be applied to the changes of soil structure and what quota will be applied to the degeneration of the fabric. However, the energy richness of the loop "D" is so marked that a high sensitivity of soil structures with built-in fabrics of tested type to repeated loading may be expected.

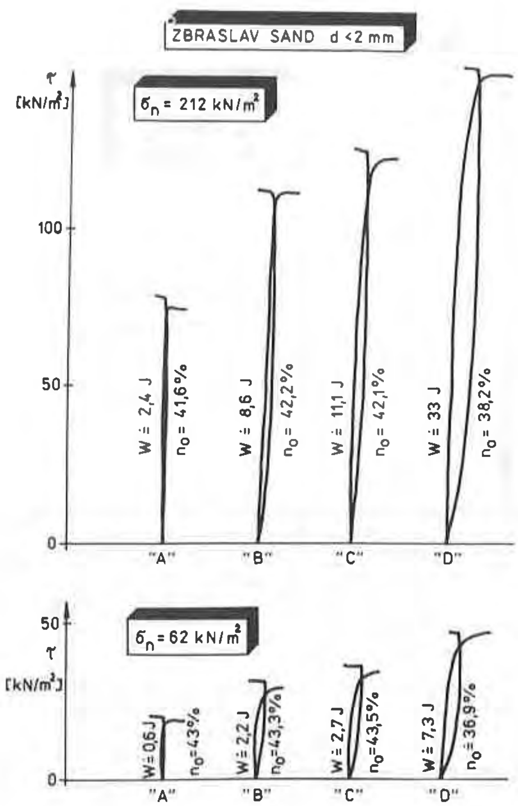

Fig.8 Comparison of first repetitions of shear stresses τ after the value of τ_f has been attained, analogously with the loop "C" in Fig.7. Every loop comorizes the corresponding energy W referred to 1 m^2 of the failure surface and the average porosity n_0 at the beginning of the test under vertical loads of 62 kN/m^2 and 212 kN/m^2 respectively. The loops correspond with the following base materials: A - duraluminium, B - wood, C - concrete, D - geotextiles /type PAD TT4-181/77 fabric/.

The reinforcing elements of geotextiles /fabrics/ should be so situated in the soil as to be loaded for a short period and dynamically as little as possible, until the soil has attained the necessary strength by consolidation. As a rule, the speed of consolidation will be very favourably influenced by the fabric.

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/6/ Kamenov, B., Kysela, Z., "Selected problems of soil dynamics" /in Czech/. Research Report, UTAM ČSAV, /Prague, 1979/.

Note: UTAM ČSAV - Institute of Theoretical and Applied Mechanics, Czechoslovak Academy of Sciences, Prague.

APPENDIX

SECOND INTERNATIONAL CONFERENCE ON GEOTEXTILES
DEUXIÈME CONGRÈS INTERNATIONAL DES GÉOTEXTILES

LIST OF SYMBOLS
LISTE DE SYMBOLES (*)

1. GENERAL SYMBOLS
SYMBOLES GENERAUX

1.1 Dimension symbols
Symboles de dimension

Symbols used for the dimensions are:		Les symboles de dimension sont:
L :	length	longueur
M :	mass	masse
T :	time	temps
- :	dimensionless	sans dimension

1.2 Unit Symbols
Symboles d'unités

m	meter	mètre
m^2	square meter	mètre carré
m^3	cubic meter	mètre cube
mm	millimeter	millimètre
μm	micron	micron
g	gram	gramme
mg	milligram	milligramme
kg	kilogram	kilogramme
s	second	seconde
N	newton	newton
kN	kilonewton	kilonewton
Pa	pascal	pascal
kPa	kilopascal	kilopascal
MPa	megapascal	mégapascal
J	joule	joule
tex	tex	tex
L	liter	litre
^{o}	degree	degré
%	percent	pourcent
-	pure number	nombre pur

(*)This list of symbols has been prepared by J. P. Giroud.
Cette liste de symboles a été établie par J. P. Giroud.

The following relationships exist: / Relations entre les unités ci-dessus:

$$1\ \mathrm{Pa} = 1\ \mathrm{N/m^2}$$

$$1\ \mathrm{tex} = 1\ \mathrm{mg/m}$$

$$1\ \mathrm{J} = 1\ \mathrm{mN}$$

1.3 Mathematical symbols
Symboles mathématiques

ln x	natural logarithm of x	logarithme naturel de x
lg x	logarithm of x base 10	logarithme décimal de x

1.4 Subscripts
Indices

Subindex / Indice	Applies to / Se rapporte à	
a	air or active (earth pressure)	air ou actif (poussée)
d	dry state	état sec
f	failure or final	rupture ou final
g	geotextile	géotextile
H	horizontal	horizontal
i	immediate or initial	immédiat ou initial
p	passive (earth pressure)	passif (butée)
r	radial	radial
s	solid particles	particules solides
sat	saturated	saturé
sec	secant	sécant
u	undrained conditions	conditions non drainées
V	vertical	vertical
w	water	eau
x, y	two orthogonal horizontal axes	deux axes orthogonaux horizontaux
z	vertical axis	axe vertical
o	at rest or initial conditions	au repos ou conditions initiales
1,2,3	principal directions	directions principales

1.5 Geometry and kinematics
Géométrie et cinématique

L	L	(m)	length	longueur
B	L	(m)	breadth	largeur
H	L	(m)	height, thickness	hauteur, épaisseur
D	L	(m)	depth	profondeur
z	L	(m)	vertical coordinate	abscisse verticale
d	L	(m)	diameter	diamètre

A	L^2	(m^2)	area	aire
V	L^3	(m^3)	volume	volume
t	T	(s)	time	temps
v	$L\,T^{-1}$	(m/s)	velocity	vitesse
g	$L\,T^{-2}$	(m/s^2)	acceleration due to gravity ($g = 9.81\ m/s^2$)	accélération de la pesanteur ($g = 9.81\ m/s^2$)

1.6 Stress and strain
Contraintes et déformations

u	$ML^{-1}T^{-2}$	(kPa)	pore pressure	pression interstitielle
			pressure of the fluid in the voids of a porous medium (soil, geotextile)	*pression du fluide dans les vides d'un milieu poreux (sol, géotextile)*
σ	$ML^{-1}T^{-2}$	(kPa)	normal stress	contrainte normale
σ'	$ML^{-1}T^{-2}$	(kPa)	effective normal stress	contrainte normale effective
			normal stress transmitted by intergranular contacts ($\sigma' = \sigma - u$ for saturated soils)	*contrainte normale transmise par contacts intergranulaires ($\sigma = \sigma - u$ pour les sols saturés)*
τ	$ML^{-1}T^{-2}$	(kPa)	shear stress	contrainte de cisaillement
$\sigma_1,\sigma_2,\sigma_3$	$ML^{-1}T^{-2}$	(kPa)	principal stresses	contraintes principales
ε	-	(-, %)	strain, élongation	déformation, élongation
			change in length per unit length in a given direction (called elongation if expressed in % and applied to a geotextile)	*variation relative de longueur dans une direction donnée (appelée élongation lorsqu'elle est exprimée en % et appliquée à un géotextile)*
$\varepsilon_1,\varepsilon_2,\varepsilon_3$	-	(-,%)	principal strains	déformations principales

1.7 Hydraulic parameters
Paramètres hydrauliques

h	L	(m)	hydraulic head or potential	charge hydraulique ou potentiel hydraulique
			sum of pressure height (u / γ_w) and geometrical height (z) above a given reference level	*somme de la hauteur piézométrique (u / γ_w) et de la hauteur géométrique (z) au-dessus d'un niveau de référence*
q	L^3T^{-1}	(m^3/s)	rate of discharge	débit
			volume of water seeping through a given area per unit of time	*volume d'eau percolant à travers une section donnée d'un sol, par unité de temps*
v	$L\,T^{-1}$	(m/s)	discharge velocity	vitesse d'écoulement
			rate of discharge per total unit area perpendicular to direction of flow	*débit qui s'écoule à travers une section totale unitaire du milieu, perpendiculaire à la direction de l'écoulement*

i	-	(-)	hydraulic gradient	gradient hydraulique
			loss of hydraulic head per unit length in direction of flow	*perte de charge hydraulique par unité de longueur dans la direction de l'écoulement*
j	$ML^{-2}T^{-2}$	(kN/m^3)	seepage force per unit volume	force de filtration (ou d'écoulement) par unité de volume
			force per unit volume of a porous medium generated by action of the seeping fluid upon the solid elements of the porous medium ($j = i\gamma_w$)	*force volumique exercée sur les éléments solides d'un milieu poreux par un fluide en écoulement: ($j = i.\gamma_w$)*

2. PROPERTIES OF FLUIDS

PROPRIETES DES FLUIDES

ρ_w	ML^{-3}	(kg/m^3)	density of water	masse volumique de l'eau
γ_w	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of water	poids volumique de l'eau
η_w	$ML^{-1}T^{-1}$	(kg/ms)	dynamic viscosity of water	viscosité dynamique de l'eau

NOTE: Instead of w , use any other appropriate subscript of other fluids (e.g.: η_a, dynamic viscosity of air)

A la place de w, on peut utiliser d'autres indices relatifs à d'autres fluides que l'eau (par exemple: η_a, viscosité dynamique de l'air)

3. PROPERTIES OF SOILS
PROPRIETES DES SOLS

3.1 Solid particles and their distribution
Particules solides et leur distribution

ρ_s	ML^{-3}	(kg/m^3)	density of solid particles	masse volumique des particules solides
			ratio between mass and volume of solid particles	*quotient de la masse des particules solides par leur volume*
γ_s	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of solid particles	poids volumique des particules solides
			weight of solid particles per unit volume	*quotient du poids des particules solides par leur volume*
d	L	(mm)	particule diameter	diamètre de particule
			particle size as determined by sieve analysis or wet mechanical analysis	*taille de particule déterminée dans l'analyse granulométrique par tamisage ou sédimentométrie*
d_n	L	(mm)	n percent-diameter	diamètre à n pourcent
			diameter corresponding to n percent by weight of finer particles	*diamètre correspondant a un tamisat de n pourcent sur la courbe granulométrique (n% des particules ont des dimensions inférieures à ce diamètre)*

C_U	-	(-)	uniformity coefficient defined as : d_{60}/d_{10}	coefficient d'uniformité defini par : d_{60}/d_{10}

3.2 Physical properties of soils
Propriétés physiques des sols

ρ	ML^{-3}	(kg/m^3)	density of soil *ratio between total mass and total volume of soil*	masse volumique du sol *quotient de la masse totale du sol par son volume*
γ	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of soil *ratio between total weight and total volume of soil*	poids volumique du sol *quotient du poids total du sol par son volume*
ρ_d	ML^{-3}	(kg/m^3)	density of dry soil *ratio between mass of solid particles and total volume of soil*	masse volumique du sol sec *quotient de la masse des particules solides par le volume total du sol*
γ_d	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of dry soil *ratio between weight of solid particles and volume of soil*	poids volumique du sol sec *quotient du poids des particules solides par le volume total de sol*
ρ_{sat}	ML^{-3}	(kg/m^3)	density of saturated soil *ratio between total mass and total volume of completely saturated soil*	masse volumique du sol saturé *quotient de la masse totale du sol complètement saturé par son volume total*
γ_{sat}	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of saturated soil *ratio between total weight and total volume of completely saturated soil*	poids volumique du sol saturé *quotient du poids total du sol complètement saturé par son volume total*
ρ'	ML^{-3}	(kg/m^3)	density of submerged soil *difference between density of soil and density of water*	masse volumique du sol déjaugé *différence entre la masse volumique du sol et la masse volumique de l'eau*
γ'	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of submerged soil *difference between unit weight of soil and unit weight of water*	poids volumique du sol déjaugé *différence entre le poids volumique du sol et le poids volumique de l'eau*
e	-	(-)	void ratio *ratio between volume of voids and volume of solid particles*	indice des vides *rapport entre le volume des vides et le volume des particules solides*
n	-	(-, %)	porosity *ratio between volume of voids and total volume of soil*	porosité *rapport entre le volume des vides et le volume total du sol*

w	-	(-.%)	water content *ratio between weight of pore water and weight of solid particles (expressed in percentage)*	teneur en eau *rapport entre le poids de l'eau interstitielle et le poids des grains solides (exprimé en pourcents)*
S_r	-	(-. %)	degree of saturation *ratio between volume of pore water and volume of voids*	degré de saturation *rapport entre le volume de l'eau interstitielle et le volume des vides*

3.3 Permeability of soils
Perméabilité des sols

k	LT^{-1}	(m/s)	coefficient of permeability (or hydraulic conductivity) *ratio between discharge velocity and corresponding hydraulic gradient (v / i)*	coefficient de perméabilité (ou conductivité hydraulique) *quotient de la vitesse d'écoulement par le gradient hydraulique correspondant (v / i)*

3.4 Mechanical properties of soils
Propriétés mécaniques des sols

E	$ML^{-1}T^{-2}$	(kPa)	deformation modulus *ratio between a given normal stress change and the strain change in the same direction (all other stresses being constant)*	module de déformation *quotient de la variation d'une contrainte principale par la déformation obtenue dans la même direction, les autres contraintes restant inchangées.*
υ	-	(-)	Poisson's ratio *ratio between strain changes perpendicular to and in the direction of a given uniaxial stress change*	coefficient de Poisson *rapport entre les deux déformations respectivement perpendiculaire et parallèle à la direction d'une sollicitation uniaxiale*
k_s	$ML^{-2}T^{-2}$	(kN/m^3)	modulus of subgrade reaction *ratio between change of vertical stress on a rigid plate and corresponding change of vertical settlement of the plate*	module de réaction *quotient de la variation de la contrainte verticale sur une plaque rigide par la variation correspondante de tassement vertical de la plaque*
E_{oed}	$ML^{-1}T^{-2}$	(kPa)	oedometric modulus	module oedométrique
C_c	-	(-)	compression index *slope of virgin compression curve in a semi-logarithmic plot "effective pressure-void ratio": $Cc = -\Delta e / \Delta lg\sigma'$*	indice de compression *pente de la courbe de compression vierge dans un diagramme semi-logarithmique "contrainte effective-indice des vides": $Cc = -\Delta e / \Delta lg\sigma'$*
c_v	L^2T^{-1}	(m^2/s)	coefficient of consolidation	coefficient de consolidation
T_v	-	(-)	time factor *defined as $T_v = t\, c_v / d^2$, t being the time elapsed since application of a change in total normal stress*	facteur temps *défini par $T_v = t\, c_v / d^2$, t étant le temps écoulé depuis l'application d'une variation de contrainte normale totale*

τ_f	$ML^{-1}T^{-2}$	(kPa)	shear strength	résistance au cisaillement
			shear stress at failure in rupture plane through a given point	*contrainte de cisaillement, lors de la rupture, dans le plan de rupture en un point donné*
c'	$ML^{-1}T^{-2}$	(kPa)	effective cohesion	cohésion effective
ϕ'	-	(°)	effective angle of internal friction	angle de frottement effectif
			shear strength parameter with respect to effective stresses. Defined by the equation: $\tau_f = c' + \sigma' \tan \phi'$	*paramètre de résistance au cisaillement en contraintes effectives, défini par l'équation :* $\tau_f = c' + \sigma' \operatorname{tg} \phi'$
c_u	$ML^{-1}T^{-2}$	(kPa)	apparent cohesion	cohésion apparente
ϕ_u	-	(°)	apparent angle of internal friction	angle de frottement apparent
			shear strength parameters with respect to total stresses. Defined by the equation: $\tau_f = c_u + \sigma \tan \phi_u$. *In undrained situation, with saturated cohesion soils,* c_u *is also called undrained shear strength*	*paramètres de résistance au cisaillement en contraintes totales, défini par l'équation:* $\tau_f = c_u + \sigma \operatorname{tg} \phi_u$. *Pour les sols cohérents saturés en sollicitation non drainée,* c_u *est appelé également cohésion non drainée*
CBR	-	(-)	California Bearing Ratio	indice portant californien
q_c	$ML^{-1}T^{-2}$	(kPa)	static point resistance (or cone resistance)	résistance de pointe statique (ou résistance de cône)
			average pressure acting on the conical point in the standard static penetration test	*pression moyenne agissant sur la pointe conique dans l'essai normalisé de pénétration statique*
f_s	$ML^{-1}T^{-2}$	(kPa)	local side friction	frottement latéral unitaire
			average unit side friction acting on the friction sleeve in the standard static cone penetration test	*frottement latéral par unité de surface du manchon de frottement dans l'essai normalisé de pénétration statique au cône*
N	-	(-)	SPT blow count	nombre de coups SPT
			standardized result of the Standard Penetration Test (number of blows for 30 cm)	*résultat normalisé de l'essai SPT (nombre de coups pour 30 cm)*
P_l	$ML^{-1}T^{-2}$	(kPa)	pressuremeter limit pressure	pression limite pressiométrique
			limit pressure defined in the standard Ménard pressuremeter test	*pression limite définie dans l'essai pressiométrique normal Ménard*
E_M	$ML^{-1}T^{-2}$	(kPa)	pressuremeter modulus	module pressiométrique
			conventional modulus defined in the standard Ménard pressuremeter test.	*module conventionnel défini dans l'essai pressiométrique normal Ménard*

3.5 Consistency of soils
Consistance des sols

w_L	-	(-.%)	liquid limit *water content of a remolded soil at transition between liquid and plastic states (determined by a standard laboratory test)*	limite de liquidité *teneur en eau d'un sol remanié au point de transition entre les états liquide et plastique (déterminée par un essai normalisé de laboratoire)*
w_p	-	(-, %)	plastic limit *water content of a remolded soil at transition between plastic and semi-solid states (determined by a standard laboratory test)*	limite de plasticité *teneur en eau d'un sol remanié au point de transition entre les états plastique et solide avec retrait (déterminée par un essai normalisé de laboratoire)*
w_s	-	(-, %)	shrinkage limit *maximum water content at which a reduction of water content will not cause a decrease in volume of the soil mass*	limite de retrait *teneur en eau maximale pour laquelle une réduction de teneur en eau ne cause plus de diminution de volume du sol*
I_p	-	(-,%)	plasticity index *difference between liquid and plastic limits*	indice de plasticité *différence entre les limites de liquidité et de plasticité*
I_L	-	(-,%)	liquidity index *defined as* $(w - w_p) / I_p$	indice de liquidité *défini par* $(w - w_p) / I_p$
I_C	-	(-,%)	consistency index *defined as* $(w_L - w) / I_p$	indice de consistance *défini par* $(w_L - w) / I_p$
e_{max}	-	(-)	void ratio in loosest state *maximum void ratio obtainable by a standard laboratory procedure*	indice des vides dans l'état le plus lâche *maximum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
e_{min}	-	(-)	void ratio in densest state *maximum void ratio obtainable by a standard laboratory procedure*	indice des vides dans l'état le plus dense *minimum de l'indice des vides obtenu dans un essai normalisé de laboratoire*
I_D	-	(-)	density index *defined as* $(e_{max} - e) / (e_{max} - e_{min})$	indice de densité *défini par* $(e_{max} - e) / (e_{max} - e_{min})$

4. PROPERTIES OF GEOTEXTILES
PROPRIETES DES GEOTEXTILES

4.1 Solid elements and their distribution
Eléments solides et leur distribution

ρ_f	ML^{-3}	(kg/m^3)	density of fibers or filaments	masse volumique des fibres ou des filaments
d_f	L	(µm)	diameter of fibers or filaments	diamètre des fibres ou filaments

λ	ML^{-1}	(tex)	linear density of yarns, fibers or filaments *mass per unit length of yarns, fibers or filaments*	masse linéique (titre) des fils, fibres ou filaments *masse par unité de longueur des fils, fibres ou filaments.*
O_n	L	(mm, µm)	n-percent opening size *opening size corresponding to n-percent of finer openings (as measured by a test to be defined, such as sieving glass beads) (e.g. O_{95}, opening size corresponding to 95% of finer openings, sometimes called "Equivalent Opening Size")*	ouverture à n pourcent *dimension d'ouverture telle que n pourcent d'ouvertures soient plus petites (préciser l'essai utilisé, par exemple tamisage de billes de verre) (par exemple: O_{95}, dimension de l'ouverture telle que 95% des ouvertures soient plus petites, quelquefois appelée "Dimension d'Ouverture Equivalente")*

Note: Standard sieve numbers vary from one country to another. Opening sizes expressed by a standard sieve number would not be understood by most of the readers. Consequently, authors are strongly requested to express opening size in millimeters (mm) or microns (µm). Standard sieve numbers can be given in parenthesis after the value in mm or µm.

Note: Les numéros standards de tamis diffèrent d'un pays à l'autre. Les dimensions d'ouverture exprimées par un numéro standard de tamis ne signifieraient rien pour la plupart des lecteurs. En conséquence, il est fortement conseillé aux auteurs d'exprimer les dimensions d'ouverture en millimètres (mm) ou en microns (µm). Le numéro standard de tamis peut être donné entre parenthèses après la valeur en mm ou µm.

4.2 Physical properties of a geotextile
Propriétés physiques d'un géotextile

μ	ML^{-2}	(kg/m^2, g/m^2)	mass per unit area *ratio between mass and area of geotextile*	masse surfacique *masse par unité de surface d'un géotextile*
e	–	(-)	void ratio *ratio between volume of voids and volume of solid elements of a geotextile*	indice des vides *quotient du volume des vides par le volume des éléments solides d'un géotextile*
n	–	(-, %)	porosity *ratio between volume of voids and total volume of geotextiles*	porosité *quotient du volume des vides par le volume total d'un géotextile*
T_g	L	(mm)	thickness of geotextile	épaisseur d'un géotextile

4.3 Hydraulic properties of a geotextile
Propriétés hydrauliques d'un géotextile

k_n	LT^{-1}	(m/s)	coefficient of normal permeability of a geotextile	coefficient de perméabilité normale d'un géotextile
k_p	LT^{-1}	(m/s)	coefficient of permeability in the plane of a geotextile	coefficient de perméabilité dans le plan d'un géotextile
Ψ	T^{-1}	(s^{-1})	permittivity of a geotextile ($\Psi = k_n/H_g$)	permittivité d'un géotextile ($\Psi = k_n/H_g$)
θ	L^2T^{-1}	(m^2/s)	transmissivity of a geotextile ($\theta = k_p/H_g$)	transmissivité d'un géotextile ($\theta = k_p H_g$)

4.4 Mechanical properties of a geotextile
Propriétés mécaniques d'un géotextile

α_ε	MT^{-2}	(kN/m)	force per unit width of the geotextile at a given elongation ε (e.g. α_{30} is the force per unit width of the geotextile at 30% elongation)	force par unité de largeur du géotextile pour une élongation ε donnée (par exemple: α_{30} est la force par unité de largeur du géotextile pour une élongation de 30%)
α_f	MT^{-2}	(kN/m)	force per unit width of the geotextile at failure	force par unité de largeur du géotextile à la rupture
J	MT^{-2}	(kN/m)	"modulus" of the geotextile	"module" du géotextile
			Note: The "modulus" of a geotextile is defined as a force per unit width while modulus is usually defined as a force per unit area.	Note: Le "module" d'un géotextile est défini comme une force par unité de largeur, alors qu'un module represente généralement une force par unité de surface.
J_i	MT^{-2}	(kN/m)	initial (tangent) "modulus" of the geotextile	"module" (tangent) initial du géotextile
$J_{t\varepsilon}$	MT^{-2}	(kN/m)	tangent "modulus" of the geotextile at elongation ε (e.g. J_{t30} is the tangent "modulus" of the geotextile at 30% elongation)	"module" tangent du géotextile pour une élongation ε donnée (par exemple J_{t30} est le "module" tangent du géotextile pour une élongation de 30%)
$J_{sec\ \varepsilon}$	MT^{-2}	(kN/m)	secant "modulus" of the geotextile between 0 and elongation ε (e.g. $J_{sec\ 30}$ is the secant "modulus" of geotextile between 0 and 30% elongation)	"module" sécant du géotextile entre 0 et l'élongation ε (par exemple, $J_{sec\ 30}$ est le "module" sécant du géotextile entre 0 et 30% d'élongation)
υ_g	-	(-)	Poisson's ratio of a geotextile	coefficient de Poisson d'un géotextile
ζ_t	M^2T^{-2}	(N/tex)	tenacity of a thread *quotient of the tensile force at failure of a thread to its linear density*	tenacité d'un fil *quotient de la force de traction à la rupture d'un fil par sa masse linéique*
ζ_g	M^2T^{-2}	(J/g)	tenacity of a geotextile *quotient of the force per unit width at failure of a geotextile to its mass per unit area*	tenacité d'un géotextile *quotient de la force par unité de largeur d'un géotextile à la rupture par sa masse surfacique*

(Note: 1 J/g = 1000 N/tex)

F_G	MLT^{-2}	(N,kN)	breaking force of geotextile as measured in grab test	résistance du géotextile dans l'essai d'arrachement
F_p	MLT^{-2}	(N, kN)	breaking force of geotextile in a puncture test (to be defined)	résistance du géotextile à la perforation (préciser l'essai utilisé)
F_T	MLT^{-2}	(N, kN)	breaking force of geotextile in a tear test (to be defined)	résistance du géotextile à la déchirure (préciser l'essai utilisé)
p_B	$ML^{-1}T^{-2}$	(kPa, MPa)	bursting pressure of a geotextile.	résistance du géotextile à l'éclatement

5. PRACTICAL PROBLEMS
PROBLEMES PRATIQUES

5.1 General
Généralités

FS	–	(-)	Factor of safety	Coefficient de sécurité

5.2 Earth pressure
Pression des terres

δ	–	(°)	angle of wall friction	angle de frottement sol-mur
			angle of friction between wall and adjacent soil	*angle de frottement entre le mur et le sol adjacent*
a	$ML^{-1}T^{-2}$	(kPa)	wall adhesion	adhésion sol-mur
			adhesion between wall and adjacent soil	*adhésion entre le mur et le sol adjacent*
K_a, K_p	–	(-)	active and passive earth pressure coefficients	coefficients de poussée et de butée des terres
			dimensionless coefficients used in expressions for active and passive earth pressure	*coefficients sans dimension intervenant dans les expressions de poussée et de butée*
K_o	–	(-)	coefficient of earth pressure at rest	coefficient de pression des terres au repos
			ratio of lateral to vertical effective principal stress in the case of no lateral strain and a horizontal ground surface	*rapport entre les contraintes effectives horizontale et verticale à déformation horizontale nulle et lorsque la surface libre du sol est horizontale*

5.3 Foundations and embankments
Fondations et remblais

B	L	(m)	breadth of foundation or embankment	largeur de la fondation ou du remblai
L	L	(m)	length of foundation or embankment	longueur de la fondation ou du remblai
D	L	(m)	depth of foundation or base of embankment beneath ground	profondeur de la fondation ou de la base du remblai au-dessous du niveau du terrain
s	L	(m)	settlement	tassement
U	-	(-,%)	degree of consolidation	degré de consolidation
			ratio of settlement at a given time to final settlement	*quotient du tassement à un temps donné par le tassement final*

5.4 Slopes
Pentes

H	L	(m)	vertical height of slope	hauteur verticale du talus
D	L	(m)	depth below toe of slope to hard stratum	profondeur du substratum rigide sous le pied du talus
β	–	(°)	angle of slope to horizontal	angle d'inclinaison du talus avec l'horizontale

Conversion factors
Facteurs de conversion

Conversion factors between the English system and SI are as follows:

Length, area, volume	
1 mil	= 25.4 microns (µm)
1 inch	= 25.4 millimeters (mm)
1 foot	= 0.305 meter (m)
1 yard	= 0.914 meter (m)
1 square foot	= 9.29×10^{-2} square meters (m^2)
1 acre	= 4047 square meters (m^2)
1 cubic foot	= 2.83×10^{-2} cubic meters (m^3)
1 gallon	= 3.79×10^{-3} cubic meters (m^3)
1 gallon	= 3.79 liters (L)
Mass	
1 ounce	= 28.3 grams (g)
1 pound	= 0.454 kilogram (kg)
1 ton (US)	= 907 kilograms (kg)
1 long ton (GB)	= 1016 kilograms (kg)
Linear Density	
1 denier	= 0.111 milligram per meter (mg/m)
1 denier	= 0.111 tex (tex)
Mass per unit area	
1 ounce per square yard	= 33.9 gram per square meter (g/m^2)
1 ounce per square yard	= 3.39×10^{-2} kilogram per square meter (kg/m^2)
Density	
1 pound (mass) per cubic foot	= 16.0 kilograms per cubic meter (kg/m^3)
Force	
1 pound	= 4.45 newtons (N)
1 kip	= 4.45 kilonewtons (kN)
Force per unit width or per unit length	
1 pound per inch	= 175 newtons per meter (N/m)
1 pound per inch	= 0.175 kilonewton per meter (kN/m)
Stress, pressure	
1 pound per square inch	= 6895 pascals (Pa)
1 pound per square inch	= 6.89 kilopascals (kPa)
1 pound per square foot	= 47.9 pascals (Pa)
1 kip per square foot	= 47.9 kilopascals (kPa)
1 ton (US) per square foot	= 95.8 kilopascals (kPa)
Unit weight or reaction modulus	
1 pound (force) per cubic foot	= 157 newtons per cubic meter (N/m^3)
1 pound (force) per cubic foot	= 0.157 kilonewton per cubic meter (kN/m^3)

Velocity, Coefficient of Permeability

1 centimeter per second	= 0.010 meter per second (m/s)

Flow rate

1 gallon per minute	= 6.31×10^{-2} liter per second (L/s)
1 gallon per minute	= 6.31×10^{-5} cubic meter per second (m^3/s)
1 cubic foot per second	= 2.83×10^{-2} cubic meter per second (m^3/s)
1 cubic foot per minute	= 0.472 liter per second (L/s)
1 cubic foot per minute	= 4.72×10^{-4} cubic meter per second (m^3/s)
1 gallon per day	= 4.38×10^{-5} liter per second (L/s)

Dynamic viscosity

1 poise	= 0.100 kilogram per meter-second (kg/sm)
1 pound (force) second per square foot	= 0.478 kilogram per meter-second (kg/sm)
	(Note: 1 kg/sm = 1 Pa.s)

Gravity

g = 32.2 feet per square second	= 9.81 meters per square second (9.81 m/s^2)

U. S. Sieve Designation	Opening Size	
	mm	µm
200	0.075	75
170	0.090	90
140	0.106	106
120	0.125	125
100	0.150	150
80	0.180	180
70	0.212	212
60	0.250	250
50	0.300	300
45	0.355	355
40	0.425	425
35	0.500	500
30	0.600	600
25	0.710	710
20	0.850	850
18	1.000	
16	1.18	
14	1.40	
12	1.70	
10	2.00	
8	2.36	
7	2.80	
6	3.35	
5	4.00	
4	4.75	

Second International Conference on Geotextiles

Deuxième Congrès International des Géotextiles

AUGUST 1–6, 1982

1–6 AOUT, 1982

LAS VEGAS, NEVADA

U.S.A.

PROCEEDINGS
COMPTES-RENDUS

Volume IV

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The Plastics Experts.

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DEUXIEME CONGRES INTERNATIONAL DES GEOTEXTILES

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Volume IV

IN MEMORIAM
A LA MEMOIRE DE

T. ALLAN HALIBURTON
1938–1982

Dr. T. Allan (Al) Haliburton, who actively participated in the organization of the Second International Conference on Geotextiles as a member of the Technical Program Committee, passed away shortly after the conference.

Dr. T. Allan Haliburton received the BSCE degree in Civil Engineering and the MSCE and Ph.D. degrees in Civil/Geotechnical Engineering from the University of Texas, Austin, and joined the faculty of the School of Civil Engineering at Oklahoma State University, Stillwater, Oklahoma, in 1965. Dr. Haliburton held the rank of Professor of Civil Engineering at Oklahoma State University and was also President of Haliburton Associates, an engineering consulting firm. He was a member of ASCE and a Registered Professional Engineer in 13 states.

Dr. Haliburton first became involved with geotextiles during the early 1970's and was involved in both research and field applications of geotextiles for the U.S. Army Corps of Engineers, the Association of American Railroads, the U.S. Air Force, the Federal Highway Administration, and several fabric manufacturers, as well as numerous private industrial and engineering groups.

Dr. Haliburton was truly a pioneer in the engineering application of geotextiles. His unending efforts and practical approach to complex problems greatly enhanced the expansion of the entire industry, and his strength and presence will be long missed by colleagues and clients alike. We remember him here with the respect and fond admiration due to a man of his great stature.

Dr. T. Allan (Al) Haliburton, qui a participé activement à l'organisation du Deuxième Congrès International des Géotextiles comme membre du Comité du Programme Technique, est décédé soudainement peu de temps après ce congrès.

Dr. Haliburton a obtenu le diplôme de baccalauréat en génie civil ainsi que les diplômes de maîtrise et de doctorat en géotechnique à l'Université de Texas, à Austin. Il a rejoint en 1965 le département de génie civil de l'Université d'Etat d'Oklahoma, à Stillwater, où il a occupé la chaire de professeur en génie civil. Il était en outre président de la firme d'ingénieurs conseils Haliburton Associates. Il était membre de la Société Américaine des Ingénieurs Civils ainsi qu'ingénieur agréé dans 13 états des Etats Unis.

Dr. Haliburton a commencé à s'intéresser de près aux géotextiles dès le début des années 70. Il a effectué des recherches sur les géotextiles et travaillé sur leurs applications pour le compte du Corps des Ingénieurs de l'Armée Américaine, de l'Association des Chemins de Fer Américains, de l'Armée de l'Air, de l'Administration Fédérale des Routes, de plusieurs fabricants de géotextiles ainsi que de nombreuses firmes du secteur privé.

Dr. Haliburton fut un pionnier dans le domaine des applications des géotextiles. Ses efforts incessants ainsi que sa façon pratique d'aborder les problèmes complexes ont énormément contribué à l'expansion de l'industrie entière; sa personnalité et sa présence marqueront pendant longtemps ses collègues et ses clients. Nous nous souvenons de lui ici avec tout le respect et la profonde admiration dus à un homme de sa stature.

Conference Invited Papers

GIROUD, Jean-Pierre
Woodward-Clyde Consultants, Chicago, IL, USA
Conference Chairman

Opening Address
Allocution d'ouverture

Ladies and Gentlemen:

On behalf of the organizing committee, I declare the Second International Conference on Geotextiles open.

Welcome

I am pleased to extend a warm welcome to all of the participants. It is inspiring to see that so many people traveled to Las Vegas, to devote a week to geotextiles. Participants to this conference have come from almost forty different countries, from all continents. This clearly demonstrates the worldwide interest for geotextiles.

During this week the world of geotextiles will belong to you. You will have access to five years of progress made since the first international conference, and you will have many opportunities to meet with colleagues from all over the world.

Thank you for coming, for without you the efforts undertaken by the sponsors and the organizing committee would have been in vain.

I would now like to say a few words about the organization of this conference.

Sponsorship

This conference is sponsored by IFAI, the Industrial Fabrics Association International. The fact that this conference is sponsored by an association of geotextile manufacturers demonstrates the vitality of the industry, and we must congratulate the IFAI for having undertaken the long and costly venture as sponsor of this conference.

I take this opportunity to thank the other associations that have contributed to the promotion of this conference. These include: ASTM, the American Society for Testing and Materials, ASCE, the American Society of Civil Engineers, and TRB, the Transportation Research Board, all three from the United States; also RILEM, the International Union of Testing and Research Laboratories for Materials and Structures, and ENPC, l'Ecole Nationale des Ponts et Chaussées, from Europe; and CGSB, the Canadian General Standards Board.

I would also like to thank all the persons who helped in the promotion of the conference in many countries, which again clearly demonstrated international interest in geotextiles.

Organizing Committee

Now, I am pleased to introduce to you the members of the organizing committee. Professor Dick Bell, assisted by Dr Lee Murch, had the difficult task of organizing the technical sessions. Dick will present the technical program later this morning.

In the organizing committee: ASTM is represented by Ivan Johnson who was chairman of the ASTM committee on Soil and Rock for many years; ASTM is also represented by Trudy Raumann, the first chairman of the ASTM subcommittee on Geotextiles; ASCE is represented by professor Ara Arman, chairman of ASCE Soil Placement and Improvement Committee; RILEM is represented by Dr. Etienne Leflaive who promoted the conference in Europe; and, last but not least, IFAI is represented by Joe Fluet, chairman of the Geotextile Division of the IFAI, and Steve Warner, who is Secretary General of the organizing committee and has been in charge of the many practical aspects of this conference.

The exhibition, to which we are all invited Tuesday evening and Wednesday, has been organized by Joe Fluet and Steve Warner.

The geotextiles

The existence of this conference indicates the growing importance of geotextiles. Today, geotechnical engineering would be unthinkable without geotextiles. Several years ago, geotextiles were mostly used to replace a sand layer or to separate two materials. Basically they were used only where there was a horizontal line in the cross-section of a project! Salesmen quickly understood that there are many horizontal lines in geotechnical projects and the geotextile market grew rapidly. Today, some designers are aware of the many possibilities offered by geotextiles and they do not restrict the use of geotextiles to the famous horizontal lines. However, many designers are not as yet well-enough informed about geotextiles. They may have the right attitude, which begins with attending this conference. But there are also two kinds of negative attitudes. One consists of avoiding the use of geotextiles. The other negative attitude consists of using over-simplistic criteria for the selection of geotextiles. For example, in too many cases, selection of geotextiles is presented in terms of wovens versus nonwovens. Such a simplistic approach may support one's personal convictions or serve commercial purposes. But, the development of geotextiles is the result of a rational process, not of simplistic approaches as it will be shown later this morning by Etienne Leflaive.

Rational process implies research. During the past ten years, considerable research effort has been undertaken on geotextiles. This effort has led to a better understanding of not only geotextiles but also fabric mechanics and soil mechanics. For example, theories of soil mechanics are being improved to explain the behavior of soil-fabric systems. Geotextiles are not only a technical and commercial success, they are also the subject of a new scientific discipline with beneficial effects on other disciplines. The creativity of geotextile research is the result of combined efforts of textile and geotechnical specialists.

The international community of geotextiles

The success of geotextiles has led to the creation of committees in many countries. These committees prepare standards and specifications. Geotextiles are transported from continent to continent and must be tested with equivalent procedures. Coordination between the various committees is therefore necessary. This subject will be discussed, later this morning, by Trudy Raumann.

During this conference, much will be done to promote international cooperation. The work done by various committees will be presented during a session devoted to international standards. Also, a meeting has been scheduled Wednesday evening to discuss the creation of an International Society on Geotextiles. This society could perform many tasks such as: promote the exchange of information, organize conferences (including a third international conference on geotextiles), contribute to the publication of a journal on geotextiles, and many other activities. This is a very important matter for the future of geotextiles and I urge all interested people to attend the meeting Wednesday evening.

International matters involve various languages. I will therefore say a few words in French to conclude.

Quelques mots en français

Ainsi, Mesdames et Messieurs, le Deuxième Congrès International des Géotextiles est ouvert. Cinq ans déjà depuis le congrès de Paris.

Ce congrès avait eu un succès immense,
dépassant les espérances.
Une science naissait,
s'étonnant de son succès.
L'atmosphère, à Paris,
était à l'euphorie.

La communauté internationale des géotextiles se découvrait. Qu'il est bon, aujourd'hui, de la savoir aussi vivante. Il n'y a pas de meilleur symbole de vitalité internationale que de voir se rencontrer des spécialistes parlant des langues différents. On peut remercier l'IFAI d'avoir dégagé les crédits permettant d'assurer la traduction simultanée entre les deux langues officielles du congrès.

Le premier congrès avait eu lieu en Europe, berceau du grand essor des géotextiles au début de la décennie précédente. Il était tout indiqué que le deuxième congrès ait lieu en Amérique du Nord, partie du monde où le marché des géotextiles s'est le plus développé ces dernières années. Demain, espérons-le, le succès croissant des géotextiles entraînera un congrès international sur un autre continent. L'important, aujourd'hui, est que des participants, venus de tous les continents, se retrouvent pendant une semaine ensemble, pour apprendre ensemble, pour travailler ensemble, pour préparer l'avenir des géotextiles, ensemble.

LEFLAIVE, Etienne
Laboratoire Central des Ponts et Chaussées, Paris, FRANCE

Geotextiles: Just a New Technique or a Far-Reaching Innovation?

"Les Géotextiles: une simple nouveauté technique ou une innovation fondamentale?"

Mon but, au cours de l'exposé introductif que j'ai l'honneur de présenter ici aujourd'hui est simplement de réfléchir avec vous, à l'ouverture de ce congrès, pour que nous nous interrogions sur les causes réelles du développement des géotextiles ; il apparaît en effet utile de se poser cette question, pour placer dans une juste perspective les discussions que nous allons avoir au cours de cette semaine.

Il s'agit de savoir si le succès des géotextiles est un succès marginal dû à des causes secondaires ou conjoncturelles, ou bien s'il tient à des causes plus profondes ; dans ce dernier cas, ces causes pourraient être à l'origine d'un développement de longue haleine conduisant les fibres de polymère du statut de matériau mineur qu'elles ont aujourd'hui dans les travaux publics, à celui de matériau majeur du génie civil, avec toutes les conséquences techniques et économiques d'une telle évolution.

En outre, connaître un peu mieux les causes du développement des géotextiles peut avoir différents usages :

- par exemple avoir des arguments pour convaincre ceux qui ne croient pas que les géotextiles soient une innovation importante ; ils sont encore nombreux

- approfondir le dialogue entre la profession textile et celle du génie civil en sachant mieux situer les géotextiles par rapport aux autres matériaux et en percevant de façon juste et complète ce que nous pouvons en attendre ;

- enfin voir plus clairement les voies à explorer et les moyens nécessaires pour la recherche et le développement, en comprenant mieux le rôle et la place spécifique des géotextiles.

Je vais donc essayer d'apporter quelques éléments de réponse à la question du pourquoi des géotextiles.

*

Je rappelerai tout d'abord les fonctions des nappes géotextiles dont nous avons vu le développement au cours des années passées. Ce sont les rôles de

1 - séparation ; exemple : piste de chantier en gravier propre sur sol boueux ;

2 - filtre, lorsque le géotextile tapisse les parois d'une tranchée drainante ;

3 - drain, si le géotextile peut transporter de l'eau dans son épaisseur ;

4 - renforcement, lorsque l'apport mécanique du textile est suffisant pour s'opposer aux déformations du sol.

Dans beaucoup de cas pratiques, plusieurs de ces fonctions sont assurées simultanément. C'est ainsi que dans la construction d'un remblai sur sol compressible la nappe géotextile placée entre le sol naturel et le remblai sert de séparation dans la réalisation de la première couche, de filtre lors de la consolidation du sol compressible, de renfort mécanique de l'ensemble et parfois de drain en facilitant l'évacuation latérale de l'eau de consolidation.

La connaissance de ces fonctions nous permet de comprendre l'utilité des géotextiles ; elle ne nous dit pas, cependant, pourquoi les textiles sont davantage capables de remplir ces fonctions que d'autres matériaux, ou quels avantages supplémentaires ils ont sur ces derniers.

C'est précisément ce que je voudrais essayer de voir avec vous : quels sont les éléments qui rendent les géotextiles particulièrement intéressants ?

*

Les facteurs qui font l'intérêt des géotextiles, j'en ai retenu cinq que je résume par les termes suivants :

- performances des fibres,
- vertus du mariage sol-fibres,
- intérêt des structures déformables,
- économies de transport,
- simplicité et fiabilité des méthodes de construction.

*

Les qualités techniques propres des fibres de polymère sont, bien sûr, l'élément fondamental qui les rendent intéressantes, ces qualités étant principalement :

- la résistance, exprimée en performance pour un coût de matière donné,
- la gamme de modules où ils se situent,
- la stabilité chimique.

Je ne m'étendrai pas sur ces aspects, qui seront évoqués également par madame Raumann, et je me limiterai seulement à une observation non technique : comment se fait-il que ces qualités qui sont connues des chimistes et des textiliens depuis plusieurs dizaines d'années, n'aient été découvertes que récemment par les ingénieurs du génie civil ? Un premier élément d'explication se trouve certainement dans l'évolution des prix des matériaux ; je pense cependant qu'au-delà de cette question du prix il y a une part d'explication dans le fait que le génie civil d'une part, le textile d'autre part, sont deux mondes un peu isolés dans l'univers technico-scientifique.

Le génie civil n'est pas une industrie : son organisation et sa psychologie lui sont propres, en raison de la dispersion géographique des chantiers, du caractère toujours particulier de cnacun d'eux et du rôle joué par la nature à laquelle l'homme est directement confronté dans les travaux. Les relations du génie civil avec le reste de la communauté scientifique sont influencés par ces particularités.

Le textile, lui, me semble-t-il, est influencé par le fait que son principal débouché est l'habillement ; comme un vêtement n'est pas quelque chose qui se calcule, l'industrie textile apparaît, aux yeux de ceux qui ne la connaissent pas, comme un domaine de faible technicité, et sa soumission à la mode n'améliore pas son image auprès de ceux qui prétendent à des attitudes rationnelles.

Pourtant l'industrie textile est un domaine où science et technique sont bien présents ; mais ici intervient un autre élément, qui est le décalage historique qu'il y a eu entre le développement industriel du textile, qui est très ancien, et celui des autres techniques. Il en est résulté une certaine coupure du monde textile par rapport aux autres professions, coupure qui subsiste probablement encore.

La distance psychologique qui séparait au départ les producteurs de fibres et les constructeurs était donc considérable ; la rencontre a maintenant eu lieu, mais il reste toujours à poursuivre l'effort d'information des techniciens du génie civil sur la réalité de l'industrie textile, de même qu'est toujours à approfondir la prise de conscience par les professionnels du textile des particularités du marché des travaux publics.

Au sujet des performances des fibres j'ai parlé de résistance, de module et de stabilité chimique. Une autre caractéristique qui joue un grand rôle est le diamètre des fibres ; il est le plus souvent de quelques dizaines de microns et permet aux géotextiles d'avoir les excellentes propriétés hydrauliques que nous leurs connaissons, à savoir forte perméabilité associée à un diamètre de filtration faible, de l'ordre de grandeur de la centaine de microns. Une telle combinaison perméabilité élevée-diamètre de filtration petit n'est pas possible avec un matériau granulaire ; c'est un avantage spécifique des structures fibreuses.

Notons en passant combien la stabilité chimique est importante pour des matériaux ayant une aussi grande surface spécifique. Faire durer des fibres de 30 microns de diamètre plusieurs dizaines d'années dans des sols marécageux n'est possible qu'avec une matière remarquablement stable, ce qui est le cas des polymères les plus utilisés.

*

Le second facteur qui fait l'intérêt des géotextiles, je l'ai intitulé : "Vertus du mariage sol-fibres".

Nous venons d'en voir un des aspects, qui est l'adaptation des textiles à la fonction de filtre dans les sols.

Un autre aspect favorable du mariage sol-fibres est l'adaptation des modules des fibres et des textiles synthétiques au bon fonctionnement mécanique du composite sol-géotextile. Les déformabilités des deux éléments d'un composite doivent en effet être compatibles pour que l'on puisse utiliser simultanément la capacité de résistance des deux composants ; or beaucoup de sols naturels sont assez déformables ; on ne peut donc leur associer un élément trop rigide ; les textiles se révèlent être des éléments de caractéristiques particulièrement adaptées.

Préciser ce point en citant des chiffres exigerait des développements trop longs pour cet exposé, car la mécanique des systèmes sol-géotextile n'est pas simple, comme l'illustreront différentes communications du congrès. La mécanique des sols est en effet complexe et la mécanique des textiles encore relativement peu explorée ; la mécanique des systèmes sol-géotextile n'est donc pas un domaine où l'on puisse trancher en quelques mots.

Insistons cependant sur l'avantage énorme qu'il y a à associer un élément de continuité au milieu discontinu qu'est un sol. C'est en effet non seulement en augmentant sa résistance que le textile est utile mécaniqement, mais aussi et peut-être surtout en créant la continuité qui modifie profondément les déformations possibles dans la masse du sol.

Il apparaît donc bien clairement, et de nombreux auteurs l'ont déjà montré, que l'association des deux matériaux est exceptionnellement heureuse, sous les deux angles sous lesquels la géotechnique impose toujours d'examiner les matériaux : l'hydraulique et le mécanique.

*

Les fibres chimiques sont donc d'excellents composants pour améliorer les sols. Mais a-t-on vraiment besoin d'améliorer les sols ? ne suffit-il pas de béton et d'acier pour construire et donner leur solidité aux ouvrages ? Il y a deux réponses à cela :

1 - même si l'on ne construit pas avec le sol, on construit toujours sur le sol ; et beaucoup de terrains où l'on doit construire sont humides, mous, sujets à l'érosion, aux tremblements de terre ...

2 - l'économie des projets conduit souvent, pour certains types d'ouvrage, à employer le sol comme matériau.

De nombreux ouvrages doivent être construits sur des sols peu stables. Quel que soit l'ouvrage à réaliser, il faut alors s'accommoder du caractère plus ou moins déformable, compressible, peu résistant et hétérogène du terrain. Plus la structure que l'on veut édifier est exigeante du point de vue des déformations tolérées, plus le problème de la fondation est aigu ; s'il ne peut être résolu par des fondations profondes reportant les charges sur un substratum rigide, on ne peut éluder la question de la liaison entre le milieu naturel mou et l'édifice construit rigide.

C'est là que des structures d'une déformabilité intermédiaire sont nécessaires, qui supportent les tassements différentiels du sol et qui constituent ensuite pour l'ouvrage un support suffisant et homogène.

Lorsqu'il n'est pas nécessaire que la construction réalisée soit elle-même rigide, une digue par exemple, on doit la concevoir avec une technique tolérant les déformations du sol naturel sous son poids. C'est un des motifs de l'emploi de la terre comme matériau de construction de nombreux barrages, que de pouvoir s'adapter aux tassements des dépôts d'alluvions sur lesquels ils sont construits.

On voit alors un intérêt essentiel des géotextiles : c'est de permettre d'utiliser le sol comme matériau, en conservant ses avantages de faible coût et de déformabilité, et en mettant en plus à profit les qualités mécaniques et hydrauliques des textiles. Les géotextiles permettent de concevoir des structures de niveau de déformabilité adapté, utilisant bien les capacités du sol et des fibres. Ces structures répondent à un besoin des constructeurs à la fois permanent, car les sols de fondation médiocres sont les plus nombreux, et très vaste, car les digues, les ouvrages maritimes, les barrages, les routes, les aérodromes, les terre-plains industriels, les ouvrages de fondation proprement dits représentent une part importante du génie civil.

Ces structures sol-fibres sont encore très peu développées dans la pratique à l'heure actuelle, car il faut bien savoir qu'il s'agit de possibilités profondément nouvelles. Certes, on peut dire que l'emploi de fibres naturelles, de branchages ou de paille est très ancien ; c'est vrai. Mais cela ne représente pratiquement rien dans la technique d'aujourd'hui, pour des raisons de main d'oeuvre, de coût, de performance et de durabilité. Les fibres chimiques, elles, permettent de reconcevoir complètement la technique des ouvrages en terre et des fondations, domaine où il est tout à fait certain que des progrès sont profondément utiles et ont un vaste champ d'application.

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Après les aspects techniques que nous venons de voir, deux autres facteurs sont des atouts importants pour les géotextiles.

Le premier est la réduction des dépenses de transport résultant de la légèreté des géotextiles. Le meilleur exemple est le filtre ou la couche anti-contaminante : un géotextile de 250 g/m^2 peut remplacer une couche granulaire de 15 cm d'épaisseur qui pèse 250 kg/m^2, soit 1 000 fois plus. Pour 1 kilomètre de chaussée de 8 m de large, on passe de 2 000 tonnes de matériau, ce qui exige 200 voyages de camions de 10 tonnes de charge utile, à 2 tonnes, ce qui demande 1 seul voyage d'un camion léger.

Les économies de transport dues à l'emploi des géotextiles peuvent aussi être considérables en permettant d'utiliser les matériaux naturels disponibles sur le site, au lieu de faire venir d'autres matériaux d'une plus ou moins grande distance.

Ces économies de transport sont d'autant plus importantes que la région où l'on construit est pauvre en matériaux naturels convenables, que les installations de préparation de matériaux y sont peu nombreuses et que le coût au kilomètre de transport y est élevé, soit qu'il s'agisse de zone urbaine, de région montagneuse, ou autre.

J'aurais voulu vous donner quelques chiffres sur cette question des transports pour montrer l'ampleur de l'économie que peuvent apporter les géotextiles. Malheureusement, comme je le disais tout à l'heure, les travaux publics sont caractérisés par la dispersion et la diversité des chantiers. Les aspects économiques y sont donc difficiles à appréhender ; d'ailleurs, à l'inverse de l'industrie, c'est un domaine qui semble n'avoir jamais beaucoup intéressé les économistes.

Je profite donc de cette occasion pour émettre un voeu, c'est que l'on engage des études économiques sur l'impact des géotextiles dans les travaux publics. Leur réussite déjà substantielle montre qu'ils sont économiquement justifiés : une étude plus approfondie, notamment sous l'angle des transports, permettrait de convaincre davantage de responsables de l'intérêt de leur développement.

Elle pourrait répondre aussi à la question de savoir si les géotextiles ne sont pas, sous ce même angle, particulièrement adaptés aux travaux dans les pays en voie de développement, alors qu'ils sont souvent considérés dans ces pays comme un luxe.

*

Le dernier facteur qui fait l'intérêt des géotextiles est la garantie qu'ils apportent dans la qualité de la construction.

Cette garantie de qualité résulte à la fois de leur simplicité de mise en oeuvre, de la constance de leur qualité par rapport à la variabilité des matériaux naturels, et du rôle qu'ils jouent dans les ouvrages.

Plus l'exécution est simple, plus elle a de chances d'être bien faite et plus le contrôle de chantier peut être efficace. Mais surtout, comme on l'a vu, les géotextiles introduisent un élément de continuité dans les sols qui a pour effet d'homogénéiser leur comportement et de rendre donc celui-ci moins aléatoire, aussi bien au plan mécanique qu'hydraulique.

Le géotextile a donc pour effet d'accroître la fiabilité des ouvrages en terre ; or ceci est très important car c'est le point faible de ce type d'ouvrage, par ailleurs très souvent attrayant du point de vue du coût.

*

Nous venons donc de voir qu'il existe un ensemble de raisons fondamentales au développement des géotextiles, à savoir :

- les qualités propres des polymères et des textiles chimiques,
- leur compatibilité spécialement favorable avec les sols,
- le besoin de matériaux de construction qui ne soient pas trop rigides,
- l'économie de transport,
- la simplicité et la fiabilité de l'exécution.

Ces facteurs favorables vont-ils entraîner un développement exponentiel des géotextiles pendant de longues années ? Quel volume de production peut-on imaginer ?

Madame Raumann donnera des chiffres sur la situation actuelle et son évolution.

Je voudrais, pour ma part, faire trois remarques sur l'évolution future possible.

D'abord le volume potentiel de l'expansion des géotextiles est certainement très grand, si l'on considère par exemple que le chiffre d'affaire des géotextiles ne représente actuellement que 1 pour mille de celui du ciment, le ciment étant pris simplement comme un indice de l'activité des travaux publics. On voit que le géotextile est encore bien modeste sur le plan quantitatif, et que s'il doit devenir progressivement un matériau courant du génie civil, cela implique des volumes de production beaucoup plus élevés, même s'il ne doit pénétrer que dans certains domaines des travaux publics.

La deuxième remarque est que les bureaux d'études n'introduiront vraiment les géotextiles dans leurs projets que s'ils disposent de données techniques sérieuses et complètes sur ces matériaux. Cela devient progressivement possible par le développement de techniques de mesure appropriées et par les constatations sur les ouvrages existants. Je crois à cet égard que le 2^{e} congrès international des géotextiles marquera une étape très importante sur le chapitre de la qualification des produits et que l'on peut être optimiste pour l'avenir quant à la possibilité, pour les constructeurs, d'avoir des données techniques chiffrées solides.

Il faut cependant être vigilant, dans l'intérêt de tous, pour éviter que telle ou telle présentation des faits n'engendre la confusion ou n'introduise des idées fausses. Pour la durabilité, par exemple, soutenir commercialement un produit en montrant que le concurrent, fait avec un autre polymère, peut être attaqué par tel ou tel composé chimique concentré à chaud, ou peut être ramolli ou fondu à telle ou telle température -non seulement cela n'a généralement aucune signification vis-à-vis des conditions d'emploi réelles, mais cela introduit dans l'esprit des utilisateurs l'idée que les polymères en cause, tous confondus, sont des produits de mauvaise qualité-.

Il vaudrait mieux que les chimistes, et tous les géotextiliens, essaient de redresser l'image de marque actuelle du "plastique", que beaucoup d'applications courantes ont placée très bas et qui n'est pas du tout au niveau des performances réelles de la plupart des polymères.

Je crois que cette observation vaut d'ailleurs autant pour les propriétés mécaniques que pour la durabilité.

La troisième et dernière remarque est que, en tout état de cause, même s'il existe de nombreuses conditions favorables au développement des géotextiles, ce développement ne se fera pas tout seul.

Certes, l'assistance à ce congrès, en ces temps difficiles, montre que les volontés et les compétences sont nombreuses et disponibles, mais je veux dire par là que le développement des géotextiles ne résidera pas, à mon sens, dans la simple augmentation des surfaces de nappes produites et vendues.

Le marché de la nappe que l'on déroule sur le terrain est limité, et l'intérêt des géotextiles, comme on l'a vu, va au-delà de la notion de nappe textile.

Le développement des géotextiles, c'est la pénétration des polymère, sous forme de fibres, dans le génie civil.

Ce développement implique de ne pas s'en tenir à la nappe simplement déroulée, et d'imaginer de nouveaux procédés de construction mettant à profit toutes les possibilités de l'association testile-sol.

En effet dans la plupart des applications actuelles des géotextiles, ceux-ci s'introduisent dans une conception classique qui existait avant que l'on connaisse les géotextiles, en y jouant le plus souvant un rôle d'interface entre deux couches. Le marché actuel est donc essentiellement celui de produits livrés en rouleaux et utilisables directement sans changer beaucoup les habitudes des bureaux d'études et des entrepreneurs.

Mais il existe un autre marché, le marché de l'avenir, qui est celui où les notions de polymère, de fibre et de textile, dans toute la richesse du terme, auront été introduites au niveau de la conception des ouvrages, pour créer de nouvelles techniques de construction utilisant le textile sous toutes ses formes : fibres, fils, câbles, bandes larges ou étroites, nappes, produits composites, produits façonnés de géométries diverses, structures tubulaires, alvéolaires, fermées ou ouvertes, etc.

Ces nouvelles techniques de construction mettront aussi en jeu de nouveaux procédés d'exécution et de nouvelles machines ; association du textile et du transport hydraulique ou pneumatique des sols, matériels de manipulation et d'assemblage des éléments textiles et bien d'autres dispositifs imaginables.

Certains de ces procédés commencent à apparaître, comme par exemple les soutènements à armature géotextile ; d'autres, de protection contre l'érosion, de renforcement des sols, de drainage, de construction routière ..., seront présentés au cours de ce congrès ; d'autres, enfin, sont pour un avenir qu'il faut souhaiter pas trop lointain ; je pense en particulier à l'habitat en terre, où le mariage sol-fibres naturelles a été utilisé depuis toujours et où les fibres modernes ont certainement un place ; or, n'oublions pas que plus de la moitié des habitants du globe vivent dans des habitations en terre.

Ma conviction profonde est que le développement de ces nouvelles techniques de construction et de travaux publics est le véritable avenir des géotextiles. Mais il faut savoir que c'est une tâche difficile et de longue haleine, aux aspects multiples, et qu'elle exige absolument la collaboration étroite des deux communautés professionnelles, textile et génie civil.

C'est une tâche qui vaut la peine d'être entreprise, car elle apportera de bonnes solutions à d'innombrables problèmes de construction, ce qui est une des clés du développement.

On peut être certain que son accomplissement changera notablement l'art de construire.

Cela se fera si nous avons la volonté de collaborer, l'audace d'innover et le courage de persévérer.

RAUMANN, G.
Monsanto, Decatur, AL, USA

Geotextiles: Construction Materials in Evolution

"Les Géotextiles: un matériau de construction en évolution"

INTRODUCTION

It is with great pleasure and deep appreciation of the honor of having been invited, that I address this large, international gathering of so many of the leading persons in the field of geotextiles.

The purpose of this talk is to review briefly the status and developments in geotextiles from the textile viewpoint. M. Leflaive presented some engineering aspects and I will add my personal view as a physicist who came into this field via the textile rather than the civil engineering discipline.

In thinking about the subject the analogy with biological evolution came to mind. From the first few, simple structures, we saw how some fabrics evolved, mutated, branched and specialized, how some remained unchanged and how a few, unfit to survive the selective processes of the market place, became extinct.

I will take the opportunity while tracing the evolution of fabric geometry and structure to describe some of the fabrics, to correlate their structures with some salient properties, and to acquaint those of you new to this subject with some textile concepts, terms and test methods now being discussed and developed.

With the physical changes of the fabrics we can also trace a correlating evolution of our knowledge of how different types of fabrics behave and of our understanding of what is required for specific applications.

Finally, I will include some selected data of the growth and diversity of the geotextile market and the textile industry and to trace the evolutionary process in that respect.

FABRIC EVOLUTION

Various claims have been made as to the first use of a geotextile. Isolated cases of fabrics in geotechnical use have been recorded in 1926, or possibly earlier, but I believe that the techniques and the technology which we are met here to discuss originated with the geotechnical fabrics installed in Europe about 25 years ago. There are reports of the use of woven polypropylene fabrics as filtration layers in coastal works in Europe and in a few U.S. installations in the late 1950's, but the true impetus occurred in Europe in the mid 60's with the introduction of nonwoven fabrics. By the late 1960's two nonwoven fabrics: one a needled polyester the other a heatbonded fabric based on polypropylene were introduced for geotechnical use. They were found to facilitate construction in adverse conditions, to realize cost savings and to open possibilities of novel civil engineering techniques. Progress was rapid in Europe with applications in stabilization, in roads and embankments, and coastal erosion protection. The new nonwoven technology and the fabrics were introduced to America in the early 1970's.

These early geotextiles were adapted with minimal changes from industrial fabrics used as carpet backing, tarpaulins, shade cloths and similar applications. Over the last 20 years many important changes in fiber type and fabric construction have taken place. Some were low key as manufacturers realized the desirability of a different combination of properties. Some quite major changes in fiber geometry or blend, or fabric construction occurred without even a change in fabric designation. Other changes were accompanied by new designations, or heralded by new names and great advertising fanfare.

What were some of the changes? The experience of the U.S. market can serve as an illustration: a typical early geotextile was a woven polypropylene fabric made of roughly rectangular cross section monofils. (This term, which is an abbreviated form of "monofilaments", describes coarse filaments which are processed as single entities.) They typically had aspect ratios (width:thickness) of 3:1 and were black because of the added carbon black for good U.V. protection. They had good tensile strength, modulus and about 20% elongation at break in the directions of the two orthogonal sets of filaments - in other directions the mechanical responses could be very different.

They were produced with well controlled fiber spacings and uniform openings that could be compared to established sieve sizes. They were costly to produce due to low weaving speeds, fairly narrow widths. They had a tendency to fray cut (but unsealed) edges and showed a high bending stiffness due to the thick monofils, which, when made to conform to uneven ground could give rise to stress concentrations (and possible tearing) or shear (with attendant change in size of openings). They are shown in Figure 1.

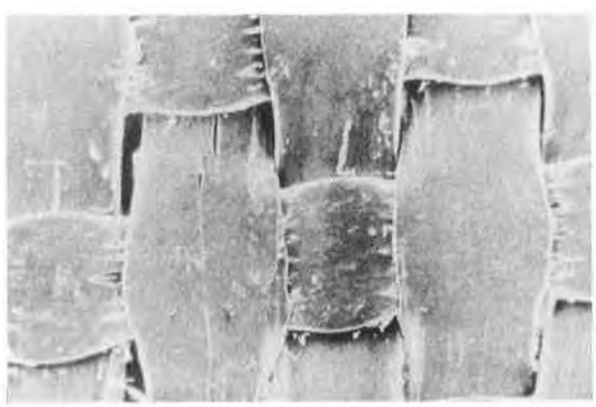

Fig. 1. Photomicrograph of a woven monofil geotextile.

A newer generation of woven polypropylene fabrics was introduced using slit film ribbons which can be produced more economically and woven on faster, wider looms. Superficially they look similar to the monofil structures and many are also black and of similar construction and weave (i.e. ends per metre). The tensile strength, elongation at break and modulus were similar to monofils for fabrics of the same area density but they differed in several important respects: they were lower in price and had a lower bending stiffness since the width to thickness aspect ratios for the slit ribbons were typically 20:1. The ribbons often showed irregular folding or twisting patterns and the fabrics had areas with fairly large average hole sizes and other regions with small, tortuous passages and the comparison to a sieve was no longer appropriate. (See Figure 2.)

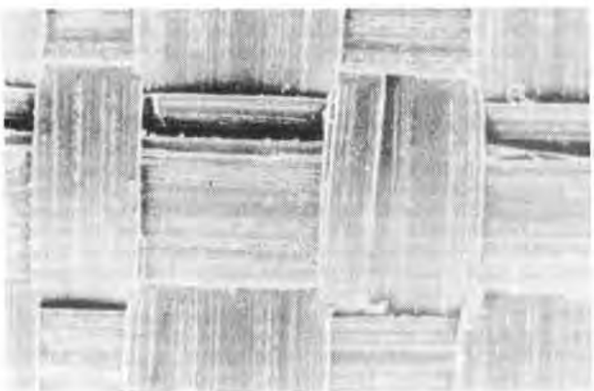

Fig. 2. Photomicrograph of a woven slit film geotextile.

The introduction of slit film fabrics also required alternative methods of light stabilization for adequate outdoor performance. Because of the increased surface area the carbon black, found effective in monofils, no longer gave the same degree of protection. Other, more costly compounds had to be added or, for applications where outdoor exposure is minor, the fabrics were marketed without carbon black and were normally buff in color.

The two types of woven fabrics just described are still the predominant types but woven fabrics are also available which are made from multifilament yarns or twisted strands of slit films and extend over a wide range of construction parameters. The most commonly used polymer is polypropylene, but nylon, polyester, polyvinylchloride and polyethylene and some natural fibers are also used.

Woven geotextiles are often the materials of choice in applications where high strength and high modulus in the principal directions are more important considerations than similar properties in all directions, and where ease of handling and installation on uneven terrain are not critical. For filtration some engineers also prefer to specify on the basis of equivalent opening size in woven fabrics since regular, visible holes can be related to the soil particle sizes to be retained. The major uses for woven geotextiles are in coastal works, waterways, embankments and in or near dams. Woven fabrics of special construction are used in concrete forming.

While the woven geotextiles were diversifying, the development in nonwoven fabrics was even greater. Figure 3 shows some of the stages of manufacture of a continuous filament nonwoven fabric. The molten polymer is forced through fine holes in a spinnerette to make the fibers which, after cooling and stretching to impart strength, are more or less randomly laid onto a moving belt. This random web is then bonded. Spunbonded* fabrics are produced by chemical or heatbonding, normally with the use of calendars: heavy rollers that help to produce good bonds and smooth, thin fabrics. A different method of bonding uses a physical entanglement of fibers by means of hooked needles to produce needled fabrics with open, bulky structures. This method of bonding can also be used on mats of staple (cut) fibers.

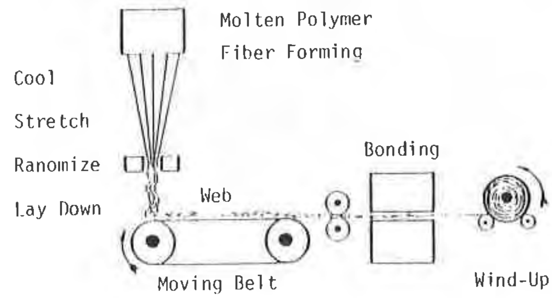

Fig. 3. Diagram of manufacture of nonwoven geotextiles.

The earliest nonwoven geotextiles, a continuous filament needled polyester and a spunbonded* polypropylene-based fabric (Figures 4 & 5), were shortly joined by a needlebonded heattreated staple (cut fibers) polypropylene fabric. These nonwovens, although made of different polymers and by quite different processes, had certain common characteristics - quite different from the woven fabrics.

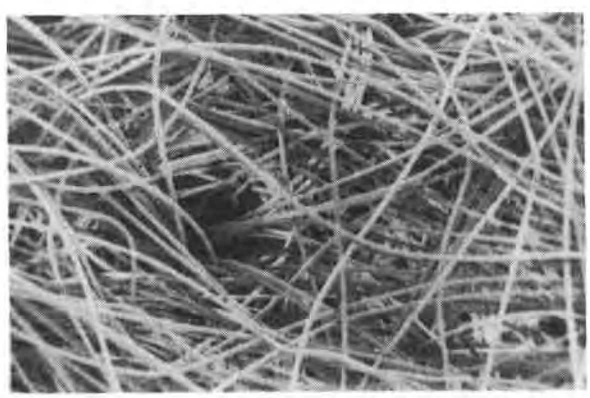

Fig. 4. Photomicrograph of a continuous filament needled nonwoven geotextile. Surface view.

*Footnote: Spunbonded is a term first applied specifically to nonwoven fabrics made of continuous filaments which are heat- or chemically-bonded, primarily at filament crossover points. More recently the term has also been applied to continuous filament needled fabrics.

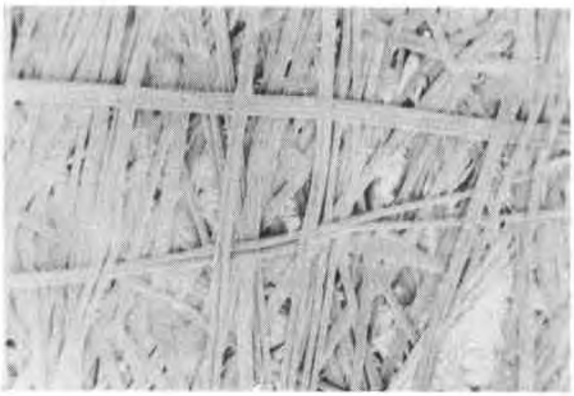

Fig. 5. Photomicrograph of a spunbonded nonwoven geotextile. Surface view.

Some of the differences between woven and nonwoven fabrics are shown in Table 1.

TABLE 1

COMPARISON OF PROPERTIES OF WOVEN AND NONWOVEN FABRICS

(SIMILAR AREA DENSITY)

	Woven	Nonwoven
Fiber Arrangement :	Orthogonal	Random
Properties :	Directional	Nondirectional
Breaking Strength* :	Higher	Lower
Breaking Elongation*:	Lower	Higher
Initial Modulus* :	Higher	Lower
Tear Resistance :	Lower	Higher
Openings :	Can Be Regular	Irregular
Filtration :	Single Layer	Often Multilayer
Porosity :	35% to 45%	55% to 93%
Inplane Flow :	Low	Can be High
Edge :	May Ravel	Does Not Ravel

*Machine Direction

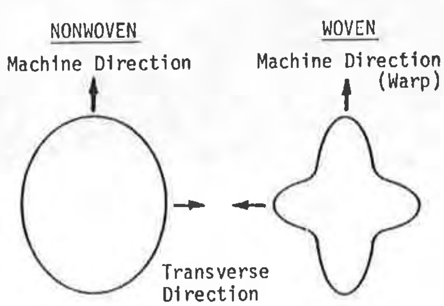

Fig. 6. Polar diagrams of the tensile modulus of nonwoven and woven geotextiles.

Another important difference is shown in Figure 6: This figure illustrates how the mechanical properties of nonwoven geotextiles, because of their essentially random fiber orientation in the fabric plane, are almost independent of direction. If differences exist they show an elliptical not a cruciform polar diagram as is the case for typical woven geotextiles.

Also, nonwovens are made of individual, very thin fibers (rather than monofils, slit film ribbons or twisted yarns as found in woven geotextiles) hence they are typically more flexible and deformable and have a lower modulus and a higher elongation at break. The needled fabrics having no rigid fiber-to-fiber bonds show the least resistance to deformation by localized forces. They can deform (e.g. when required to conform to uneven ground) by rearranging their structure and spreading the localized stresses over a wider area, while when overall restrained (as occurs when they are in position underground) this rearrangement is prevented and they show much greater resistance to deformation on the larger scale.

It was the existence of such fabric behavior which led to the realization that traditional textile test methods could give quite misleading information about the performance of fabrics in geotechnical applications. Figure 7 shows how force elongation curves by conventional fabric tests could give values of one third of the breaking strength and twice the elongation of those obtained in a restrained tensile test. The development of tests to relate to field conditions and to be appropriate in scale for the intended geotechnical use have been the concern of several organizations involved in standards and specification development both in North America and Europe. Reports on the progress of these are the subject of a special session of this conference.

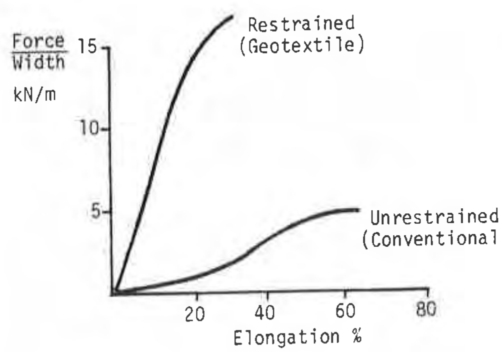

Fig. 7. Effect of restraint on the tensile stress-strain curves of a needled geotextile.

Another property found only in certain nonwoven fabrics is the range of openness of their structure. Depending on how they are produced the openness when expressed as the percent open space (i.e. the porosity) can vary from 93% for thick, needled fabrics (Figure 8) to 55% porosity for the calendared, heatbonded fabrics*. Thus, the thick, highly porous, needled fabrics are very suitable for applications where in-plane water transport of the fabrics is the major or a very important function. Again, special test procedures had to be devel-

*Footnote: At the lower end of this range the term "porosity" may not be appropriate since much of this implied openness is not truly in the structure but rather the unevenness associated with the two surfaces.

oped to measure their permeability--which is of similar magnitude to that of coarse sand.

Returning again to the topic of the evolution of nonwoven geotextiles: In the late 1970's, undoubtedly triggered by the impetus and interest generated at the First International Conference on Geotextiles, the numbers and types of nonwoven geotextiles increased rapidly. Most fabric producers marketed a variety of styles for different end uses. Continuous filament or staple needled polypropylene and polyester of many thicknesses and degrees of needling and several types of spunbonded fabrics were available. Special combinations of thickness, width, strength, porosity and opening size have found application for specific end uses notably in embankments, hydraulic works and dams, in earthwalls, unpaved, flexible and rigid roadways, in railroad track construction and soil stabilization. Bulky nonwovens can act as a dewatering drain and protective layer for geomembranes. In addition, combinations of woven and nonwoven layers have been produced: Nonwovens can be used as a "cap" in combination with slit film wovens to act as filter mat to overcome the non-uniformity of opening size and sometimes to act as a drainage layer or impart better frictional characteristics. Open weave wovens are used as a "scrim" in nonwoven structures for increased strength or modulus in specific directions. The developments in the field of these compound structures underscore the evolution of geotextiles from a "one-size-fits-all" commodity item to specialty products "tailor made" for clearly defined applications.

Fig. 6. Section of a needlebonded geotextile with porosity of 93%.

The fabrics so far discussed comprised variants of traditional textile structures, but even newer structures are being used: The geonets and geogrids shown in Figures 9 and 10 often have orthogonal structures reminiscent of woven fabrics and the bonded crossover points of spunbondeds, the coarse structured geomats can be considered as extensions of needled nonwovens. Products are increasingly being specified where their very open structures or their very large geometrical repeat units confer special drainage or frictional advantages for selected end uses. On the other end of the scale have been experiments of soil reinforced with mats of chopped fibers - a new concept and yet reminiscent of the straw in the bricks of Mesopotamia of 6000 years ago.

Purists among textile practitioners may take exception to designating such structures as textiles at all - but the traditional cut-off point has been established because these newer products were not known at all or only in intermediate stages of manufacture. Technology is constantly advancing and it would not be the first case of traditionalists having to rethink their concepts, redefine their terms, and widen their terminology because of the progress of technology.

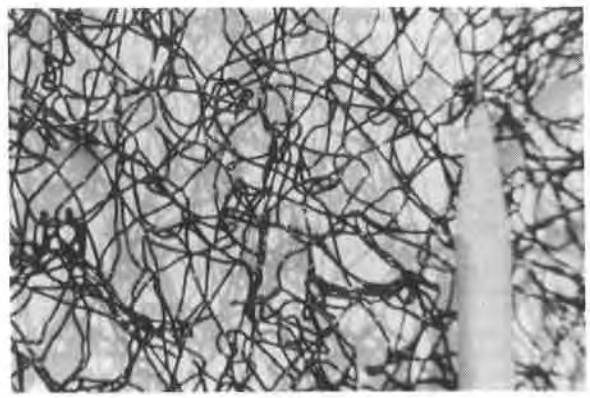

Fig. 9. View of a geonet.

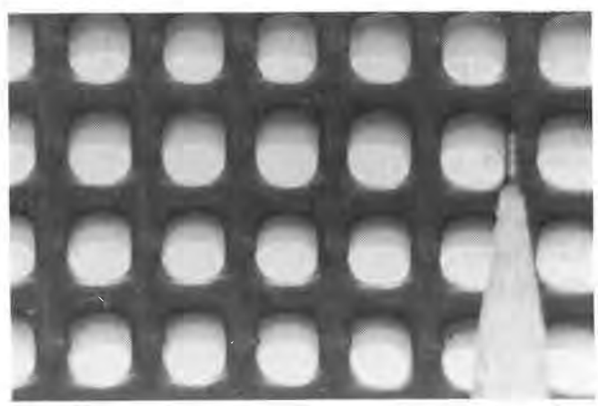

Fig. 10. View of a geogrid.

THE GEOTEXTILE MARKET AND THE TEXTILE INDUSTRY

Using best estimates from a number of sources the 1980 annual world production of about 115 million m^2 geotextiles (which includes some textile based geomembranes) can be broken down by region as shown in Table 2. Europe and North America account for about 90% of the total production, Europe and the U.S. producing, roughly equal areas. Combining the U.S. figures of 45 million m^2 and a use of 8.5 million kg one arrives at an average area density of about 0.2 kg/m^2 - a credible figure.

TABLE 2

GEOTEXTILE PRODUCTION BY REGION (1980)

(Includes some Geomembranes)

Europe	45	million m^2/yr	(40%)
U.S.A.*	45	" "	(40%)
Canada	11	" "	(10%)
Rest of World	12	" "	(10%)
Total	113	" "	(100%)

* The total U.S. production has been estimated by various sources and figures as widely different as 24 to 87 million m^2 have been quoted, hence all the figures shown here must be considered very approximate.

The types of applications for which geotextiles were used in the U.S. in 1980 is given in Table 3. It was prepared from a list of the customers of geotextile suppliers and the final end use was not always clearly identifiable. The largest single item listed, i.e. 26% for asphalt road construction must be understood to include a significant area of geotextiles supplied to Highway Departments which was used in drainage and site stabilization and not for direct application in the roadway. The term Sports and Recreational includes such uses as bicycle, race and running tracks and tennis courts. The 6% used by government departments included coastal and waterway use and forestry roads and staging areas.

TABLE 3

U.S. GEOTEXTILE APPLICATIONS (1980)(BY FABRIC AREA)

Asphalt Road Construction	26%	By Area
Worksite Stabilization	21%	" "
Commercial Forestry	15%	" "
Sports & Recreational	11%	" "
Government	6%	" "
Railroads	6%	" "
Erosion Control & Embankments	5%	" "
Agricultural	5%	" "
Mining, Oil & Gas Exploration	5%	" "
Total	100%	

The analysis of geotextiles by polymer type and method of manufacture is known to a much greater degree of certainty than the data quoted above. As shown in Table 4, 95% of all geotextiles used in 1980 in the U.S. were made from polypropylene: one half are needled products, over one third are spunbonded and about 7% of the total are woven. The remaining 5% are needled polyester fabrics both as continuous filament and in the form of staple fibers. The nonwovens (polypropylene and polyester) have steadily been increasing their share of the geotextile market. In the U.S. in the late 1960's 95% of all geotextiles were woven polypropylene since then that percentage has been steadily declining, although, because of the rapid overall market expansion, the total area of all woven geotextiles has been increasing.

TABLE 4

U.S. GEOTEXTILE USE BY FABRIC TYPE (1980)

		By Weight	By Area
Polypropylene	Needled	54%	52%
"	Spunbonded	32%	36%
"	Woven Monofil	6%	5%
"	Woven Slit Film	1%	2%
Polyester	Needled	7%	5%

Having looked at the geotextile market I would like to take a somewhat wider view - the total textile industry: The importance of the industry in the world today is often overlooked. Textiles may be less noticeable than large engineered structures but they are more widespread. Table 5 compares the world industrial production of some engineering materials and textiles.

TABLE 5

U.S. GEOTEXTILE USE BY

Cement	760	million tons
Steel	670	" "
Asphalt Cement	80	" "
Textiles	30	" "

The world total of 30 million tons (metric) of textiles includes only industrially produced textiles and does not include home spun products. Nearly one third of the world output is produced in Europe and one quarter is produced in the U.S. as shown in Table 6. The 34% not specifically listed by origin consists mainly of cotton fibers not used in geotextiles and produced chiefly by India, Pakistan, China, Egypt, Sudan, and others.

TABLE 6

WORLD PRODUCTION OF TEXTILE FIBERS BY REGION (1980)

U.S.A.	7.2	million tons	(23%)
Eastern Europe	5.7	" "	(18%)
Western Europe	4.3	" "	(14%)
Japan, Taiwan, Korea	3.1	" "	(10%)
All Others	10.6	" "	(34%)
Total	30.9	" "	(100%)

The breakdown into the world's main fiber types (Table 7) shows that synthetics take second place to cotton with somewhat more than one third of world production. They are the only materials sufficiently longlasting to be suitable for most geotextile applications.

TABLE 7

WORLD PRODUCTION OF TEXTILE FIBERS BY TYPE (1980)

Cotton	14.4 million tons	(47%)
Synthetics	11.6 " "	(37%)
Cellulosics	3.4 " "	(11%)
Wool	1.5 " "	(5%)
Total	30.9 " "	(100%)

Taking a closer look at the U.S. market we see that all industrial fabrics account for somewhat under one quarter and the proportion of this figure that goes into geotextiles at this time is less than 1% (Table 8). With such a small segment why is the textile industry so clearly interested in geotextiles? The reason is shown in Figure 11 where (on a logarithmic scale) an annual growth rate over the last few years is extrapolated to a conservative 20% per annum for the next 5 to 10 years.

TABLE 8

U.S. TEXTILE FIBER END USE (1980)

Apparel	2.3 million tons	(42%)
Furnishings	1.7 " "	(31%)
Industrial	1.3 " "	(23%)
Other	0.2 " "	(4%)
Total	5.5 " "	(100%)
Geotextiles	8.5 million kg	

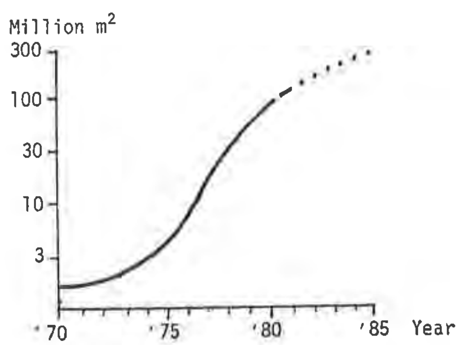

Fig. 11. Use of Geotextiles in the U.S. since 1970 (Logarithmic scale).

With this growth rate continuing, at the end of the next 5 years geotextiles in the U.S. will have penetrated a mere 1% to 2% of the total U.S. uses for which geotextiles are known to be applicable. Additional market growth can be projected for the following 20 years or so. These penetration estimates are based on presently known uses and do not allow for completely new applications, exports, or quite new uses of fibers as discussed by M. Leflaive. The above growth rates are far greater than the average growth rate in the industry where even the fastest expanding sector, the synthetic fibers, are showing less than 5% per annum growth. We see here an industry with excess production capacity available and well equipped with the highly sophisticated technologies of modern synthetic fiber and plastics plants. They are geared to take on new challenges and expand into new markets. It is for this reason that we see why the textile industry is courting the geotechnical community as is evidenced by the many exhibits that you will be able to visit during the course of this week.

CONCLUSION

We have seen how in the short span of 25 years geotextiles have seen rapid diversification of fabrics and good a progress in the appreciation of how fabric properties affect their performance in geotechnical applications. Early empirical and intuitive use is being replaced by proven working theories and sound design formulas. We are still far from the desirable state where, starting from a design analysis of system stresses, hydraulic and frictional requirements, we can compute expected loads, deformations, and flows in the various direction in fabrics - and produce a tailor-made fabric for that end use. As we can see from the contributions to this conference we are moving in that direction. Alas, geotextile technology still awaits the unifying theories of a Euclid, Pasteur or Newton - but there is a good chance that very person may be in this room right now.

From the viewpoint of the long established textile industry geotextiles are challenging newcomers opening opportunities of expansion and potential for development of new technology, new fabrics and new concepts. The practitioners in the textile industry share a feeling of optimism and excitement with the civil and geotechnical engineers as we embark together on a journey of discovery through this week of conference meetings and trade exhibits and on to the developments of the years ahead.

Conference Presented Papers: Questions and Responses

CEDERGREN, H. R.
Consulting Engineer, Sacramento, California, USA

Overcoming Psychological Hang-Ups is Biggest Drainage Challenge
Vaincre les préjugés est le problème prioritaire du drainage

Vol. 1, p. 1

Question:

In the case history you cited:

1) Did you consider a geotextile on top of the sand/gravel drainage layer instead of a conventional soil filter?

2) Did you consider using a plastic mesh (products like Enkadrain or Filtran) instead of the sand/gravel drainage layer?

3) What do you think of the future of those composite systems?

Answer by H. R. Cedergren:

1) A geotextile was not considered on top of the open-graded drainage layer for the downstream blanket drain for this earth dam in a California Sierra-Nevada foothill area because a cover layer of well-graded sand and gravel was needed: (a) to protect the drainage layer from erosion by surface water runoff, and (b) to keep mud and other soil fines from entering the drainage layer from the top and clogging this layer.

   A geotextile was placed on the ground surface prepared for the drainage blanket, to serve as a separator to prevent mixing of the fine subgrade soil with the sand filter or the open-graded drainage layer.

2) We did not consider using a plastic mesh to provide the conducting components of this drainage system, because the water-conducting capabilities of the plastic materials would have been too small to remove the calculated seepage quantities (as estimated with Darcy's law, flow nets, etc.) without excessive head build-up in the system. As constructed, this system contained a 12 in. (30 cm) thick layer of good quality, washed aggregate in the size range of 1 in. (2.5 cm) to 1/4 in. (0.6 cm) with a coefficient of permeability in the order of 20,000 to 50,000 ft/day (7 cm/sec to 18 cm/sec). Under rather severe conditions it has performed extremely well.

3) I feel that composite systems that make use of a designed conducting layer of open-graded mineral aggregate (such as from 3/4 in. (1.9 cm) or 1 in. (2.5 cm) maximum size to 1/4 in. (0.6 cm) minimum size), enclosed within protective filters constructed with carefully selected and designed plastic filters have a great future. Such systems offer a way to make use of the good filtering properties of properly selected plastic filters and the unbeatable water-conducting properties of good-quality mineral aggregates containing no fines (no material smaller than about 1/4 in. (0.6 cm)). At the present state of the art, systems composed entirely of plastic materials should not be promoted in situations where appreciable amounts of water need to be removed, unless seepage calculations demonstrate that they have the needed discharge capabilities over the expected life of a project.

Vol. 1, p. 1

Question:

Have you ever had clogging problems with geotextiles? What were the conditions?

Answer by H. R. Cedergren:

I have personally been involved with only one project where there were problems with clogging of geotextiles. This was with an edge drain for an airport pavement in the S.E. part of the United States, where an aggregate drain was enveloped within a fabric. Practically no water came out of the drain--only a small dribble. This project is being investigated to determine the cause of the problem which is suspected to be either from (a) the use of a concrete-sand type of drainage material whose low permeability blocked the flow of water to the pipes, or (b) clogging of the enveloping fabric because of pulsating ground water levels around the drain (which was installed in fine sandy silt).

Question:

Which kind of geotextile (woven or nonwoven) is better for drainage?

Answer by H. R. Cedergren:

There are good woven products and good nonwoven products. In selecting a type for a project, test results on the geotextiles being considered should meet the requirements of the use of the product in a specific installation. Where I have recommended the use of geotextiles, the owners of projects have used materials with a good track record.

Question:

Re: Opening Sizes O_{90} O_{95} O_{98} etc.

When determining this parameter what method of measurement is used,((1) Ballotine Ball Method, (2) Micro photography and Frequency Distributions, etc.) to comply with the filter criteria mentioned.

Apparently opening size values vary according to the method used.

Answer by H. R. Cedergren:

While I am familiar with most of the methods used, I do not get involved with these testing methods and rely on experts who specialize in testing and evaluating fabrics, such as the U.S. Army's Waterways Experiment Station in Vicksburg, Miss., U.S.A. Where geotextiles are to be used in drainage systems I am always concerned about adequate levels of permeabilities remaining under the pressures and other conditions that will exist in the field. Also, that opening sizes will be consistently small enough to prevent harmful movement of soils into or through the systems.

CANCELLI, A.
Department of Structural Engineering, Technical University, Milan, Italy

Performance of Geotextiles in Stabilization of Clay Slopes in Italy
Comportement des géotextiles dans la stabilization de pentes argileuses en Italie

Vol. 1, p. 7

Question:

Was there any piezometric measurement made to ascertain that the trench was really effective? (The fact that no movement was observed could be related to the scarceness of precipitations.)

Answer by A. Cancelli:

Piezometers were installed while studying the landslide and piezometric measurements were made before and during the construction of the trench. Unfortunately, all the piezometers were destroyed after the construction by an operating machine, because, as you well know, very often slope stability works interact with agricultural works. However, we are sure that the movement was stopped, because the area was subject to high rainy periods during winters 1979/80 and 1980/81 (on the contrary, only the last winter was a dry season). Of course, I think that the observation times should be not only two or three years, but at least ten years long, in order to assess the good results of slope stabilization works.

Question:

The variation of production usually is maximum 10%. How can the decrease of 20% of strength after only 30 months be explained, especially because there did not occur high forces attacking the geotextile?

Answer by A. Cancelli:

In this particular case, I think that a so high variation could be ascribed to a newly installed production plant. However, it's my opinion that geotextiles are employed in these works not for their strength but for their permeability, because the draining trench and the geotextile itself have the main scope to lower the water pressure and not to resist to tear efforts. When you lower water pressures in a slope, you stop the movement and the strength of the geotextile is not so important.

Question:

Considering the depth at which the geotextile was used, was any consideration given to variation in pore size and permeability under these normal loads?

Answer by A. Cancelli:

The geotextile was used at a maximum depth of 6 m, corresponding to a vertical normal pressure of about 100 kPa. As it can be seen in fig. 7 of the paper, the variation in permeability (both planar and normal) under such pressure is relatively low: the decrease was considered to be tolerable with reference to the amount of water that should be drained away from a clayey soil.

COLLINS, T. G. and NEWKIRK, D. D.
Crown Zellerbach Corporation, Camas, Washington, USA

The Use of Geotextile Fabrics in Pond Construction Beneath an Impermeable Membrane (Geomembrane)

Utilisation de géotextiles sous une membrane imperméable dans la construction des bassins

Vol. 1, p. 13

Question:

Are there any essays/results relating to acid or basic waters attacks to the geotextile?

Answer by T. G. Collins:

There are some preliminary data being developed by the Chemical Resistance Group of the A.S.T.M. Test Methods Development Section. The geotextile that we used was needle punched, nonwoven, polypropylene and it has a Ph resistant range from 1 to 13 over relatively long use periods.

Question:

Are there any data indicating life expectancy of the membranes when used in gaseous or corrossive environments?

Answer by T. G. Collins:

Dr. Giroud cited several examples of ponds constructed in Europe using geotextiles that are now 10 years old. Because the contained fluids can vary so much, each fluid must be evaluated separately and life expectancy derived from laboratory analysis.

Question:

With the double fabric above and below the membrane, were there different properties or characteristics between the fabric below the membrane and the fabric exposed to the pond liquid?

Answer by T. G. Collins:

No, in the cases cited, the geotextile fabric was needle punched, nonwoven polypropylene fabric in both layers. But there are times when the contained fluid in a pond would require that the upper geotextile, in contact with the fluid, be comprised of a different, chemically resistant polymer.

Vol. 1, p. 13

Question:

Since most Geomembranes evaluated appeared to be unsupported films, is there a need or place for coated fabrics as Geomembranes?

Answer by T. G. Collins:

There are several coated fabrics currently being used as Geomembranes. I would think that this trend would continue as the market expands and strength requirements become more critical.

Question:

--Estimation du marché des reservoirs en 1982 aux etats-unis (marché annuel)? (En 7^2)

--Part des géotextiles dans de marché?

--Proeression des géotextiles à prevoir dans de marché aux USA?

Answer by T. G. Collins:

I do not know the market figures for Geomembranes but the 1982 U.S. market for geotextiles used in conjunction with Geomembranes was approximately 1 million square yards.

PUIG, J.
Laboratoire Régional Ponts et Chaussées, Toulouse, France
PRUDON, R.
SODOCA Neuf Brisach, Courbevoie, France

The Use of Geotextile for Wrapping Large Depth Drain

Utilisation d'un géotextile en tranchée drainante de profondeur importante

Vol 1, p. 19

Question:

Could you expand on the reasons for your filter equations 2 and 3? In particular the significance of the O_{95}.

Answer by J. Puig:

Le choix d'un filtre est fonction de la granulometrie du sol d'ou vient l'écoulement. On compare la porométrie du textile exprimée par son diamètre de filtration (O_{95}) à la granulométrie du sol exprimée par son B 85 dans le cas des sols à granulométrie continue et à son B 50 dans le cas de sols à granulométrie étroite.

(O_{95}) est par définition le diamètre tel que 95% des pores du géotextile aient un diamètre inférieur a O_{95} et 5% un diamètre supérieur.

HUNT, J. A.
ICI Fibres, U.K.

The Development of Fin Drains for Structure Drainage
Le développement des drains en épi pour le drainage des ouvrages

Vol. 1, p. 25

Question:

What is the performance of pre-fabricated drains in non-vertical situations?

Answer by J. A. Hunt:

The flow of water in a non-vertical situation is a function of fin drain capacity and gradient. Even in near horizontal situations a fin drain can perform satisfactorily, providing adequate outlets are available to cope with a drain at optimum capacity. On flat continuous gradients any undulations will reduce or limit flow unless anticipated pore pressures can overcome the restriction.

Although my personal experience on near horizontal installations is limited, there is considerable interest. No problems are anticipated, provided the above limitations are noted.

GIROUD, J. P.
Woodward-Clyde Consultants, Chicago, Illinois, USA

Design of Geotextiles Associated with Geomembranes
Dimensionnement des géotextiles associés à des géomembranes

Vol. 1, p. 37

Question:

What geotextiles are used in conjunction with separating nets? What significance has the modulus of the fabric in avoiding the fabric being pushed into the net?

Answer by J. P. Giroud:

Selection of geotextiles to be used in conjunction with plastic nets should be done considering: (i) the function of the geotextile; and (ii) the earth pressure pushing the geotextile into the net. From that latter viewpoint, a stiff geotextile is preferable.

Question:

Dr. Giroud, when you discussed the transmissivity of geotextiles, you mentioned that one needs to increase the thickness or the number of layers of geotextiles as the compressive stress increases due to the lack of resistance of the geotextile to compression.

Can we not also select structures other than nonwovens (or modified nonwoven structures) combined with a suitable fiber of higher modulus to achieve the level of required compression resistance without increasing the number of layers of fabrics?

Answer by J. P. Giroud:

Of course, the less compressible the geotextile is, the less necessary it is to multiply the number of layers. For example, a new type of geotextiles, the plastic nets, usually have a very low compressibility.

Question:

Vous avez montré des applications de géotextiles comme protection de geomembranes. Cela me parait une possibilité intéressante; pouvez vous préciser les avantages que les géotextiles ont par rapport a d'autres type de protections de geomembranes?

Answer by J. P. Giroud:

Traditionally, a layer of sand is used to protect geomembranes. Sand has many drawbacks: (i) sand may be eroded by wind or run-off water before installation of the geomembrane; (ii) sand may be eroded by running water after the installation of the geomembrane; (iii) sand may be lost in a gravel drain due to gravity or running water before or after installation of the geomembrane; (iv) wave action on the geomembrane may displace the sand under the geomembrane; (v) wave action may wash away the sand placed on the geomembrane to protect it from a cover made of stones, concrete blocks, or concrete slabs; (vi) sand is not stable on steep slopes. Obviously, geotextiles do not have such drawbacks.

FAYOUX, D.
Cemagref, Antony, France
EVON, E.
Sommer, Sedan, France

Influence of the Fiber Size on the Filtration Characteristics of Needled-Punched Geotextiles

Influence de la fibrométrie sur les caractérestiques de filtration des géotextiles aiguilletés

Vol. 1, p. 49

Question:

Did you find that different manufacturing methods (i.e., double punching, single needling) have great influence on the performance of needled felts in geotextiles?

Answer by D. Fayoux

Nous n'avons pas encore fait d'essai pour tester l'effet du mode de fabrication (en particulier aiguilletage simple et double) sur les caractéristiques de filtration. Cependant, dans l'étude qui est présentée, on constate que les caractéristiques de filtration sont fonction non seulement de la masse surfacique de l'épaisseur, de l'indice des vides et du diamètre des fibres, mais aussi de la structure.

Le mode de fabrication qui conditionne cette structure, a donc une influence sur les caractéristiques de filtration.

On peut s'attendre, d'après nos résultats, à des écarts de l'ordre de 30% sur les diamètres de filtration, mais ceci doit être cependant vérifié par des essais.

Question:

Needle punched fabrics are not random. All speakers have assumed they are. Has any one used some of the techniques which are available to measure the structural parameters rather than merely looking at Wt/m^2 and fiber diameter?

Answer by D. Fayoux:

L'examen sur lame mince permet de mesurer la répartition des fibres parallèles à la nappe textile, pour laquelle une répartition au hasard semble donner des résultats satisfaisants (cf. travaux de ROLLIN). Par contre, la repártition des fibres perpendiculaires à la nappe, au niveau des points d'aiguilletage, semble beaucoup plus difficile à mesurer. Or nos essais montrent que celle-ci a une influence directe sur la répartition des fibres horizontales et, donc, les caractéristiques de filtration, ceci, pour des valeurs données de μ et de c.

Je pense, sous réserves d'essais complémentaires, qu'une mesure indirecte de cette structure peut être donnée par la valeur du coefficient A, dans la représentation $D_f + d_\varepsilon = A\ V$ avec $V = \sqrt{1/L}$, en microns.

RAUMANN, G.
Monsanto, Decatur, Alabama, U.S.A.

Inplane Permeability of Compressed Geotextiles

La perméabilité dans le plan des géotextiles comprimés

Vol. 1, p. 55

Question:

Had there been any influence of materials with different permeabilities in production direction and cross direction to your permeability test?

Answer by G. Raumann:

Our tests use a ring shaped specimen and the total amount of water transmitted was measured without attempting to measure the contribution from flow in different directions. Hence our tests measure the average permeability in the plane of the fabric.

Other investigators and some preliminary tests by us using linear flows have shown small differences of planar permeability with direction. In our case the differences were too small to be considered significant.

Question:

Under high pressure kept const. over a long time have you observed some creep? If the creep would happen, the thickness of geotextile should change and so the permeability coefficient?

Answer by G. Raumann:

We concluded some tests on the effect of prolonged pressure on the permeability of geotextiles. We noted thickness and permeability decrease with time. The permeability decreased with fabric thickness as indicated by the porosity function. If the compressive creep of the (dry) fabric is known then the change of permeability with prolonged compression can be calculated from the given formula, thus avoiding long term permeability tests.

Vol. 1, p. 55

Question:

What about the repeatability of your testing method? The dimension of the specimen (difference in radii) seems to me to be quite small compared with the natural heterogeneity of geotextile. In Liege we have been working with radii of 25 and 5 cm.

Answer by G. Raumann:

The points shown on the diagrams in my paper are the averages of at least three different specimens and the agreements were very satisfactory. I agree with M. Rigo that larger specimens might be preferable but this procedure has the disadvantage of the weight (minimum compression) of the apparatus and the much larger volumes of deaired water required. We did consider larger scale testing but decided on the basis of our good repeatability not to change. One advantage of larger specimens would be that the permeated water could be collected in separate collectors corresponding to machine and transverse direction flow.

Question:

What about the boundary condition (sheet-geotextile)? Rilem 47SM proposes an interface of rubber to penetrate the geotextile thickness.

Answer by G. Raumann:

The boundary between our fabrics and the apparatus was a very thin pliable sheet of soft natural rubber (thickness 0.5 mm) and this worked well with all of the thicker fabrics (which are the only ones realistically suitable for in-plane water transport in geotextile applications). The thin nonwoven and woven fabrics were included in our study to determine the limits of the experimental technique and applicability of the permeability theories which we investigated.

RYCROFT, D.
University of Southampton, Southampton, U.K.
DENNIS JONES, P.
ICI Fibres, U.K.

Geotextile Filtration Performance and Current Filter Criteria

Les performances de filtration des géotextiles et les critères de filtre courants

Vol. 1, p. 67

Question:

Inaccuracies in the measurement of woven geotextiles thickness will be reflected in the values of min. (e.g., Figure F). Would it not be better to characterize the permeability of the geotextile by, e.g., head loss per unit discharge?

Answer by D. W. Rycroft and C. P. Dennis Jones:

It is agreed that in general, the hydraulic conductivity of the geotextile is a better characterization of the probable head losses during flow through the fabric. However, the "α" value reflects the characteristics of the geotextile used as a surround to a particular flow system, in this case a slotted pipe. Expressed in dimensionless form, it enables the probable head losses to be deduced, for any known flow rate and know hydraulic conductivity (K values) of soil, etc., surrounding the pipe. A knowledge of the geotextile's hydraulic conductivity would be insufficient in this situation since it would require additional assumptions about the nature of the flow, e.g., radial, and the area of flow involved which, in the case of a pipe, might just be the area above the slots.

The thickness of the fabric would not enter into a measurement of head loss/unit discharge or for that matter "α.". It would, however, enter into calculations of the K value and might make using K less relevant in practical situations.

Question:

What effect does the amount of air in the water supply have on your long term permeability experiments?

Answer by D. W. Rycroft and C. P. Dennis Jones:

Several authorative researchers* have pointed to the presence of air as a major cause of inaccuracy in soil permeability testing.

We have observed that pockets of air can become entrapped in the soil system and so alter the flow geometry. Although flow rates eventually become steady, severe changes of flow rate can result if the bubbles are mobilized, e.g., by increased headloss.

Secondary changes due to differential consolidation because of the entrapped air is also considered to be a factor influencing permeability. Present thinking is to use de-aerated water in these tests. Recently a system to provide water at a controlled temperature and oxygen content has been installed. It is, however, too early to draw any conclusions from this work.

*Allison. L E 1947. Effect of microorganisms on permeability of soil under prolonged submergence. Soil Sci. 63:439-450

Reeve, R C 1957. The measurement of permeability in the laboratory. Agron. 7:414-419

WITTMANN, L.
Consultant, Karlsruhe, Federal Republic of Germany

Soil Filtration Phenomena of Geotextiles
Phénomènes de filtration des sols par des géotextiles

Vol. 1, p. 79

Questions:

1) Could author please clarify the term filtration length?

2) Does the author prefer thick geotextiles or thin geotextiles?

Answer by L. Wittmann:

1) Filtration length takes into account the thickness of filter media. In a dimensionless form the real thickness is divided by the representative element of the filter (grain, fiber etc.) and thus leads to
$m = L_F/D$ resp. $m^* = h/d$
as can be seen in the paper (p.80 and 81).

2) As thickness of geotextiles is depending on the applied surcharge and on the type of soil (fine or coarse grained), the definition of thin or thick geotextiles has to be given first. In general, the preference of one of these types depends very much on the special application (filter, drain, separation etc.) and on the boundary conditions. It cannot be answered in general.

Question:

Partant du fait que les géotextiles ont souvent a filtrer des sols a granulometrie souvent étalée (a coefficient d'uniformité superieur), les auteurs ont ils des commentaires a apporter sur la dimension representative des grains du sol a proteger, a prendre en compte dans les criteres de filtre pour tenir compte de l'auto-filtration (self-healing)?

Answer by L. Wittmann:

Up to now the scope of grains (grain-size-distribution) is characterized by the coefficient of uniformity c_u. Constant grain-size-ratios (f.e. D_{15}/d_{85}), as being used in standard filter criteria, are not valid in general. It can be demonstrated, that with increasing coefficient c_u the amount of particles being bigger than existing pores or openings, has to increase too, to establish the observed self-filtering. This can be seen in filter criteria for granular filters by CISTIN/ZIEMS or in the extension of this criteria to geotextiles by SCHOBER/TEINDL (see session 4A, Drainage III, paper of J.P. Giroud).

Question:

Can you define filtration length of woven geotextiles?

Answer by L. Wittmann:

The general definition of filtration length for geotextiles is $m^* = h/d_f$ with h=thickness of geotextile, d_f= fiber diameter. Thus simple woven geotextiles show a filtration length of $m^*=1$, which means a poor sieve-filtering effect.

Question:

As the parameter $m = L_f/D$ is dimensionless, it is misleading to call it "filtration-length", which implies a length dimension. I suggest that another name be given to the factor m, for example "effective filtration ratio".

Answer by L. Wittmann:

From a physical point of view I agree with this proposal, but as up to now the influence of thickness of filter media was not taken into account quantitatively, this "dimensionless filtration length" has been chosen, to point out the phenomenon.

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Laboratory Studies on Long-Term Drainage Capability of Geotextiles

Etudes de laboratoires sur la capacité d'écoulement de long terme de géotextiles

Vol. 1, p. 91

Question:

What effect does previous wetting/saturation/de-airing of the fabric and soil prior to the commencement of the water flow in the permeameter test make to your results?

In particular are you sure that air bubbles are not contributing to the initial changes in permeability in the first few days?

Answer by R. M. Koerner:

Prewetting of fabrics prior to hydaulic tests does not measurably affect performance for woven or needled nonwoven fabrics. For some heat set spunbonded nonwovens it does have an effect. See, for example, the 1980 INDA Proceedings paper which we presented for break-in times for various fabrics and its influence on hydraulic conductivity.

Air bubbles in the soil during the initial portion of the test is a genuine matter of concern. It is even more so with our configuration which results in downward water flow and the distinct possibility of entrapped air in the soil or fabric for a considerable time. Hopefully, the relatively high hydraulic gradients (about 4) and the uniformity of test procedures kept this unassessed variable relatively constant.

Question:

It was mentioned in the paper that no deaired water has been used. To what extent is flow reduction due to particle intrusion in geotextile and air bubble intrusion? Do you have any information to be given on the pressure distribution?

Answer by R. M. Koerner:

The major thurst of long term testing is to determine if, and to what extent, is particle intrusion (called clogging in the paper) a problem in using geotextiles as drainage and/or filter materials. Whenever the long term slope of the drainage vs. time curve is large such an adverse phenomenona is occurring. Visual observations of embedded particles clearly indicated this situation. Air bubble intrusion was possible, but hopefully, uniformity of test procedure kept this variable constant and relative differences in behavior should be valid. It is felt to be of minor significance for the long term tests as performed but should be confirmed before more definite statements can be made.

Pressure distributions were monitored and are reported in the written text. These pressure distributions were used in calculating gradient ratios as in the Haliburton paper in these Proceedings.

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Question:

If I understood you right you did not use deaired water for the experiments. Isn't deaired water more up to the situation in the drainage trench and doesn't use of deaired water also give better test results in the laboratory for such experiments?

Answer by R. M. Koerner:

Deaired water would certainly result in as high, or higher, flow rates but the relative differences between tests should be representative as presented in the paper. Whether deaired water occurs naturally, under low hydraulic gradients typical of highway underdrains, is somewhat doubtful. As the head increases the air goes into solution in the water, but this is only at considerable depths, e.g., as in consolidation of deep fine grained soil strata.

Question:

Some recently completed laboratory studies, indicated a potential permeability loss due to growth of anaerobic bacteria. These tests were conducted using sterilized materials over a relatively long period, 6 weeks or more as I recalled. The question is in three parts.

1. Did your long term tests show any such bacterial action? If so, do you feel this is a significant problem?

2. Have you seen any evidence of this type of action in the field on either fabric or aggregate filters?

Answer by R. M. Koerner:

1. Only on one soil tested, a river transported clayey silt with a high organic content, did any bacterial action evidence itself. Out of curiosity, the test was continued but no change in flow rate was observed.

   Bacterial growth can certainly be a problem in long term tests such as these. Short of sterilizing the materials, one can wrap the plastic container with aluminum foil keeping the soil/water system in drakness during the test.

2. In the field this type of growth has been observed, but its consequences have not been assessed to my knowledge.

Question:

The speaker has referred to the relation between flow and time is linear. This is only when the relation was based on log-time. So accordingly it is as follows:

$f = e^{kt}$ which is an exponential function. When taking the log it is a linear relation. I wonder if the speaker can comment on that?

Answer by R. M. Koerner:

This type of exponential function is precisely the observed behavior of the test results, i.e., piecewise linear on semi-log paper. If an empirical relationship were desired it would be of the form suggested. In the absence of considerably more testing, however, it seems to be a bit presumptuous to generate such "hard" information. Statistical reliability appears to be the next step.

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Question:

What was the level of agreement between the laboratory tests you presented and the larger models you have used?

Answer by R. M. Koerner:

The general form of the response curves was the same in both these lab tests and the large model tests. However, the magnitudes were quite different. Considering that these large model tests (about 1 cu. yd. size) are so difficult to set up we have gone entirely to the small scale laboratory tests as presented here.

Question:

What kind of behaviour of flow do you expect for an upflow (upstream flow)? Would any reverse flow condition affect any long-term drainage capabilities?

Answer by R. M. Koerner:

We are currently working on alternating flow tests. In these tests we change the direction of the flow every 12 hours (simulating a tidal situation). Data is unavailable at this time but we anticipate higher flow rates for equivalent situations and a breakdown of some of the structural aspects presented in the paper for one way, steady state flow. Alternating flow is a very important aspect of geotextile drainage and filtering mechanisms and one which requires investigation.

Question:

How is the filter cloth supported and fixed in the tubes?

Answer by R. M. Koerner:

The filter cloth is supported between two mated flanges and can easily support the weight of soil and water above it.

Question:

This seems a reasonable test for field labs and personnel on a job site. Do you think suitable and reliable results could be obtained with generally untrained personnel under supervision?

Answer by R. M. Koerner:

The test is readily available for field adaptation and could easily be performed by personnel with a minimum of technical ability. As the question suggests, data interpretation requires insight into the phenomena involved.

Question:

Do you think similar results could be obtained using falling head rather than constant head? That is, prepare a standpipe, fill with water, and read as it drains.

Answer by R. M. Koerner:

Using a falling head test, I would expect similar trends but probably different magnitudes. Thus the test still could be used for overall fabric vs. soil compatibility.

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Question:

For long term soil drainage: There may not be an ideal fabric for all soil, as you reported, but can you rank order the following fabrics for acceptability in most soil drainage applications:

Woven fabric - open mesh

Woven slit film

Nonwoven needle punched

Nonwoven heat bonded

Answer by R. M. Koerner:

Based on our tests to date, there is not an ideal fabric for all soils. As far as ranking the fabrics on the basis of its construction or manufacture, I feel very uncomfortable. The thrust of the paper was to take the soil from the actual field site, select a number of candidate geotextiles and visually observe which produces the highest equilibrium long term flow rates. If a number of fabrics are adequate, the final decision becomes one of cost and availability.

Question:

Could any of the authors comment on the validity of filter criteria when unsaturated flow conditions occur, an example of a fine grained soil above a geotextile wrapper coarse filter drain that accepts surface water.

Answer by R. M. Koerner:

Unsaturated flow in soils by themselves is a very complex matter, let alone for the soil/fabric situation. I know of no work specifically on this topic in the cross plane direction. We are doing experiments in the transverse direction aimed at air transmissivity beneath pond liners (geomembranes). A paper on the subject will be available next year.

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Filter Criteria for Geotextiles

Critères de filtre pour les géotextiles

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Question:

Do you know examples of failures caused by clogging?

Answere by J. P. Giroud:

I know two examples of failures caused by clogging. In both cases, clogging occurred because the geotextile was not placed in good contact with the soil. As a result, fine soil particles were free to move and accumulate at the surface of the geotextile. These two examples were: (i) in the first example, coarse stones, used in French drains, did not apply uniform pressure on the geotextile; and (ii) in the second example, stones, placed over a geotextile to protect a bank, moved slightly downward, thus inducing wrinkles in the geotextile. In the latter example, clogging of the geotextile caused a catastrophic failure of the whole slope protection during the first rapid drawdown of the reservoir.

Question:

Would the soil retention criteria for large c_u values not lead to clogging and blinding (blocking) of the geotextile?

In adverse, larger pore, sizes will allow the finer particles to pass through, but in the long run, a graded granular layer, acting as a filter, may be formed in the soil directly next to the geotextile.

Would you please comment on the above.

Answer by J. P. Giroud:

The retention criterion presented in my paper is intended to provide optimum geometrical conditions for the geotextile filter to work without allowing a significant amount of particles to pass through. I agree that other mechanisms are possible. For example, another approach is to use a geotextile with large openings and allow fine particles to pass through until the coefficient of uniformity of the soil decreases enough so the retention criterion is fulfilled.

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Question:

Si on extrapole le critere de la figure 7 pour des moraines avec des coefficients d'uniformite superieurs a 100, il apparait que les geotextiles soient difficilement utilisables devant de tels sols.

Ne croyez vous pas que des considerations purement geometriques ne masquent un reequilibre dynamique a l'interface sol-geotextile comme la formation d'un cake?

En remarque finale, des essais faits a polytechnique demontrent que le critere de la figure 7 est beaucoup trop severe!

Answer by J. P. Giroud:

The retention criterion presented in Fig. 7 is probably too restrictive for highly cohesive soils. However, the simplistic criterion, used by many and criticized in my paper, would be dangerous in the case of soils with a large coefficient of uniformity and a medium cohesion. For example, for the silty sand of Valcros Dam, discussed in my paper, a geotextile filter, designed using the simplistic criterion, would have had 7mm openings which would have probably led to a high risk of piping.

Question:

You suggested two equivalent expressions for the permeability criteria for geotextiles -- one using the coefficient of permeability and the other using permittivity.

For which types of geotextiles do you recommend the first (coefficient of permeability) and for which do you recommend the second (permittivity)?

Answer by J. P. Giroud:

In my paper, I propose a permeability criterion for geotextile filters. This criterion can be written in two different ways: (i) coefficient of permeability of the geotextile larger than one tenth of the coefficient of permeability of the soil; and (ii) permittivity of the geotextile (expressed in s^{-1}) larger than ten times the coefficient of permeability of the soil (expressed in m/s). In practice, the second criterion can be used with all types of geotextiles because permittivity is easy to measure for all geotextiles. The first criterion can be used only with geotextiles the thickness of which is sufficient to allow accurate measurement of their coefficient of permeability.

Question:

Dans le critere de retention pour les filtres geotextiles l'un des parametres. Bien entendu, est le dimension des ourvertures du geotextile. Comment mesurer et expimer cette caracteristique du geotextile? Que pensez vous des methodes de tamisage?

Answer by J. P. Giroud:

Several methods are available to measure opening size of geotextiles: calculations knowing mass per unit area, thickness, fiber diameter, and polymer density; direct measurement using image analyser; indirect measurement either using suction or sieving calibrated particles such as glass beads. Sieving has been criticized because it is an indirect measurement and because the lower part of the porometric curve obtained from sieving depends on the duration of the test (which does not significantly affect O_{95} determined from the upper part of the curve). However, I believe sieving is, in many cases, the most convenient method available at present because of the limitations of the other methods. Calculations give only an average opening size. The image analyser method is applicable only to thick geotextiles such as needlepunched nonwoven. The suction method does not seem to give reliable results.

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Question:

A. Is there experimental or long term field experience that supports your retention criteria?

B. Is the action shown in your drawing of dense and loose soils moving through a geotextile opening an initial phenomenon that stabilizes after start of flow-through the system or is this a long term phenomenon that could lead to substantial loss of soil?

Answer by J. P. Giroud:

A. The answer to question A may be found in Appendix 4 of my paper.

B. The answer to question B is: the action shown in Fig. 6 is supposed to be an initial phenomenon that stabilizes.

Question:

Pour les sols dont Cu>3 comment distinguez vous ceux qui ont une discontinuité dans la courbe granulometrique de ceux qui n'en ont pas.

Les criteres de rétention puivent-ils êtes différents?

Answer by J. P. Giroud:

The proposed retention criterion is related to soils having a continuous grading curve. The case of soils with a discontinuous grading curve is discussed in Appendix 1 (Fig. 3) of my paper.

Question:

Would you agree that the primary function of geotextiles in protecting soil is to prevent it from being moved by water action?

If continuous soil movement can occur through the geotextile, the geotextile would not effectively function even if gradient ratio data appear favorable.

Answer by J. P. Giroud:

It is impossible to say in general that the primary role of a filter is to prevent movement of particles. Filtration is a compromise. Filter openings should be large enough not to disturb significantly the flow of water (permeability criterion) and small enough to retain soil particles (retention criterion). In some instances, piping must absolutely be prevented and some clogging is acceptable, therefore, geotextiles with the smallest openings authorized by the permeability criterion should be used. In other instances, clogging must absolutely be prevented and some loss of particles is acceptable, therefore, geotextiles with the largest openings authorized by the retention criterion should be used. To govern a compromise, the two criteria (permeability criterion and retention criterion) are needed. As it is implied in the comment by Tan, a geotextile filter, designed with only one criterion, may not work.

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Contribution to the Study of the Clogging of Geotextiles. Morphological Approach

Contribution à l'étude du colmatage des géotextiles. Approche Morphologique

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Question:

During the presentation, the author showed a slide of woven geotextiles which is embedded in the middle of completely clogged sort. How did it happen that the clogging occurred on both sides of the fabrics almost equally in dimension and density?

Answer by M. Sotton:

Je répète que les géotextiles étudiés ont presque tous été prélevés dans des ouvrages dans lesquels ils assuraient une fonction predominante d'anticontaminant.

Les examens microscopiques des lames minces realisées dans les complexes "sol - textile" révelent très bien la discontinuité que constitue la présence du géotextile entre le sol support et le sol d'apport.

Néanmoins si on examine plus intimement la contamination des géotextiles dans leur épaisseur, on constate:

- pour les non tissés aiguilletés, une contamination différente à trois niveaux: base - centre - partie supérieure.
- pour les tisses plus minces, une contamination plus homogene, qui peut faire penser sur les clichés à un colmatage "sur deux faces".

Question:

Has Mr. Sotton any knowledge of alteration of physical properties of geotextile due to clogging? In one case I encountered, a needle-punched polyester fabric became clogged by an iron precipitate, after which it appeared to be more susceptiblè to mechanical damage (tearing) than the clean fabric. Can Mr. Sotton comment, please?

Answer by M. Sotton:

Nous avons testé de nombreux échantillons de géotextiles non tissés aiguilletés présentant des degrés de colmatage parfois important.

Les essais mécaniques en traction font souvent apparaître une augmentation du module de l'échantillon colmaté par rapport à l'échantillon neuf, une perte de l'allongement de rupture et une perte de résistance qui en moyenne est de 30% . . .

Il convient d'être prudent et de ne pas attribuer ces modifications de propriétés au seul colmatage, mais de prendre aussi en considération le "travail" rempli par le géotextile à la mise en oeuvre et lors du fonctionnement de l'ouvrage. En ce qui concerne le colmatage ferrique, il est souvent à associer à une activité bactérienne qui peut se développer indifféremment dans le sol et/ou le géotextile.

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Question:

In the systems shown in your slides, much clay and silt had penetrated the fabrics. By rising the usual criteria (e.g., O_{95}/D_{85} 1) for fabrics and soil being protected, would such high levels of contamination be expected?

Answer by M. Sotton:

Il faut souligner que les géotextiles étudiés ont été prélevés dans des ouvrages où, pour l'essentiel, ils jouaient le rôle d'anticontaminant.

Ils ont ensuite été mis en contact avec des sols comportant beaucoup de fines.

La pénétration de ces minuscules particules dans le géotextile devient importante dès que leur teneur dans le sol excède 30%:

dans de telles conditions les règles de filtres usuelles ne sont plus applicables.

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Test Areas with Vertical Drainage Systems

Champs d'essai aux systèmes de drainage vertical

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Question:

Les surpressions interstitielles des figures 5 & 6 ont une variation beaucoup plus importante que les mesures dutassement. Il semble alors probable que les capteurs ne sont pas places dans des couches rigouresement identiques et q'on ne puisse comparer les resultats entreux.

Plusieurs essais comparatifs entre drains verticaux ont ete realises de par lemonde, mais a notre connaissance. N'ont jamais presente des resultats aussi disperses. Peut-on en conclure que les resultats ne s'appliquent qu'aux seuls sols hollandais?

Answer by A.C. Maagdenberg

Settlements

In practice the functioning of a vertical drainage system is mainly checked by means of measuring the settlements. The course of recorded settlements is then compared with the settlement prognosis, which is based on calculations using the parameters obtained from laboratory test results on undisturbed soil samples. This usual procedure is also followed here, to judge the settlement recordings of the different test areas.

The recorded course of settlements falls clearly behind the calculated settlement prognosis.

The said settlement prognosis is, by the way, based on the real applied filling-schedule and the real performed loading-schedule in practice. The calculated settlements, 9 months after the completion of the fill, are on an average 1.00 m higher than the recorded settlements in-situ. This phenomenon, in fact, is not alarming. More than once the calculations, based on the soil-parameters, obtained from laboratory tests, reveal a course of settlements which does not match completely with the settlement behaviour in-situ.

Presumably the cause of this ascertained difference between theory and practice can be found in the behaviour of the peat layers in the subsoil of the test areas.

In my paper, I have already made a mutual comparison between the settlement behavior of the different test areas, based on an adapted (and sustained by experience) settlement prognosis for the settlement recordings up to January 1982. However, the data in table 4 should be interpreted in the right way. Drawing conclusions, with respect to any possible difference in the subsoil of the test areas, from a comparison between column 4 and 5 in fact is impermissible. It is not allowed to compare, without due consideration, a settlement prognosis for an embankment of 5.8 m height, with a measured settlement of a circa 0.8 m thick sandlayer. I like to emphasize the qualification "circa 0.8 m." Even the most careful engineering method, used for these test areas, leads to a variation from 0.80 m to 0.90 m of the thickness of the first sandlayer. The effect of the variation in layer thickness of the first stage will especially show in the final settlement of this relatively small first sandlayer. Such a variation of the thickness becomes less important when the load, i.e. the thickness of the sandlayer increases. Every settlement, recorded in table 4, is the average of five settlement plates in the test area concerned. For reasons of illustration the settlement curve of the central settlement plate in each test area has been plotted in figure 7.

From the results of the measurements up to December 1982 there are good reasons to modify the settlement prognosis a little more. Such modifications would lead to adapted percentages of settlements, which probably also will be a poor approximation of the reality and which are again based on assumed values. Therefore, it appears more realistic to wait with the mutual comparison of the test areas, based on the course of settlement, until the completion of the measurements and until the real composition of the subsoil of each test area has been determined, from the results of borings (1983).

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Answer by A. C. Maagdenberg (cont.):

Pore water pressures

At the present stage of the project it is preferable to make a mutual comparison between the different drainage systems, based on more direct information, particularly by studying the course of excess pore water pressures. On account of the piëzometer-readings at different depths, there are no reasons to doubt the accuracy of the piëzometers. The filter-depth of the piëzometers was measured frequently, so that it is well known in which soil layer these piëzometers reach. At t, the filter of piëzometer B of test area IV reaches into the clay-layer.

The stratigraphy of the subsoil at the location of the test areas is typical for the major part of the western part of the Netherlands. Of course this stratigraphy will differ from other parts of the world, so that the test-results have to be handled with care when using them with other projects.

For each field-test, concerning the material soil, it counts that the results cannot be translated simply into situations from one point in the world to another. The real engineer, involved with soil mechanics, will interpret the results of a certain field test by considering the history of the project, the soil properties and other local circumstances.

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Hydraulic Function and Performance of Various Geotextiles in Drainage and Related Applications

Performance et rôle hydraulique des géotextiles dans le drainage et les applications similaires

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Question:

There are several references to tests carried out at Colorado State University. Have these tests been published, and if not, could some indication be given as to the soil types, number of tests, etc.?

What is the justification in changing the conclusion of Colorado State that the P_{95} over the D_{95} should be less or equal to 3? Surely it is not the 1972 test conducted for a period of 60 hours.

Answer by H. H. Tan:

The Colorado State University test results were presented at the Drainage Symposium of the American Society of Civil Engineers in 1980, also at the INDA Conference in New Orleans in 1981.

Colorado State University researchers used five different soil types with varying clay content and observed that for soils that satisfy the ratio:

$$\frac{D_{85}}{D_{50}}, \frac{D_{50}}{D_{35}}, \frac{D_{35}}{D_{15}} < 5,$$

geotextiles having $\frac{P_{95}}{D_{85}} <$ (0.04 to 3.82) could effectively restrain the soils.

Their prolonged permeameter tests were not for only 60 hours but for more than 1000 hours. They, therefore, proposed an average ratio:

$$\frac{P_{95}\ \text{(EOS of fabric)}}{D_{85}\ \text{(of soil)}} < 2$$

Question:

Do you use any standard permeability test to evaluate the drainage properties of typar? What is that test?

Is such a permeability test adequate for other applications (such as for subgrade stabilization)?

Answer by H. H. Tan:

Permeability tests for evaluating Typar® are conducted using a DuPont -- developed method that eliminates errors, such as due to entrapped air from water on or in the geotextile, or uncontrollable water flow rates. The method using internally designed apparatus is convenient for measuring the ability of geotextiles to pass water at certain heads. Compared with the flow rates of water through normal layers of soil, the flow rates through geotextiles are hundreds of times greater. Hence, the soils usually control water flow and are the limiting factors. Because of the very great difference between thicknesses of soil layers and of geotextiles, it is obvious that their coefficients of permeability should not be used for comparing their true permeabilities.

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Question:

Could you please give some more details about the dams you showed us and the function of the geotextile in each case?

Answer by H. H. Tan:

In the dams I showed the spunbonded polypropylene geotextile Typar® was used in the drainage systems to prevent piping of the compacted soil of the dams. The geotextile was installed in chimney and blanket drains, also in a French-type drain system within the body of earth dams. In a rock-filled mine tailings dam in Chile, South America, the geotextile was used between a layer of crushed stones and a sand blanket to keep the mine tailings and the sand from washing into the stones.

In one of the earth dams the geotextile was also used to protect the dam's entire upstream slope and a 10 meter wide band at its downstream toe against surface erosion by changing water levels; the upstream slope against the reservoir's water, the downstream slope against tidal changes in the riverbasin.

Question:

When speaking about filter conditions the geometry criterion is involved. The second criterion is that of hydraulic nature. Of course if the water flow rate is high some particles can go through but will be stopped by low water flow rate.

Do you think that the general filter criterion should have two parts: geometric one and hydraulic one?

Answer by H. H. Tan:

The function of geotextiles used in drainage and for protecting soil against scouring by water in soil erosion control systems is to prevent soil from being moved by water action, while permitting unreduced flow of water.

Although often referred as filter fabrics, in order to function in long lasting, stable systems, the geotextiles must perform <u>not</u> as a normal filter for collecting water-borne particles, but as an effective soil restraint that does not become clogged.

The key factor is the soil quality, indicated by its particle size distribution. Soils with particle size distribution that satisfy the ratio:

$$\frac{D_{85}}{D_{50}}, \frac{D_{50}}{D_{35}}, \frac{D_{35}}{D_{15}} < 5$$

are self-filtering.

A layer of such soils can prevent soil particles behind it from moving through. A geotextile that can satisfy:

$$\frac{P_{95}\ (95\%\ \text{of geotextile opening sizes})}{P_{85}\ (85\%\ \text{of soil particles size})} < 2$$

will effectively restrain self-filtering soils that actually function as filtering media.

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Compressibility and Hydraulic Conductivity of Geotextiles
La compressibilité et la conductivité hydraulique de géotextiles

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Question:

In your formula for filter criterion β you have introduced 0.5. Has this figure some physical meaning or is it just used to fit the experimental data?

Answer by A. McGown:

No, it is just the fit obtained for experimental data.

Question:

Why is the measurement of the transmissivity coefficient n , according to the formula $i = bv^n$ under unity. Is it not due to the fact that it is measured in radial flow?

Answer by A. McGown:

The values which are given in the paper for radial flow are in fact apparatus specific and all radial flow test data are apparatus specific. Only planar tests give data which can be usefully used for design, and that is stated in the paper.

Question:

Accepting that the use of multi-fabric layers give consistent results, could this method be really justified or creditability placed on its values when, in general, the fabric is used as a single layer? Could you also state the percentage of air in the water used?

Answer by A. McGown:

Work in France shows that such multi-layer tests are credible for nonwovens, but for wovens they are not. In our tests we used fully de-aired water (or, at least, as near that as is possible to achieve).

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Transmissivity of Geotextiles and Geotextile/Soil Systems
Transmissibilité transversale de géotextiles et systèmes géotextile/sols

Vol. 1, p. 173

Question:

In the transmissivity test with soil, were the soils adjacent to the geotextiles compacted?

Answer by R. M. Koerner:

The soils adjacent to the geotextile were placed loose and the only compaction was in the leveling process. It was felt that this was the most critical situation and if movement into the fabric were to occur it could easily do so in this state.

Question:

Was back-pressure used when measuring fine-soil-fabric permeability?

I personally think that the difference can be tremendous.

Answer by R. M. Koerner:

Back pressure could indeed change the magnitude of the transmissivity values. I would suspect the values would be increased and that our values represent a lower bound in this regard.

Our test apparatus (measuring planar transmissivity) does not allow for back pressure adaptation and, as such, is a limiting feature. Other systems (Raumann, Raymond, Swiss method, etc.) can perform such tests. These all measure radial transmissivity in devices similar to consolidometers used in soil testing for compressibility.

Question:

What was the running time of your transmissivity tests with the fabric/soil combinations? Clogging is a long-term developing.

Answer by R. M. Koerner:

All tests were performed after drainage for 24 hours. This was initially felt to be adequate, but after performing long term cross fabric drainage tests, it was seen that up to 200 hours are required for soil/fabric structure to materialize and stabilize. The tests themselves were variable in their duration, those with fine grained soils being longer than those with coarser soils. The usual flow volume was one liter, the time varying considerably for this volume to accumulate.

SCHEURENBERG, R. J.
Watermeyer Legge Piesold and Uhlmann, Braamfontein, Republic of South Africa

Experiences in the Use of Geofabrics in Underdrainage of Residue Deposits
Experimentation dans l'usage de géotextiles pour le drainage sous les residus

Vol. 1, p. 199

Question:

Do you have seismic activity in South Africa that would affect the stability (safety) of your tailings dam's structures?

Answer by R. J. Scheurenberg:

Moderate seismic activity can occur in the Witwatersrand region, largely due to movements in the undermined rock, sometimes at great depth. Effective drainage and good consolidation of the tailings deposits, especially in the outer wall sections, are important to avoid liquefaction under seismic loading.

Question:

Do you regard the sand blanket over the horizontal parts of the geotextile as a necessary insurance against clogging or is it only an incidental advantage:

Is there a long term programme for monitoring the underdrainage water for

a) flow rate

b) chemistry

Could you say how you chose the geotextile most appropriate for your application?

Answer by R. J. Scheurenberg:

Both functions of the sand blanket are considered important: to provide a barrier against blinding of the geotextile by fine particles in the tailings and to extend the area of drain effectively collecting seepage water.

The filter drain outlets are observed regularly to check that they are flowing but the actual flow rate is not measured. Chemical analyses of underdrainage water are performed occasionally. These activities are part of a comprehensive long term monitoring programme for the whole tailings dam.

The most appropriate geotextile was originally selected after laboratory simulation tests of the several geotextiles then available in South Africa. At that time (1976) the theoretical background for geotextile design was very limited so we had to rely on their performance in the laboratory permeameter tests.

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Question:

With respect to the clogging of the exposed fabric, have you explored the possibility that the encrustation may be due to precipitation resulting from evaporation of fluids wicked through the partially exposed fabric from saturated sediments below? Please comment.

Why was the 3-layer composite drain required? Would not a thicker nonwoven fabric have sufficed for the cut off drain?

Answer by R. J. Scheurenberg:

It is possible that the precipitate may originate from the sediments underlying the slimes deposit at the problem area of the Ergo tailings dam, but we do not think that this is so. The underlying soil cover in the affected area is a thin layer, 0,5 to 1,0 m, of "black turf" decomposed dolerite on top of sound dolerite rock. The encrustation had the appearance of an iron precipitate and this was confirmed by chemical analysis. It is almost certain that the iron is derived from the tailings rather than from the natural soil below.

For the fin drain at Crown Sands, a thick nonwoven fabric was considered but did not have sufficient capacity to transmit the design seepage water flow.

Question:

Could the precipitate be caused by algae or other biological agents?

Answer by R. J. Scheurenberg:

Bacterial activity did not appear to have contributed to the formation of the precipitate initially. Only after some months did we observe what appeared to be the products also of bacterial action.

Question:

In your laboratory model to evaluate the failed drain section in the second dam, was the filtering section of the fabric directly exposed to the sunlight or wetting and drying conditions?

Both conditions appeared to exist in the field and would have contributed to the condition of the fabric. Please discuss.

Answer by R. J. Scheurenberg:

In the laboratory model, the fabric was covered by a layer of slimes about 100 mm thick and was thus not directly exposed to sunlight. It was exposed to moist air in the voids between the stones below the fabric.

In the field, the fabric is usually covered by sand and slimes. Access to air would facilitate oxidation of a continuously replenished supply of dissolved iron oxides leading to the observed precipitation and clogging. This could take place within the filter drain even when covered. Exposure to sunlight would be unusual and this could degrade the fabric itself but might not contribute to formation of the precipitate.

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Question:

Are there any records of the variation of pH level in the tailings?

Answer by R. J. Scheurenberg:

The pH level of the tailings as it arrives at the dam depends on the treatment process used on the ore. At Ergo the pH is usually about 6 or 6,5, although at times the pH may drop to below 5. At Crown Sands the pH is about 10,5. However, the pyrites present in gold tailings cause slow chemical changes to take place in the deposits and in time the seepage water may become highly acidic with pH dropping to as low as 2.

Question:

For what length of time could polyester fabric around the filter drains be exposed to sunlight if they are not covered by a sand blanket?

Answer by R. J. Scheurenberg:

Depending on the configuration of the tailings dam, the filter drains could be exposed for a few months or up to 3 or 4 years before they would be covered by the tailings deposit.

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Hidroconsult, São Paulo, Brazil
SMITH, R. T.
Transpavi—Codrasa, São Paulo, Brazil
VERTEMATTI, J. C.
Rhodia, São Paulo, Brazil
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D.A.E.E., São Paulo, Brazil

Continuous Retaining Dikes by Means of Geotextiles

Digues continues de rétention au moyen de géotextiles

Vol. 1, p. 211

Question:

1) Did you investigate stacking the sausages?

2) Do you have any observations on long term u-v exposure and abrasion of the fabric?

Answer by J. Vertematti:

1) No, we didn't yet. But we do intend to investigate this possibility, specially the shear resistance between the sausages, and the effect of surcharges (bursting and/or creep of geotextiles).

2) Related to u - v rays, please see our answer to the previous question. About abrasion I've got remarks to do:

- We had a bursting of the sausage in Sao Luis site, due to internal abrasion on the fabric caused by the sand plus little shells. A double layer of fabric solved the problem.

- Some months after the dikes were concluded in Cubatão site, we have observed superficial abrasion on top of some dikes due to walking on it: the sausages became an access way for the workers. This fact caused no damage to the dikes though around 10% of the geotextile filaments have been destructed. In future works, a simple passage made of wooden boards can prevent abrasion.

Question:

How long did the sausage dikes withstand the exposure for sunlight?

Answer by J. Vertematti:

The dikes were supposed to be exposed only for three or four months but due to a delay in the works they have been exposed to date, that is around twenty months and no bursting accurred. Since this is a definitive work all dikes will be recovered by a sandy layer as soon as possible.

COUCH, F. B., Jr.
U. S. Corps of Engineers, Nashville, TN, U.S.A.

Geotextile Applications to Slope Protection for the Tennessee-Tombigbee Waterway Divide Cut

Applications géotextiles pour la protection des terrassements de partage des eaux du Tennessee-Tombigbee Waterway

Vol. 1, p. 217

Question:

1. How far up the slope will high waters reach and what will be the water velocity?
2. Was the affect of a geotextile introduced in calculations for determining the stone sizes?
3. What were the physical characteristics for the geotextile?
4. Please indicate a cost per square for the finished construction.

Answer by F. B. Couch:

1. High water can reach the top of the slope protection; however, it should be a rare occurrence. The canal is part of a reservoir system. Therefore, no velocities as such will be present. Traffic generated waves are estimated to be 0.9 meters.
2. No.
3. Set forth in Table 1 of the paper.
4. Filter Fabric - $1.80 per square yard stone - $15.00 - 17.00 per ton.

Question:

What weight of fabric was used under the rip-rap on this project?

Answer by F. B. Couch:

About 4 oz/sq yd.

Question:

Could a changing of the fabric requirements like strength and water-permeability have been a solution for tearing and water overpressures?

Answer by F. B. Couch:

A stronger fabric would have been more resistance to puncturing. However, as mentioned in the paper, the placement method and surface dressing might also puncture a heavier fabric. Higher water-permeability and better resistance to clogging would definitely have been an improvement.

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Question:

The experience on German waterways showed that stone dumping operations don't damage the geotextile using heavy needle-punched nonwovens. The fabrics used have to fulfill the requirements of the very severe penetration test of the Federal Institute for Waterways Engineering (BAW).

What do you think of using needle-punched nonwoven fabrics?

Answer by F. B. Couch:

A heavy nonwoven fabric would be a definite improvement over the material used on our project, although, it would probably be more costly. As previously mentioned, surface dressing with hydraulic backhoes ,etc., may push stones through nonwoven fabric, as well as what we used. We are using some nonwoven fabrics on some of our later projects.

GYSSELS, E.
UCO, Ledeberg, Belgium

The Expansion of the Belgian Zeebrugge Harbour in the Sea and the Use of the Woven Geotextiles

L'expansion du port de zeebrugge en mer et l'emploi des géotextiles tissés

Vol. 1, p. 229

Question:

Do you have any experience with dumping stones directly on fabrics? Especially the manner of dumping, the weight of stones are interesting related to the results of Mr. Couch.

Answer by E. Gyssels:

The principle is not to dump stones directly on top of fabrics, and it all depends on the application.

1. Dike protection against erosion, example:
   Local conditions: inclination of dike 12/4
   ballasting 1700 kg/m3
   geotextile used as filter: monofilament
   P.E. 280 gr/m2
   tensile strength: lengthwise 280 kg/5 cm
   broadw. 165 kg/5cm
   stones are dumped with a crane and directly on top of the fabric. A network of fascines must be used as well in order to avoid the gliding of the stones dumped on top of the fabric.

2. As seamattresses: dumping of stones always realized on top of fabric protected with a network of fascines. Even if the fabric should resist the direct dumping of the stones, then still the fascine network must be used for other reasons.
   weight of quarry blocks: see 1st question

Question:

What weight of rock can be dumped on the woven textile fascine combination?

Answer by E. Gyssels:

Weight of rock: depends on local circumstances:

1. Stream
2. wave and screw-water action of ships
3. waterlevel on the dike construction or behind.

Ex.: for a large stream with tidal water (difference of 5 m height - high - low waterlevel) layer of quarry blocks (not peeling off)
weight of stones 5 to 25 kg (with max. 5% lighter weights)
density of stones about 1700 kg/m3
slope 12/4

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Geotextiles Against Wind Erosion
Les géotextiles dans la lutte contre l'érosion éolienne

Vol. 1, p. 241

Question:

Will you please comment on U.V. stability of geotextiles in this type of application ?

Answer by E. Auriat:

U.V. stability

Le temps de conservation des caractéristiques de ces articles doit correspondre à la période durant laquelle elles sont indispensables.

a) la grille : celle-ci doit rester suffisament résistante jusqu'à ce qu'elle soit ensevelie totalement par le sable qu'elle aura arrêté. Cela peut donc durer de quelques heures à quelques mois.

   Cette première installation sera alors pemplacée c'est-à-dire surmontée par une deuxième du même type et ainsi de suite jus-qu'à l'obtention d'une dune fixée créant une autoprotection.

b) Le réseau de couverture ; il doit conserver une certaine résistance dynamométr-ique jusqu'au démarrage de la végétation.

Question:

What were the physical characteristics of the geotextiles?

Answer by E. Auriat:

Caractéristiques physiques des géotextiles :

Grille : 1100 dtex polyester chaine = 4fils/cm
trame = 2fils/cm
Réseau de fibres : mèche de 27000 filaments de : IIdtex.

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Question:

1. How do you determine the height of a grid when it's used against wind erosion in open areas such as the edges of a desert?

2. Would the geotextile used as a cover allow the growth of vegitation through it and if it does, do you anticipate any problems it might cause to the cover?

Answer by E. Auriat:

1) 90% du sable se déplace sur une hauteur de 30 cm (dixit UNESCO s lon étude USA).
Une grille de 1 mètre de haut arrête donc la presque totalité du sable qui s'accumule et l'ensevelit. Elle devra donc être surmontée par une 2 ème grille . . .
De plus, 1 mètre (ou ses multiples) est une bonne largeur de fabrication.

2) Le géotextile de couverture permet la croissance des graminées entre ses mailles, sans qu'il soit soulevé . . .
Cependant pour les plants d'arbres de pépinières il faut les planter avant la pose du réseau. Ensuite on pratique une ouverture à la main pour faire passer les petites branches à l'air libre.
Méthode appliquée en Afrique et confirmée par un essai a l'Institut National de la Recherche Agronomique.

Question:

Has there been any scientific research on the migration of soil particles through the geotextile under wind erosion?

Answer by E. Auriat:

Il n'y a pas eu de recherches scientifiques sur la migration des particules de sable à travers les deux géotextiles.

Il s'agit essentiellement de recherches expérimentales ayant abouti à un début de commercialisation. Ces expérimentations étaient cependant fondées sur des études, des informations provenant d'organismes spécialisés de différents pays.

Question:

Le Geotextile est il biodegradable?

Answer by E. Auriat:

Ces deux articles sont photodégradables.

Question:

Peut on planter des arbres à travers le textile ou est-ce que ca cause des ruptures?

Answer:

La plantation de petits arbres (#30 cm) de pápinières est la solution préférable pour la fixation des sables.
Processus: a) Plantation habituelle
b) Pose du réseau de couverture par dessus les plants.
c) Pratiquer manuellement une ouverture
dans le réseau en écartant les fibres au-dessus de chaque plant afin de faire passer à l'air libre les petites branches Ainsi les fibres sont plaquées au sol de chaque côté de la base de l'arbre.

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Question:

1. Le geotextile est quel materiel?

2. Comment a ete la reaction des authorites? Quand veulent - ils commencer a installer des grandes quantites?

Answer by E. Auriat:

1. Le géotextile de couverture est à l'origine une mèche de fibres textiles en acrylique, formée de quelques dizaines de milliers de filaments de 5 à 10 microns de diamètre.

2. Réaction des autorités: positive. Des essais, elles sont passées à l'utilisation courante sur 100 ha environ. Opération arrêtée momentanément pour des raisons indépendantes de cet article, chez le fournisseur.

Question:

Quelle est la dureé de vie es comptee des géotextiles utilisés contre l'erosion éolienne, et quelle est leur nature?

On pourrait imaginer d'utilises ce type de geotextile pour la lutte contre les congères de neige. Dans ce cas, le géotextile n'a pas pour objectif de s'effacer lorsque la stabilisation du sol a eu lieu.

Answer by E. Auriat:

1) Nature des géotextiles :
   grille : tissage dit "turbine"
   fils 1100 dtex polyester
   contexture/cm : CH : 4fils, TR : 2 fils.
   Réseau : mèche filaments acryliques parallèles apptoximativement, ouverte sur le sol.
   27 à 100 000 filaments de 3 à 11 dtex.

2) Durée de vie (résistance aux U.V.)
   grille avec traitement anti U.V. : 1 à 2 ans; avec imprégnation vinylique environ 10 ans.
   réseau : 1 à 2 ans.

3) Dans la lutte contre les congères de neige : il s'agit je crois, de déplocer le lieu de formation habituel des congères en modifiant par un genre de brise-vent, la circulation des courants aériens occasionnés par la topographie du site . Dans ce cas le réseau de couverture est inutile, seule la grille peut être efficace. Mais compte tenu de nos essais en montagne il faudrait entreprendre toute une étude assez conplexe pour aboutir à une contexture mieux adaptée au problème de la neige.

Question:

Have you investigated the possibility of adhering seeds to the textile nets placed on the ground, to aid in establishing vegetation? Possibly the seeds could be presoaked to aid in germination?

Answer by E. Auriat:

Pas de recherches sur semences fixées au textile car cela ne résoudrait pas le problème capital du semis sur sable en climat semi-aride.

La difficulté essentielle ne réside pas dans la capacité de germination mais dans le maintien en survie de la petite plante naissante dont les racines doivent traverser la couche de sable sec (20 cm, 40 cm ou plus) pour atteindre la couche humide profonde; d'où la nécessité d'arrosage au début de la vie de la plante.

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Question:

How deep are the posts driven and what life do you put on them?

Answer by E. Auriat:

Profondeur et durée des poteaux bois :

grille : longueur : I70 cm dont 70 cm en terre
section : ≃ 7 x 7 cm

ressau : longueur ≃ 40 cm dont 30 cm en terre
section ≃ 2 x 2 cm ou plus

duree : pas de probmèmes car elle est toujours supérieure aux besoins.

Question:

1. What polymer type was used in each of the materials you discussed?

2. How long do the materials have to maintain their integrity for your applications?

Answer by E. Auriat:

1. Polymère acrylique pour le réseau de converture
Polymère polyester pour la grille.

2. Durée de conservation des caractèristiques;
   a) Grille: de quelques semaines à quelques mois c'est-à-dire qu'il faut qu'elle puisse subsister jusqu'à la première période de grand vent qui amènera trés rapidement une grande quantité de sable. Celui-ci va bientôt ensevelir la grille qui alors devra être surélevée ou plus certainement surmontée par une deuxième installation . . . et ainsi de suite jusqu'à obtenir une dune auto-protectrice qui alors pourra être fixée en surface.
   b) Réseau de couverture: il doit conserver une certaine résistance dynamométrique jusqu'au démarrage de la végétation.

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PUIG, J.
Laboratoire Régional des Ponts et Chaussées, Toulouse, France

The Use of Honeycombed Geotextile Lap to Combat Erosion

Emploi de nappes géotextiles à structure alvéolaire pour la lutte contre l'érosion

Vol 1, p. 247

Question:

1) What is the water permeability of the strips transversal to their plane?

2) Is there any experience of flattening down the upstanding strips by rough filling, for instance by dumping a complete truck load on the slope?

Answer by J. Puig:

La permeabilite des bandes est celle du produit utilise, a savoir le non tisse de filaments continus lies mecaniquement et chimiquement produit par Rhone-Poulenc France. La permeabilite au travers du produit, mesuree sur le materiel mis en place est de $k= 1,6.10^{-6}$ m/sec.

Lors du remplissage, la structure d'Armater est telle que les alveoles restent parfaitement verticales et se remplissent sans probleme. Il suffit de fixer les cotes des panneaux.

Question:

Vous avez parlé du rôle mécanique de Armater pour renforcer les pentes.

Pensez-voux que d'autres effets ou rôles soient importants pour assurer le bon fonctionnement d'une telle structure alvéolaire?

Answer by J. Puig:

Je pense que le rôle hydraulique du géotextile constituant les cotes des alvéoles, peut avoir un effet non négligeable sur le comportement de l'Armater en revêtement de talus. Le drainage de l'eau par le réseau important que constitue les alvéoles hexagonales peut contribuer dans une large mesure au maintien des caractéristiques intrinsèques initiales du matériau de remplissage et donc à son bon comportement mécanique. Des études sont actuellement en cours pour tenter de mettre en évidence ce comportement.

Question:

What is the trade name and manufacturer of the geotextile armature used?

Answer by J. Puig:

Le nom du produit est ARMATER et il est fabriqué à partir d'un non tissé BIDIM de filaments continus polyester liés mécaniquement et chimiquement produit par la Société RHONE POULENC.

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Question:

Concerning the anchoring of the lap at the top of the slope by means of a shallow continuous trench, did you take into account the risk of a mass movement along a slip surface starting at the bottom of the trench itself?

Answer by J. Puig:

Une telle tranchée d'ancrage si elle n'est pas bien remblayée et compactée, peut constituer un point faible du remblai et l'introduction d'eau peut conduire a l'apparition de désordres. C'est pourquoi nous ne réalisons plus l'ancrage au moyen d'une saignée, mais seulement en realisant un décaissement du bord de talus sur un mètre de large et sur l'épaisseur correspondant à la hauteur d'une alvéole. Après fixation par quelques piquets et remplissage des alvéoles, un compactage est indispensable.

Question:

1. What is the advantage of using geotextiles over other materials?

2. How did you construct overlaps?

Answer by J. Puig:

1. L'intérêt de l'emploi d'un géotextile non tissé pour la confection d'Armater utilisé dans la lutte contre l'érosion est dû aux caractéristiques hydrauliques des non tissés.

2. La liaison des bandes de géotextiles, est obtenue par collage.

Question:

Can you give an indication of the cost of this honeycomb system? Up to which slope angles can it be used?

Answer by J. Puig:

Les indications de prix peuvent être obtenues auprès du fabricant Armater Industries Zone Industrielle 63600 AMBERT-FRANCE.

Armater peut être employé sur des pentes de 1/1 maximum (45° C).

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THABET, R.A.H.
Delft Hydraulics Laboratory, Delft, The Netherlands

Investigation on Long-Term Behavior of Geotextiles in Bank Protection Works

Recherches sur le comportement à long terme des géotextiles utilisés dans la protection des berges

Vol. 1, p. 259

Question:

In point 5 of your conclusion you state that iron compounds shorten the geotextile's life. Is this meant in a functional or chemical sense, and if chemical, could you explain?

Answer by R. Veldhuijzen van Zanten:

Iron compounds shorten a geotextile's life both in a functional and a chemical sense.

Functional: The permeability of geotextiles decreases, due to clogging with materials containing iron compounds. Currents and waves may clean a geotextile, clogged with silt. On the other hand, iron compounds attach to a geotextile and cannot be washed away.

Chemical: Oven-aging tests (paper reference 8) on polypropylene geotextiles, containing iron compounds, achieve relatively low values.
The thermo-oxidative degradation is quickened (catalyzed) because the falling apart of hydro-peroxides in active radicals is stimulated.
When the samples are cleaned first with hydrochloric acid (eliminating the iron-compounds), then higher values are achieved (up to a factor 2).

Question:

For these applications higher values for filter criteria are recommended in the paper. Because these criteria are dimensionless, comments should be given for the grainsize-scope they are valid for.

Also for the hydraulic boundary conditions. Otherwise dangerous general applications of these criteria cannot be avoided.

Could you please offer your opinion?

Answer by R. Veldhuijzen van Zanten:

The geotextiles involved in the investigation have been taken mainly from sites alongside Dutch rivers and canals. For the conditions on these sites (both light and heavy hydraulic phenomena, both fine and coarse bank material) in our opinion less strict design criteria can be applied. The appearing hydraulic gradients are relatively small compared to the hydraulic gradients in laboratory tests, on which the original criteria have been based.

WISSE, J.D.M.
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BIRKENFELD, S.
The Netherlands Waterworks' Testing and Research Institute KIWA, Rijswijk, The Netherlands

The Long-Term Thermo-Oxidative Stability of Polypropylene Geotextiles in the Oosterschelde Project

La stabilité thermo-oxydative à longue durée de géotextiles de polypropylène dans le project de l'Oosterschelde

Vol. 1, p. 283

Question:

You give results of the long-term stability of PP.

Are there similar results available for PES/PA/PE? What do you expect on the behavior of these polymers?

Answer by J. D. M. Wisse:

This research has only been done on PP.
The thermo-oxidative stability of linear polyester (PES) is known to be much higher than in the case of polypropylene. For this kind of application it might well be the case that no further antioxidants are needed.
Polyethylene (PE) may be effectively stabilized by adding carbon black, being a moderate antioxidant and apparently protecting PE against thermooxidation for very long times.
Polyamide (PA) is also very resistant against thermooxidative breakdown. In principle this depends on the molecular weight, however, and therefore this cannot be generalized. For the "normal" compounds being used in this field it may be assumed that a "normal" stabilization yields a sufficiently long technical life time.

N.B. Influences of wear, chemicals and hydrolysis have not been considered, but must, of course, also be taken into account.

Question:

What is the chemical identity of your low leach antioxidant for polypropylene?

Answer by J.D.M. Wisse:

The exact chemical identity of the low leach antioxidant is not known to me, unfortunately. However, because of the low leach property, the antioxidant can be assumed to be connected (coupled or grafted) onto the polymer. It is supplied as a masterbatch (i.e. an antioxidant concentrate), using PP as a basis. The masterbatch is thoroughly mixed and divided into the bulk of the material, giving a stabilization that cannot be extracted to a considerable extent.
We did attempt to remove the antioxidant by boiling-tests in water, but only a small decrease in stabilization level was observed. Because the antioxidant cannot be washed out, the thermooxidative resistance is being maintained.

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McGOWN, A.
University of Strathclyde, Rottenrow, Glasgow, UK

The Selection of Testing Procedures for the Specification of Geotextiles

La sélection de procédures d'essai pour les specifications des géotextiles

Vol. 2, p. 291

Question:

In table 3 on page 295 the overall load-extension in first time loading in isolation appears to be the only index test which could be used as a guide to long term creep and rupture.

Do the Authors anticipate that further index tests for long term creep and rupture will be introduced in due course?

Answer by McGown:

Yes, a study of the long term creep and rupture characteristics of geotextiles both in isolation and in soil is now underway at Strathdyde University.

Question:

Do you have an idea of how many "typical soils" should be covered by in-soil tests?

Answer by McGown:

We suggest that no more than four soil types will be necessary in order to establish in-soil behavior in a general manner. These would be a coarse gravel, coarse sand, well graded sandy gravel and a silt. This choice is made to illustrate the influence of the number of contact points between the soil and the geotextile. Of course, it is better for design purposes for a specific case to use the field soil in contact with the geotextile for in-soil tests.

Question:

Les proprietes de stabilite chimique des geotextiles sont considerees comme importantes par certains intervenants (McGown, Studer)

Answer by McGown:

We believe this to be important but not yet fully understood.

Vol. 2, p. 291

Question:

Dr. McGown mentioned that some geotextiles show a reduction in strength when tested in soil. Could he please indicate which types of geotextiles these are, and why he believes they behave this way? Is there a change in modulus?

Answer by A. McGown:

Basically there are two types of geotextile which may show a reduction in rupture strength and modulus when tested in-soil and compared to testing in isolation. The first are tape woven geotextiles which may suffer breakage of tapes by the presence of sharp stones in contact with them. Such geotextiles have usually shown only a modest decrease in such circumstances. The second are resin bonded geotextiles which tend to suffer significant losses when in-soil due to the soil particles breaking down the resin bonds holding the fibres.

Question:

Can the results of the various tests for permeability, tensile, elongation, porosity, etc. be compared between woven and non-woven geotextile fabrics? Can these tests be the same or do they have to be modified to suit the woven or non-woven characteristics?

Answer by McGown:

It is possible to compare data between wovens and non-wovens providing the tests from which the data is generated fairly tests both materials. Unfortunately many of the tests employed at this time do not fairly test both types. Care must be taken therefore to ensure that the basis for any comparisons are built on useful data.

Question:

One of the most important issues in Geotextiles for engineers of many civil works such as dams is that of long term durability. Several speakers mentioned this matter. What are some possible test procedures which could be considered for standards to distinguish between geotextiles of reasonable durability and those of poor durability?

Answer by McGown:

At this time no agreed test procedure for durability exists, so far as we know.

Question:

Are there specifications on the number of various tests required (in view of possible variations/sample)?

Answer by McGown:

We are working to produce such test specifications in United Kingdom but they are not yet available.

Vol. 2, p. 291

Question:

All of the tests mentioned by the speakers apply to woven and non-woven materials.

1) Does a knit product fall under either of these?

2) Can the same tests be applied to knit products?

Answer by A. McGown:

Firstly, a knitted product does not fall into either of the categories mentioned but two tests we suggest would be capable of dealing with them.

Secondly, particular care must be taken with knitted products to establish that they will not unravel.

Question:

Have you tested the reliability between friction-adhesion soil-geotextile in lab conditions and in situ conditions?

Answer by A. McGown:

No

Question:

Geotextiles which are used as embankment reinforcement are subjected to transverse, longitudinal, and out of plane stress. The stress-strain properties of primary interest are often those in the transverse direction, however, these properties will generally depend upon the longitudinal and out-of-plane stress applied to the fabric. In this paper, the authors have emphasized the need to consider the effect of the out-of-plane stress.

a) I wonder if they would comment on the relative importance of the out-of-plane stress compared to the longitudinal stress (strain)?

b) In embankments, the longitudinal strain may be zero (i.e., plane strain). However, local irregularities will often result in either compressive or tensile strains being developed in the longitudinal direction. If this occurs, how applicable are the laboratory test results to field conditions?

Answer by McGown:

a) We do not believe we should be considering relative importance but recognizing that both exist and both must be taken account of.

b) Providing the local (perhaps burst) strength of the geotextile is adequate and tearing does not then occur, we believe the overall behavior will be similar in laboratory tests that fully simulate the field conditions.

GAUTSCHI, GRABE, HEERTEN, HELMPRECHT, HUHNHOLZ, MORITZ, MURRAY, WILMERS and ZERFASS
Baustoff-und Bodenprüfstelle, Federal Republic of Germany

Recommendations on Applications, Testing and Classification of Geotextiles in Road Construction in Germany

Recommandations pour les applications, essais, et classifications des géotextiles pour la construction routière en Allemagne

Vol. 2, p. 297

Question:

All of the tests mentioned by the speakers apply to woven and non-woven materials.

1) Does a knit product fall under either of these?

2) Can the same tests be applied to knit products?

Answer by Zerfass:

We have no experience with geotextiles made of knit products. At present, the tests mentioned apply to woven, non-woven and composite materials only.

Question:

Has the German Committee given any consideration to the use of geogrids for embankment reinforcement and if so what conclusions have been reached?

Answer by Zerfass:

To our knowledge geogrids so far have not been employed in Germany in conjunction with embankment reinforcement. As soon as experience is available, there should be no problem to discuss this matter in the German working party.

Question:

Dr. McGown emphasized the importance of assessing the properties of a geotextile in a soil environment for design purposes. Does Mr. Zerfass consider that the tests proposed for the German specification will adequately provide design data?

Answer by Zerfass:

There seems to be always a problem of deriving design data from laboratory test results. This is true for geotextiles being tested according to the proposals of the German working party as well as for geotextiles tested in a soil environment, although there is reason to believe the latter way of testing leads to closer results. With the current knowledge of the behavior of geotextiles, we assume the laboratory test without soil is an easy and reproductible way of identification and of assessment of the properties.

LASSALLE, J.
Président Fondateur du Comité Français des Géotextiles, Paris, France
FAYOUX, D.
Cemagref, Antony, France
BERTHIER, J. P.
Service d'Études Techniques des Routes Autoroutes, Ministère des Transports, Bagneux, France

Specifications and Recommendations of French Geotextiles Committee

Spécifications et recommandations du Comité Français des Géotextiles

Vol. 2, p. 313

Question:

L'intention du Comité Francais des Géotextiles est-elle de faire une comparaison entre les différents produits, faisant la somme par addition ou par soustraction de la valeur de classe pour chaque caractéristique, en vue de la sélection d'un produit?

Answer by J. Lassalle:

Le but du Comité Francais des Géotextiles est de donner aux projeteurs un guide pour la selection d'un produit en tenant compte de la grille ou du profil indique dans le fascicule de recommendation.

Il s'agit essentiellement de faire trouver le produit existant sur le marché et dont le profil de caractéristiques se rapproche le plus du profil recommandé, tout en étant le plus compétitif du point de vue économique.

L'addition des appréciations numériques des classes pour dégager une seule note n'a donc absolument aucun sens.

Question:

Are there specifications on the number of various tests required (in view to possible sample variations)?

Answer by J. Lassalle:

Ces spécifications varient avec chaque essai et sont précisées dans le mode opératoire de chaque essai.

Vol. 2, p. 313

Question:

Les propriétés de stabilité chimique des Géotextiles sont considérées comme importantes par certains intervenants (Mc Gown, Studer)

D'autres intervenants ne les reprennent pas dans les paramètres à considerer dans le cadre d'un dimensionnement (Gamski, Lassalle-Fayoux-Berthier)

Dans quelle mesure le bureau d'études soucieux de la durabilité de l'ouvrage qu'il concoit doit-il tenir compte du comportement chimique du Géotextile?

Answer by J. Lassalle:

Le Comité Francais des Géotextiles a établi des fiches-types de description des produits, où la composition chimique doit être précisée.

Par ailleurs, l'étude sur le vieillissement présentée dans une autre session du Congrès a montré que les géotextiles en polyester ou en polypropylène utilisées jusqu'à présent conservaient leurs caractéristiques après enfouissement dans la terre d'une dizaine d'années.

Cette étude a aussi exposé que la corrélation entre essai de vieillissement accéléré et comportement réel en place était plus qu'hasardeuse.

Nous considérons que, pour les géotextiles courants présents sur le marché, la stabilité chimique est satisfaisante.

Question:

All of the tests mentioned by speakers apply to woven and non-woven materials.

1) Does a knit product fall under either of these?

2) Can the same tests be applied to knit products?

Answer by J. Lassalle:

Les essais mis au point par le Comité Français des Géotextiles ont été étudiés pour que le résultat ne soit pas influencé par la structure du produit.

Pour être plus précis, la dimension retenue pour les éprouvettes est suffisamment grande pour que, quelle que soit la structure, toutes les liaisons pouvant jouer un rôle dans le comportement du produit puissent être mobilisées.

Nous pensons donc que ces essais sont auusi bien adaptés aux produits tricotés qu'aux produits tissés et non tissés.

STEWARD, J. and MOHNEY, J.
U. S. Forest Service, Oregon, USA

Trial Use Results and Experience Using Geotextiles for Low-Volume Forest Roads

Resultats des usages experimentaux des géotextiles dans les voies peu circulation des forets

Vol. 2, p. 335

Question:

1) Did you ever observe failure on the geotextile? If so: What was the type of failure; at what place was it torn?

2) Could you give tensile strengths and young's moduli of the geotextiles you examined; or could you give an approximate value of the stresses occurring on the fabric?

Answer by J. Steward:

The only failure observed on the test road was a tear in heaviest weight fabric (12 oz/yd 2 nonwoven polyester), which was torn by a large rock pushed by the dozer spreading the cover stone. The tear resulted from construction practices. None of the fabrics in the test road were torn when uncovered about one year after installation.

Question:

Has the U.S. Forest Service used the slit-film woven fabrics for road stabilization and have you noticed any improved performance or reduction in rock depth?

Answer by J. Steward:

Yes, the Forest Service has used slit-film woven fabrics for road stabilization. No one has reported improved performance or a reduction in rock depth. None of the slit-film have been uncovered for observation. It may take many installations for benefits or weakness of the slit-film woven fabrics to be observed and documented. Our roads are built by a variety of contractors, inspected by a variety of road inspectors, and involve a wide variety of soil conditions, resulting in long time periods and many observations to validate observations.

Question:

Don't you think that the ability of certain geotextiles to drain water in their plane enhances the bearing capacity of the subgrade?

Answer by J. Steward:

The ability of geotextiles to drain water in their plan is of secondary importance in most subgrade installations. Usually, the rocky material covering the geotextiles has high enough permeability to dissipate excess pore water pressures. The geotextile helps maintain the initial permeability of the aggregate cover. The ability of certain geotextiles to drain water in their plane could be beneficial if the geotextile cover material has permeability similar to that of the subgrade. In this case, the primary function of the geotextile is to drain water and relieve pore water pressures. The ability to drain water in the plane of the geotextile could be detrimental if it wicks water from edge of the roadway into the subgrade.

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Question:

1) What was the backfill material behind your fabric retaining walls?

2) With the facing being coated by bitumen, how did you take care of the drainage?

Answer by J. Steward:

1) Backfill behind all but two of the fabric retaining walls was a 2-inch (5.0 cm) maximum sized crushed aggregate with less than 8 percent passing the number 200 sieve. The other two fabric walls were backfilled with wood chip materials smaller than 2 inch (5.0 cm) in size. Wood chips were used to build light weight retaining walls on unstable slopes.

2) The granular backfill has enough permeability to carry water down to the granular bedding under the wall. Also, the fabric retains much of its permeability, especially at the contact point between layers. Also, slotted plastic tubes can be pushed into the wall face if needed.

Question:

How do you accommodate the threat of fire in fabric wall design, particularly in marginal-access areas? Can the concern be quantified to relate to the consequences of loss of use (say, particularly for logging areas)?

Answer by J. Steward:

The threat of fire at one of our wall sites is normally quite low. If the fire threat was considered high or the consequence of fire very severe, we would probably build another type of retaining wall or coat the fabric wall with gunite (a sand and cement mixture blown onto the wall face). Since the horizontal load on the face fabric walls is low, we expect that if the wall face was destroyed by fire, the wall face would ravel and the wall would not collapse. A new wall facing using wire mesh and gunite with anchorages into the wall would be a feasible method of repair.

Question:

Have any of the speakers had experience in stabilizing a loose, dry sand subgrade with a fabric? Would you make any comments?

Answer by J. Steward:

No direct experience.

Question:

Are there ideal specifications of geotextile fabrics for applications discussed? And, what are they?

Answer by J. Steward:

None that I know of.

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Question:

Do any of the speakers have any experience of road construction over saturated peat deposits? If so, can they advise as to whether geotextiles can offer any solution?

Answer by J. Steward:

The Alaska Region (Region 10) of the USDA Forest Service has successfully used geotextiles to build roads over peat deposits in muskeg terrain of South East Alaska. A report by W. Vischer in November 1975 titled "Use of Synthetic Fabrics on Muskeg Subgrades in Road Construction" is available. The report covers experience using geotextiles on several test sections over peat deposits and proposes a design method. The report can be obtained from Regional Geotechnical Engineer, USDA Forest Service, P.O. Box 1628, Juneau, Alaska 99802, USA.

RAMASWAMY, S. D. and AZIZ, M. A.
National University of Singapore, Kent Ridge, Singapore, Republic of Singapore

Jute Fabric in Road Construction

Les "tissés" jute pour la construction des routes

Vol. 2, p. 359

Question:

During World War II, military supply roads were built over soft subgrades using jute fabrics as separators in India and Burma. The major problem with the jute was that in saturated soil conditions it rotted within three months. However, because of the important nature of these military supply roads, the use of jute was considered to be beneficial. Subsequent light coatings of bitumen (a waterproofing agent) applied to the jute fabrics increased the inservice life of the jute fabrics from twelve to eighteen months. In light of the above and the statements in your paper attesting to the life of jute fabrics in soils, I would like to ask what detailed inground studies have you carried out establishing that the jute fabric you refer to lasts the twelve months you stated?

Answer by S. D. Ramaswamy and M. A. Aziz:

The authors thank Mr. Lawson for the information on the use of jute fabric to stabilize soft subgrades during World War II and appreciate his concern for the degradability of Jute Fabric. His observations on the effect of light coatings of bitumen applied to the jute fabrics increasing the life of the jute fabric to 18 months conforms with the long term observations made by the authors. However, the institu tests carried out by the authors on samples of untreated jute fabric spread on a prepared soft silty clay subgrade covered with a layer of sand and placing a bituminous subbase on the top have indicated that the jute fabric, although it loses its strength substantially, still retains its functions as a separator and a filter for a period of about 12 months. The rate of deterioration would, of course, depend upon the nature of soil. In the case of an organic soil, the fabric needs to be placed on a thin layer of sand spread over a prepared surface of subgrade.

The main purpose of the paper is to highlight the use of jute fabric for soft subgrade stabilization. As has been reported in the paper, the jute fabric initially has sufficient strength to act as a reinforcing membrane and helps in load distribution. Its nonclogging and drainage properties help stabilize the soft subgrade under construction and traffic loading. The initial strength of fabric is important and the lowering of its strength due to degradation with time will be compensated for by the improved strength of subgrade with time. By the time the fabric degrades, the subgrade would have developed sufficient strength by consolidation and the help of jute fabric would no more be needed. The drainage would still be maintained by the thin sand layer overlying the stabilized subgrade. By subjecting the jute fabric to chemical treatment, it is possible to prolong its life from rotting.

It is the contention of the authors that maintenance costs of roads built on soft subgrades could be greatly minimized by the use of jute fabric as described in the paper.

BOURDEAU, P. L.
Federal Institute of Technology, Lausanne, Switzerland
HARR, M. E. and HOLTZ, R. D.
Purdue University, West Lafayette, Indiana, USA

Soil-Fabric Interaction—An Analytical Model

Un modèle analytique d'interaction entre un sol et une membrane géotextile

Vol. 2, p. 387

Question:

You put a lot of effect on a right description of the geotextile-aggregate interaction, on one hand, while you described the subsoil rather roughly by a modulus of subgrade reaction.

1) Do you consider this an adequate or balanced approach?

2) Does your calculation method give a good description of the lateral restraint effect, which is mainly a geotextile-aggregate interaction?

Answer by Holtz:

1) The paper deals with geotextiles and their interaction with soils. Matters pertaining to such conditions, even for standard footings, are, at best today, poorly understood. Consequently, it should come as no surprise that an "equivalent soil modulus" was invoked. On the other hand, a model for the geotextile was introduced that would reflect its inherent properties and their influence on performance.

2) The analysis assumed a "plane condition (2-dimensional)" to govern. Hence, lateral restraint is afforded by the assumed "strip" being considered. The degree of restraint required for the system to perform adequately is an output of the developed model.

RAUMANN, G.
Monsanto, Decatur, Alabama, USA

Geotextiles in Unpaved Roads: Design Considerations
L'usage des géotextiles dans les pistes de chantier

Vol. 2, p. 417

Question:

By saying that high modulus fabrics are more beneficial than low moduli ones, are you saying that woven fabrics should be more suitable for this paved and unpaved roads than nonwoven ones?

Answer by G. Raumann:

Mr. Scott misunderstood what I had said in my presentation. I do not consider high modulus fabrics more beneficial in reinforcing unpaved roads -- in fact, the contrary is the case. The best overall performance of fabrics requires that they deflect significantly to allow the tensile forces in the fabric to have vertical reinforcing resultants. The best combination of fabric properties is found in fabrics with intermediate modulus. For this reason, nonwoven geotextiles are better suited than woven fabrics for this application.

For the case of paved roads, completely different criteria come into play. My paper did not intend to address the case of geotextile reinforcement in paved road structures.

Question:

Is it possible to determine the porosity function for woven or other interlaced or interlooped structures?

Answer by G. Raumann:

Porosity, as normally defined, is a bulk property of a material or structure. The porosity function linking permeability to the porosity of the structure is based on the assumption that the fiber assembly can be treated by statistical means. This requirement is not met with in the case of woven, knitted or otherwise interlaced planar assemblies. The openness (porosity?) of such structures is largely governed by the interface of the fabric surfaces and the adjacent layers, and to a much smaller degree, on internal open space. The concept of conventional porosity is thus not appropriate.

I am not aware of any theories linking woven structures with their in-plane permeability, but in the case of geotextiles, this would appear an unimportant group of structures for in-plane fluid transport.

HOARE, D. J.
University of Birmingham, Birmingham, U K

A Laboratory Study into Pumping Clay through Geotextiles under Dynamic Loading

Une étude en laboratoire du pompage de l'argile à travers des géotextiles, sous chargement dynamique.

Vol. 2, p. 423

Question:

I am confused concerning the ability of a geotextile to stop the migration of fines from the subgrade to the base. If you had silty or clayey subgrade, geotextile, sand, gravel, etc., available, what materials would you recommend and why?

Answer by D. J. Hoare:

This is a very important question, and no simple answer can be given to it since migration of fines from subgrade to subbase will depend inter alia on the particular materials, loading and environmental conditions prevailing.

Unless a "self-induced" filter system will form in the subgrade soil itself, on the basis of the laboratory tests undertaken, the most significant factor in reducing pumping will be to reduce the contact stresses between subbase and subgrade to as even a level as possible. This will prevent local deformation of the subgrade, which will help avoid softening and thus minimize the tendency for it to pass through the fabric.

Although not incorporated into the test program reported in my paper directly, my feeling is that the use of a system such as a sandblanket would be ideal and could be the only sensible solution under certain soil and loading conditions.

Question:

Figure 1 in the paper dimensions the thickness of the subbase is variable. What was the average thickness of the subbase placed before loading?

In the authors opinion, would the confining effects of the mold result in an accurate simulation of actual field conditions?

Answer by D. J. Hoare:

The thickness of subbase placed before loading varies with the types of subbase used. The criteria for the crushed rock (SB 1 and 2) was that 1.5 kg. of aggregate was used. The resulting subbase thickness was thus not recorded accurately but would be in the range 20 to 40 mm. depending on the grading. For the spherical glass balls (SB 3) a fixed number of balls in a single layer was used. This number was the maximum number of balls which could comfortably be fitted into the single layer.

It is exceedingly difficult, if not impossible, for any laboratory test to give an accurate simulation of actual field performance, and without doubt, the confining effects in this test will have an effect on the results. However, as a simple, easily repeatable classification test by which relative amounts of pumping may be investigated under a range of subbase, subgrade, fabric and loading conditions, it is felt that such a test could well become the basis of a standardized test method.

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Question:

Why do you use spheric glass balls in the base material. In practice, the particles of the subbase material should be angular. Do you think the difference in the shape of the particles has some effect in the results of your test?

How do you get the weight of the fine particles in the zone of contamination?

Answer by D. J. Hoare:

The test program reported in the paper involves the use of both angular shaped granular materials and spherical glass balls as subbase materials. The glass balls were used with the intention of ensuring that the vertical stress at each point of contact was similar, so that the best possible test repeatability could be investigated. Differences in the shape of the particles will undoubtedly have some effect on the amounts of pumping which will occur, as it will, for example, affect the stress concentrations beneath the points of contact between subgrade fabric and subbase, and it could cause local, severe out-of-plane deformations of the fabric (as it deforms around individual subbase particles) and this could affect fabric void size.

The weight of fine particles in the zone of contamination is obtained by before and after weighings of the fabric plus subbase. Thus the dry weight of the fabric and subbase to be used was determined before assembly in the mould. Then at the end of load cycling, the fabric and subbase material were lifted out, oven dried at a temperature of 60 degrees centigrade for 48 hours (to avoid any possibility of damage to the polymers) and reweighed. The overall increase in weight was taken as representing the subgrade soil, which had been pumped through the fabric.

Question:

What effect would fabric permeability EOS, porosity, etc., have on pumping?

How did you prevent pumping at the boundary of mould, as well as at the fabric soil interface?

How would soil properties influence pumping?

Answer by D. J. Hoare:

The tests outlined in the paper showed that there were no significant differences in the amount of pumping with thin meltbonded and thick needlepunched fabrics. Thus providing always that adequate permeability exists across the fabric to ensure that pavement drainage is not impaired, it could be tentatively concluded that such factors as fabric permeability, pore size and porosity are not major controlling influences on pumping under the conditions of test.

As shown in Fig. 1 in the paper, the fabric disc used in the test had an area greater than the plan area of the mould, which allowed a "turn-up" at the side of the mould of about 1-2 cms. This was held against the side of the mould by the subbase material. This measure proved satisfactory in preventing pumping at the boundary.

Soil properties have a great influence on pumping. For a low plasticity subgrade, if sufficient silt and sand particles exist in the soil a "self-induced" filter may form with time in the subgrade giving a stable structure with very limited, continued particle migration across the fabric. For a high plasticity subgrade, this will not occur and pumping could theoretically continue with time. The most important factor in controlling pumping is the prevention of the formation of a clay slurry at the fabric interface, and this will be increasingly important with higher clay content subgrades. This can best be achieved by ensuring adequate drainage of the pavement and by preventing softening of the subgrade, which can result from local deformations beneath highly stressed points of contact between subbase fabric and subgrade, as outlined elsewhere in the discussion to this Session. Other soil properties such as dispersivity have not been specifically investigated in relation to this test, either at Birmingham or (as reported by Dr. Bell) in Belfast.

PRUDON, R.
SODOCA Neuf Brisach, Courbevoie, France
RESAL, J.
Laboratoire Sols Sportifs, Ministère du Temps Libre, Paris, France

Geotextiles in the Sports Grounds

Les géotextiles dans les terrains de sports

Vol. 2, p. 453

Question:

Is the material B, with its low energy uptake, the same type of geotextile that is normally used in road construction?

Answer by R. Prudon:

Le géotextile type B est un nontissé épais fabriqué à partir de fibres synthétiques courtes aiguilletées. Sa masse surfacique est de 750 g/m^2.

De ce fait il est assez peu utilisé dans la construction de routes d'autant plus que ses résistances mécaniques à la rupture et à la déchirure sont relativement faibles.

Par contre il peut être utilisé comme filtre/drain sa perméabilité $K = 5.10^{-6}$ étant élevée.

En plus de son utilisation dans les terrains de sports il est généralement employé comme anti-poinconnant avec les géomembranes.

NEWBY, J. E.
Southern Pacific Transportation Company, San Francisco, California, USA

Southern Pacific Transportation Co. Utilization of Geotextiles in Railroad Subgrade Stabilization

Les géotextiles dans les fondations de voies ferrées de la Southern Pacific Transportation Company

Vol. 2, p. 467

Question:

1) If subgrade moisture is able to come in contact with a geotextile, why couldn't the moisture pass through the fabric and freely drain out the ballast, rather than drawing through the fabric itself. In other words, why is it a necessity that fabrics possess lateral permeability to allow for the removal of subgrade moisture?

2) If water is drained away from a wet subgrade by a fabric with lateral permeability, couldn't water also be wicked back into the subgrade through the same mechanism?

Answer by J. E. Newby:

1) The lower portion of ballast becomes contaminated due to downward intrusion of crusher rock dust, locomotive sand, wind deposited dust, dust from traffic and abrasion of rock ballast. These fine materials hinder free drainage at the base of the ballast, and it is a necessity that fabrics possess lateral permeability to allow for rapid removal of high pore-water pressures. In other words, my experiences have shown that non-lateral permeability fabrics resulted in more rapid railroad subgrade failures than where fabrics were not installed.

2) Water being wicked back into the subgrade by fabric does not concern me. Railroad roadbed is open and rainfall water will penetrate through the ballast down to the fabric. I have never removed any fabrics from the railroad subgrade that were not completely saturated with water. The very rapid release of high excessive pore water pressures, when rapid loads are applied, is the important factor to look for in the selection of a fabric. The loads squeeze out the excessive water in a lateral permeability fabric, allowing the fast relief or reduction of high water pore pressures.

Question:

"The needlepunch can drain water from beneath the ballast by a siphon action."

If it is true, it can also bring water to the subgrade from the sides if water table on the berms happen to be high.

Can you comment on it?

Answer by J. E. Newby:

Drainage of water from beneath the ballast by syphon action has little, if any, effect. In fact, I doubt this occurs to any extent in actual track conditions, since most of the subgrade problems are where heavy rainfall occurs. If the water table is high or water is standing in a ditch near the tract, the subgrade will be saturated. Capillary action in fine soils will result in excessive water in or just under the geotextile. The thick needlepunched, high denier geotextiles will allow the rapid high pore-water pressures under rapid loads to dissipate rapidly. If these pressures are not dissipated, liquification will occur resulting in failures of the subgrade.

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Question:

1) If there is a source of fines above the fabric which prevents water from beneath the fabric from flowing up and thus outward to the embankment edges, does not the source of fines (subgrade and ballast) which you have claimed is present within the thick nonwovens prevent lateral drainage through the fabric?

2) Have you recorded water flow from the geotextile edges, which can be attributed to water flow within the geotextile?

3) Is the quantity of lateral drainage through the geotextile large, in comparison to evaporation of the water within or above the geotextile due to vapor partial pressures?

Answer by J. E. Newby:

1) One of the great concerns is the penetration of fines into any geotextile, resulting in clogging or an impervious membrane. As of this date, I have not observed any needlepunched fabric failing to function due to clogging. The higher denier (1 mg/m) needlepunched fabrics have indicated they will flush out fines at a faster rate than low denier, when high pore-water pressures are applied.

2) Yes, I have noted in numerous locations water flow from the geotextile edges which can be directly attributed to water flow within a needlepunched geotextile.

3) The rapid removal of high pore-water pressures is important and the lateral drainage through the geotextile is the major benefit and not evaporation of the water above the geotextile.

Question:

Could you recommend a minimum thickness of ballast under the ties, in order to prevent the geotextile from being punched during tamping operation?

Answer by J. E. Newby:

The minimum thickness of combination ballast and subballast should be 30.5 cm (12 inches) to prevent the geotextile from being punched during the tamping operation. Southern Pacific Transportation Company have tamped ballast with only 15.2 cm (6 inches) in thickness, but this requires special setting of the tamping feet and well-trained operators to prevent puncture of the geotextile.

Question:

How does initial modulus of the geotextile correlate with long term railroad subgrade stabilization performance?

Answer by J. E. Newby:

The initial elongation of the geotextile should be high to prevent puncture during installation, but the modulus should be high after the initial elongation.

Vol. 2, p. 467

Question:

In your experience, were there cases where geotextiles applications were ineffective? If so, what were the reasons?

Answer by J. E. Newby:

Geotextiles do not have strength to retain a train load, and failures will occur if good engineering design is not applied. I do not know of a case where it can be determined that a geotextile application did not have any effect. I have seen failures when geotextiles were used because the design was not adequate for the subgrade conditions. These failures could have been prevented by using a combination of geotextile and subballast. The weight of geotextile and the thickness of the subballast will depend on the type of subgrade soil, type of subballast, frequency and duration of rainfall and the traffic tonnage.

Question:

How much money has Southern Pacific Transportation Co. saved by using 1000 miles of geotextiles?

Answer by J. E. Newby:

Obviously a definite answer cannot be given. Our experience has proved that the savings in maintenance can more than justify the cost and installation of fabric within a short period of time.

Question:

You mentioned the effect of diesel fuel on polypropylene. What effect does diesel fuel have on polypropylene? If there is an effect, how serious do you feel it is?

Answer by J. E. Newby:

Diesel fuel will cause polypropylene to balloon or expand resulting in loss of tensile strength and other functions.

This may not be serious in normal track, due to the low amount of diesel fuel, but it could be serious in yards, particularly near fueling facilities.

Question:

1) How high, on the average, is the section of ballast, as shown in Fig. 10?

2) I suppose that settlement of weak subgrades is not only originated from pumping, but also, drained creep or consolidation due to cyclic loading. What is your opinion about this? Which did more to contribute to settlement, pumping or drained creep?

Answer by J. E. Newby:

1) The height of the ballast, as shown in Fig. 10, should be about 30.5 cms (12 inches) below base of cross tie.

2) Cyclic loading will result in a permanent deformation under each rail, and water will collect in these deformations, resulting in a pumping action if a well-graded filter or a geotextile is not used. The amount of settlements is not of great importance in railroad subgrade, but differential settlements in local spots can be important.

   Pumping normally occurs in local spots and results in the greatest problem.

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Question:

In your paper, you champion the cause of thick needlepunched geotextiles in railway applications, because of their ability to drain in the plane of the fabric.

In Session 5B, three papers were presented on the ability of geotextiles to prevent migration of "pumping" soils. The results found suggested that all geotextiles, including thick felts, are susceptible to particle migration across their boundaries when subjected to dynamic loading. Another paper at this conference (Heerten) found that thick felts retained soil at approximately 8 times its original unit weight when subjected to dynamic hydraulic effects.

With this in mind, then it is highly unlikely that any geotextile would be able to drain in its own plane. More than likely, excess pool pressures would dissipate in the ballast layer above -- not along the plane of the geotextile.

However, you apparently disagree with this, so I would like to know what tests have you done to establish the in-plane draining capacity of thick geotextiles when subjected to dynamic loads that cause pumping.

Answer by J. E. Newby:

I have not performed any laboratory tests, nor have I seen or read of any laboratory tests using geotextiles that will simulate actual railroad field conditions. It is agreed that very fine soil particles will migrate across the boundaries of a needle punched geotextile fabric, and the fabric will retain some of these fine soils. However, I have not experienced any of the needle punched geotextiles becoming clogged, resulting in loss of in-plane drainage. The higher denier geotextiles appear to allow the flushing of fine soils better than the lower denier geotextiles.

Pumping action under dynamic loads occurs in a lateral plane just under an impervious membrane.

Question:

1) What is, in your opinion, the maximum deflexion of a well designed railroad?

2) May any decrease in deflexion due to reinforcement (not separation) be expected using a geotextile?

Answer by J. E. Newby:

1) Deflexion of a well designed railroad should be between 3 and 5 cms., and the majority of the deflection should be within the track structure.

2) No decrease in deflection can be expected by using geotextile in a well designed railroad, but will be decreased in a poor subgrade condition, due to tensile strength of the geotextile.

RAYMOND, G.
Queen's University, Kingston, Canada

Geotextiles for Railroad Bed Rehabilitation
Les géotextiles pour la rehabilitation de base du chemin de fer

Vol. 2, p. 479

Question:

What is, in your opinion, the maximum deflexion of a well designed railroad?

Answer by G. P. Raymond:

Track deflexions of a recently tamped mainline track where low ballast voiding is anticipated should range between 2 mm and 6 mm. If track is too stiff, rail damage occurs from the impact of flat wheels, and the like, while if track is too soft, movements cause abrasion of the ballast, resulting in more frequent maintenance raises.

Question:

May any decrease in deflexion due to reinforcement (not separation) be expected using a geotextile?

Answer by G. P. Raymond:

It is unlikely that geotextiles would improve the deflexions measured from static loading. The effect of geotextile reinforcement is more likely to be apparent in the measurement of plastic deformations resulting from numerous repetitions of wheel loading. Assuming a 50 mm plastic deformation between track maintenance cycles, any geotextile reinforcement would result in increased maintenance life.

Question:

If subgrade moisture is able to come in contact with a geotextile, why could not the moisture pass through the fabric and freely drain out of the ballast rather than drawing through the fabric itself. In other words, why is it a necessity that fabrics possess lateral permeability to allow the removal of subgrade moisture.

Answer by G. P. Raymond:

Moisture moves upwards through the soil profile due to the pore pressures generated during the passage of trains and downward during heavy rainstorms when there are no trains passing. Thus fines can be quickly accumulated on both sides of the fabric. Under such an environment, it was essential that the fabric be able to pass water laterally.

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Question:

If water is drained away from a wet subgrade by a fabric with lateral permeability, could not water also be wicked back into the subgrade through the same mechanism?

Answer by G. P. Raymond:

Yes. This is why such attention has been paid in my presentation to the installation of geotextiles and why, on Figure 9 of my paper, a minimum slope of 48 to 1 is suggested to allow the track bearing area to drain into a side trench with perforated pipe, which is permitted to discharge into the side ditches. Wicking requires free standing water. This cannot oocur if the geotextile is installed as shown in my Figure 9.

Question:

Don't you think that size and amount of the "opening sizes" (as producer, we hesitate to use the word holes) in the geotextiles could be mainly a result of the specific Canadian weather conditions in winter (especially ice and frost)?

Answer by G. P. Raymond:

No. Our study, while extensively in Canada, included sites in the USA, some of which were not subject to any frost action. The larger holes observed in the geotextiles were due to excessive wear and the smaller holes, such as seen in Figure 5 of my paper, are from internal abrasion of small particles against the individual fibers.

Question:

In your paper, you champion the cause of thick needlepunched geotextiles in railway applications because of their ability to drain in the plane of the fabric.

In Session 5B three papers were presented on the ability of geotextiles to prevent migration of "pumping" soils. The results found suggested that all geotextiles, including thick felts, are susceptible to particle migration across their boundaries when subjected to dynamic loading. Another paper at this conference (Heerten) found that thick felts retained soil at approximately 8 times its original unit weight when subjected to dynamic hydraulic effects.

With this in mind, then it is highly unlikely that any geotextile would be able to drain in its own plane. More than likely, excess pore pressures would dissipate in the ballast layer above -- not along the plane of the geotextile.

However, you apparently disagree with this, so I would like to know what tests have you done to establish the in-plane draining capacity of thick geotextiles when subjected to dynamic loads that cause pumping.

Answer by G. P. Raymond:

Our findings relate the amount of particles retained in the geotextile to the ability of the particles to penetrate the geotextiles which is related to the equivalent opening size and the amount of entanglement of one fiber with another. The loss of lateral permeability can be realted to the degree of geotextile fouling. Thus we find that low equivalent opening size geotextiles retain their intitial permeability better than those with high (physical size or micron) equivalent opening sizes.

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Question:

You advocate resin coated geotextiles. Are you aware that tests on Colbond P 450 fabric (polyester fiber, acrylate resin binder) shows that it allows silts and clays to pass easily in the same manner as all other geotextiles when subject to dynamic loading. Weight 450 g/m^2, fiber diameter 20 um.

Answer by G. P. Raymond:

I am not aware of the geotextile Colbond P 450. At 450 g/m^2, the geotextile is on the light side. You don't state the equivalent opening size of this material, which may be quite large even when the fiber size is small. If the equivalent opening size is large, then it is obvious why the geotextile is unable to prevent silts and clays from passing. We specify that a geotextile will be unable to pass particles of 40 microns or a No. 400 U.S. sieve. Furthermore, the geotextile should be unable to trap particles of 150 micron size or No. 100 U.S. sieve. It also helps to use a layered construction of the geotextile with each new layer needled to the adjoining layer with at least 80 penetrations per square centimeter (500 penetrations per square inch).

Question:

Have you done any work to determine whether the weight of a geotextile should be increased in relation to the amount and type of traffic on the section of trackage?

Answer by G. P. Raymond:

All our work has been related to mainline track carrying 30 metric tonne axle loads and at least 10 million metric tonnes per annum. For tracks carrying this amount of tonnage, it is generally relatively easy to show the cost effectiveness of using a heavy weight geotextile.

Question:

How does resin effect the planar permeability and elongation? You recommend a minimum 28 oz. fabric. What is the tenacity of the fibers?

Answer by G. P. Raymond:

I recommend the use of a low modulus latex based acrylic resin which increases the weight of the untreated geotextile by about 20% but not less than 12%. The low modulus resin leaves the geotextile in a relatively flexible condition, but will add some stiffness, thus reducing its elongation characteristics. The recommended tenacity of the fiber is not less than 3.8 gramm per dennier tested after soaking (i.e., ASTM-D2256 Option 2).

Question:

In the cut areas, have you used or recommended the installation of geotextile covered drainage tubing placed in the subgrade?

If so, how deep below the geotextile support sheet would be recommended for the drain?

Answer by G. P. Raymond:

Ideally this question should be answered by Mr. Newby. Our recommended practices for perforated pipe is shown in Figure 9 of my paper.

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Question:

What caused the film of water over the BIDIM C-38?

Answer by G. P. Raymond:

The cause of the ponding at the BIDIM C-38 site given in my presentation, but not included in my paper, was due to the lack of attention to a drainage slope during the preparation of the undercutting. Such practice or lack of it is referred to in my paper and illustrated in Figure 9 of the paper.

Question:

What was the thickness of the ballast between the bottom of the crosstie and the fabrics in the Conrail Loudonville & Salem Ohio Test?

Answer by G. P. Raymond:

The geotextiles at Loudonville were ideally installed at a depth of 250 mm below the ties. During the excavations at 18 months (i.e., May 1981) the recorded depths varied from 175 mm to 203 mm. Thirty months after installation (May 1982) this depth had decreased to 150 mm to 190 mm. Settlement records on the rail show that at least 50 mm of settlement has occurred over parts of the installation. At Salem the geotextile was supposed to be installed at a depth of 200 mm below the ties. Six months after installation the geotextiles were excavated at between 127 mm to 203 mm. Eighteen months after installation the excavation depths were between 127 to 178 mm. Clearly, the installation control at Salem was not as good as at Loudonville.

Question:

You recommend a small EOS fabric, yet Mr. Newby recommends 9 denier filaments to open the structure and facilitate flow. Would you comment on this recommendation, and, perhaps, state your denier recommendation?

Answer by G. P. Raymond:

Our experience is that the higher the equivalent opening size, the greater the fouling of the fabric, and the greater and more rapid is the reduction in permeability of the geotextile. In order to obtain a low equivalent opening size, you need to use a low denier filament. My own specifications states that the filament size will not be greater than 7 denier. This allows the geotextile manufacturer to use any filament size less than 7 deniers, but personally, I prefer the smallest denier size available. If this is closely needled, then a low equivalent opening size geotextile generally results.

Question:

What is effect of ballast size on choice of geotextile?

Answer by G. P. Raymond:

Generally, the larger the ballast particles, the greater the abrasion to the geotextile for the same amount of traffic. Thus some railroads will put a thin layer of small size ballast particles on the geotextile first, completing the rehabilitation with more normal 40 mm size ballast particles.

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Question:

Your conclusions indicate performance in puncture resistance and abrasion resistance improve to an apparently acceptable level as fabric weight increases, or the fabric is subjected to an acrylic resin treatment.

It is my understanding that a relatively lightweight needle punched polypropyne fabric performed very well at the F.A.S.T. track in Colorado. Are you aware of these results, and do you have a comment if you are aware of the results?

Answer by G. P. Raymond:

The F.A.S.T. report of March 1982, which includes a section on geotextiles, indicates that all the fabrics have suffered from ballast wear involving completely worn through holes. The report concludes by stating that further tests should be undertaken with geotextile weights of 15 oz/sq.yd. and higher. This would appear to be in direct contradiction to your statement.

Question:

You commented that there should be more attention given to the ability of a fabric to drain and separate than to contribute to support. You stated the fabrics in the Conrail Test (?) had large holes after six months to the present. With such large holes in the fabrics relative to the fines and ballast, how does one qualify the extreme detailed attention paid to EOS of the fabrics?

Answer by G. P. Raymond:

To be cost effective and to maintain longevity of the rehabilitation, the geotextile must remain free of, not only large holes, but small holes, as well. Thus we pay great attention to the ability of the geotextile to withstand the abrasive resistance of the track.

BUTTON, J. W., EPPS, J. A., and LYTTON, R. L.
Texas Transportation Institute, Texas A & M University, Texas, USA
HARMON, W. S.
Celanese Fibers Marketing Company, North Carolina, USA

Fabric Interlayer for Pavement Overlays

Textile interposé entre revêtement ancien et nouveau

Vol. 2, p. 523

Question:

What were the specific fabrics used in the tests, and on what basis were they selected?

Please comment on the use of woven vs. nonwoven fabrics in the area of fabric reinforced pavement.

Answer by J. W. Button:

Commercially produced fabrics, as well as experimental prototypes, were used in the test program. Construction consisted of woven and nonwoven, continuous filament and stable fibers, polypropylene and polyester and some unique combinations. Fabrics were selected to evaluate the leading commercially produced products and to investigate a wide range of fabric properties.

Commercially available woven fabrics generally hold less asphalt than the nonwoven fabrics, which appears to be a disadvantage. Their surfaces are smoother and, therefore, more susceptible to slippage at the interface. The strength and modulus of woven fabrics are highest in the warp and weft directions, which appears to be advantageous from a pavement reinforcement standpoint.

Question:

You reported that shrinkage of some fabrics, associated with high temperatures of newly placed asphalt concrete, can cause premature cracking of the overlay. Which fabrics, or which fabric groups, show that behavior?

Answer by J. W. Button:

In the limited study of this phenomenon, no fabric of a particular material or type of construction consistently exhibited cracking in the test frame. It was evident, however, that a simple measure of free shrinkage or shrinkage force upon exposure of the fabric to heat did correlate quite well to the cracking behavior. Fabrics with free linear shrinkage in excess of about seven percent and/or shrinkage forces in excess of about 100 grams may cause cracking along wrinkles or joints in the fabric during construction.

Question:

In the economic evaluation of geotextile in pavements, did you consider the benefits of providing a water seal?

Answer by J. W. Button:

Yes, however, there is not a sufficient quantity of long term data available to perform a detailed economic analysis of fabrics in asphalt concrete overlays. Therefore, only short term data were used to make a few general statements about field performance. Based on laboratory test results, the fabric/asphalt interlayer will remain intact even after significant overlay cracking and, most likely, reduce the flow of surface water through the system.

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INGLES, O. G.
University of New South Wales, Kensington, Australia

Long Term Performance of MESL Road Sections in Australia

Essais de comportement de longue durée des chaussées MESL en Australie

Vol. 2, p. 535

Question:

One of your figures showed a decrease in the soil moisture content with time inside the MESL.

Is this typical? How do you account for this decrease in moisture content with time?

Can we expect this to occur if the wetting pressure is low enough?

Answer by C. R. Lawson:

Where external fluctuating moisture conditions occur, it is quite common to observe fluctuating moisture in the MESL. Remember the membrane is not impermeable. The reason for the decrease in moisture in the MESL is because the vapor pressure outside the MESL is less than that inside, so moisture migrates out of the MESL.

I think you may be a little confused on the subject of what is wetting pressure. Wetting pressure only determines whether there is liquid or vapor flow through the membrane. If the wetting pressure of the membrane was lower than the soil water head, then liquid water would pass through the membrane, which in this case, would defeat the purpose of the encapsulating membrane. The wetting pressure of the membrane should be always greater than the external water pressure, to ensure vapor flow only occurs. Under these conditions, the permeance of the membrane governs the rate at which moisture vapor is transferred across the boundaries of the membrane. For membranes exhibiting very low permeance, very little moisture fluctuaton would occur in the MESL.

Question:

1) How to design the thickness of the layer encapsulated by the membrane.

2) Is the capping layer always necessary? How to design it? What is it composed of (kind of material)?

Answer by C. R. Lawson:

1) The thickness of the MESL can be determined using conventional design curves. The difference between an MESL design and a conventional design is that unsaturated soil parameters can be used for the MESL design, whereas, saturated soil parameters are used for the conventional design. Herein lies the major benefit of the MESL system.

2) At the time of construction (1974) little was known of the resistance of the geotextile to traffic stresses. To be conservative, we included a capping layer of fine crushed rock to ensure protection of the upper membrane layer. Subsequent events showed that this capping layer was generally unnecessary.

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Question:

Please specify types and properties of geotextiles used in your MESL pavement?

Answer by C. R. Lawson:

The type of geotextile used in the MESL pavements at that time went under the trade name of Terram 140, which was manufactured by ICI Fibres, UK. It has since been superseded by Terram 1000A.

The properties of Terram 140 were:

Type	-- Melt Bonded Heterofilament nonwoven
Polymers	-- 75% polypropylene, 25% nylon
Geotextile Weight	-- 140 g/m^2
Grab Tensile Strength (ASTM D1682)	--600 Newtons
Grab Extension At Break	-- 120%
Wing Tear (BS 4303)	-- 200 Newtons
Indicative Pore Size(O_{50})	-- 0.1 mm

Question:

Do you ever anticipate problems with the buildup of hydrostatic pressure underneath the impermeable MESL caused by rapid increase in the height of the water table?

Answer by C. R. Lawson:

No. The pavement layers have been designed on the basis of a saturated subgrade strength parameter (CBR). The subgrade, which consists of clay, is relatively impermeable, and so a rapid increase in the height of the water table would not occur.

RAUMANN, G.
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Outdoor Exposure Tests of Geotextiles
Expériences d'exposition à la lumière de divers géotextiles

Vol. 2, p. 541

Question:

Mrs. Raumann, have you carried out any research on diffuse ultraviolet effects on various plastics? I am referring to the use of geotextiles under Gabion mattresses, rock armour, etc.

Answer by G. Raumann:

We made no attempt to shield our samples from the total daily sunshine: either by selecting morning vs. afternoon sunshine or diffuse vs. full exposure. In fact, we chose full southerly orientation and 45° angles in order to obtain the most severe exposure possible at each location.

Diffuse light: i.e., light partly blocked and partly reflected light can have different compositions of wavelength than the original source, depending on the composition, color, and smoothness of the reflecting surface (i.e., the rocks in your examples) and the variables of such testing were too numberous for us to consider.

Question:

Did you carry out laboratory tests to compare results with the ones obtained outdoors? If so, did you find some correlation? If not, what do you propose as a laboratory test that would be suitable?

Answer by G. Raumann:

At the start of our testing series, we ran several series of tests in a Xenon Arc tester. These tests gave good agreements with outdoor tests in showing the general shape of the degradation curve with time of exposure, but we found little use in

(a) predicting outdoor performance at a particular location or number of sun hours exposed

(b) relating the performance of different fabrics when exposed outdoors.

Our tests may have shown high variability because we were using an older model Xenon tester. However, even the latest models are difficult to recommend as laboratory testers for geotextiles, because at this stage, we have insufficient knowledge of how rain, snow, humidity, ozone, etc., affect the physical and chemical degradation processes of exposed geotextiles.

More studies are needed to be able to make sound recommendations for laboratory testing procedures.

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Question:

Was there any significant evidence of filament diameter effect on durability that is in addition to the differences evidenced by the woven monofilament?

Answer by G. Raumann:

We did not carry out tests on fabrics of different fiber diameter, but otherwise similar construction parameters. Some of the fabrics tested had larger diameters, but no conclusions can be drawn when other variables are also changed. Observations of polypropylene ribbons and fibers all showed light attack producing cracks and fissures from the surface. I feel confident that larger diameter fibers will show better resistance to degradation, and I attribute the better performance of woven monofils over slit film fabrics to this factor.

Question:

Do the researchers have any information regarding effects of radioactivity on durability of geotextiles?

Answer by G. Raumann:

We have not carried out any experiments on the effect of radioactivity on the durability of geotextiles. Based on general considerations: radiation will tend to crosslink polymer chains. While small amounts may increase the strength (similar to the effect of small amounts of light on some of the polypropylenes tested), large amounts are almost certain to embrittle and cause loss of strength.

I am not aware that this effect has been studied for geotextiles, but I am certain the effect of radioactivity on polymers and other textiles has been studied.

Question:

Did any of the fabrics tested, belonging to the polypropylene needle punched group, have carbon black as an additive in addition to the UV inhibitors? If so, was there any apparent improvement in their performance over similar fabrics, not containing carbon black? Would you care to comment on Xenotest or other accelerated test compared to exposure as per your test procedure.

Answer by G. Raumann:

We did not carry out detailed chemical analysis of the fabrics tested; hence we have no information about which of the fabrics had added UV inhibitors. However, all the black needled polypropylenes tested performed very poorly, and presumably, had no added UV inhibitors. So our test materials cannot answer your question. I believe some work on combined carbon black and other inhibitors has been carried out in the Netherlands on woven polypropylene geotextiles.

Your second question has, in part, been answered by my reply to M. Rigo; however, I would like to comment on the subject of "accelerated tests." We found that in a sunny climate, it is at least as quick to expose a large number of good sized specimens on our door racks, than it is to have to reduce the number and size of test specimens on the limited area of the indoor tester.

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Question:

What is the present position of acrylics in geotextiles -- with regard specifically to their good UV resistance properties?

Answer by G. Raumann:

The questioner is obviously aware of the very high light stability of acrylics when compared to other textile polymers. Acrylics have been used in the past for sand bags, silt fences and similar applications. However, they have not found a permanent place in the geotextile market for three reasons:

(1) Lower strength

(2) Method of manufacture does not lend itself easily for one step nonwoven production

(3) Higher price

A significant change in any one of these factors may again bring these polymers into the running as a geotextile product.

Question:

Can more details be given, please, of the 3 test locations and their respective test conditions.

(1) What was the latitude and elevation above sea level of the 3 locations?

(2) At what time of the year were the tests started?

(3) What are the respective typical weather conditions (rain or dry, hot or cold, etc.).

This information would assist those engineers and users who are not familiar with USA geography.

Answer by G. Raumann:

The details of the three test locations are shown in the following table (none were close to industrial polution sources):

LOCATION	COORDINATES	ALTITUDE ASL	CLIMATE, AVERAGE TEMP.
Florida Near Miami	26°N, 80°W	0 m	Subtropical: Humid Small temperature variation
Arizona	34°N, 112°W	350 m	Desert: Dry Large temperature variation
North Carolina	36°N, 79°W	180 m	Continental: Humid Summers cooler than Florida

We carried out several test runs and they were started at different times of the year and most were carried on for a year (if the samples survived that long). In each series the same needled polyester fabric was included as a control sample.

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Geotextiles and Aging Tests

Les géotextiles et les tests de vieillissement accéléré

Vol. 2, p. 559

Question:

M. Sotton a mis en evidence l'importance de l'exposition volontaire on accidentelle a la lumiere dans la chute des resistances du geotextile, et des fibres. Peut on entrevoir des solutions industrielles, (a l'exclusion des precautions de chantier) pour parer a ce phenomene.

Answer by M. Sotton:

Qui, il serait possible d'améliorer la résistance des géotextiles à la lumière, en utilisant (dans la formulation des polymères) des barrières stabilisantes (anti-oxydant - anti-UV) plus efficaces et surtout en augmentant la quantité de ces additifs dans les fibres.

La solution est connue, sans pour autant garantir des durées de vie supérieures à plusieurs mois, mais elle s'accompagne d'une augmentation nette des coûts du produit: elle pourrait être retenue sur spécification.

Question:

Do the researchers have any information regarding effects of radioactivity on durability of geotextiles?

Answer by M. Sotton:

A ma connaissance aucune recherche spécifique n'a été jusqu'ici entreprise dans ce sens sur les géotextiles.

Néanmoins beaucoup de travaux ont été conduits pour étudier l'effet des rayonnements radio - actifs sur polymere et sur fibres.

Là encore, les résultats sont fonction des doses recues . . .

Une règle veut que tous polymères, comportant dans leur structure des carbones asymétriques (polypropylène) soient dégradés (doses supérieures à 1 Mégarad), alors que les polymères comportant des carbones symétriques (polyéthylène) se réticulent sous rayonnement et résistent beaucoup mieux.

Dans ces usages le verre devrait pouvoir constituer un bon matériau.

JOHN, N.
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JOHNSON, P. and RITSON, R.
Portsmouth Polytechnic, Portsmouth, UK
PETLEY, D.
University of Warwick, Warwick, UK

Behavior of Fabric Reinforced Soil Walls

Comportement des murs en terre armée avec des géotextiles

Vol. 3., p. 569

Question:

As you have mentioned that woven fabrics may be used for reinforced soil walls if their stress/strain behaviour and creep values are acceptable, I would like to know what basic material "Paraweb" is made of and what its creep values are.

Answer by N. W. John:

Paraweb consists of parellel cores of densely packed high tenacity Terylene (polyester) filaments embedded in an Alkathene (polyethylene) sheath. The same product is also made in a rope shaped version called Parafil. Tests on 3.5 tonne Parafil loaded to 40% of its rupture strength (1) showed an initial extension of 3.59% followed by creep of 0.45% in the first three days, and 0.25% during the next four years. Comparable figures for a woven polyester sheet are 5.7%, 1% and 1.7% respectively (2). The creep occurring in the first three days should not affect the structures appearance since this falls within the construction period and can easily be corrected.

(1) Anon. "Parafil ropes for underwater use," ICI Linear Composites Ltd., Harrogate, 1-7-1980.

(2) Risseuw, P. "Long term mechanical properties of Stabilenka Reinforcing Fabrics", Enka Technical Note IEN 9427, Arnhem, November 1981.

Question:

What kind of theory was used for obtaining the theoretical load on Fig. 6?

Answer by N. W. John:

The theoretical line in Figure 6 was obtained using the D.O.E. design method(3). This assumes an active earth pressure distribution and makes no allowance for compaction effects. The additional load due to the overturning moment from the backfill has not been included in the theoretical load since land reclamation behind the Jersey wall is not yet complete.

(3) Department of Environment (U.K.), Reinforced Earth Retaining Walls and Bridge Abutments for Embankments, Technical Memorandum (Bridges) BE 3/78, London, 1978.

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Question:

What is your idea of improving the theory which should be used for designing retaining walls of this kind?

Answer by N. W. John:

Current design methods do not take into account the effects of wall deformation on stress distribution (4). Since there are similarities between the deformation of reinforced soil walls and strutted excavations it seems logical to use the same earth pressure distribution. In both the Southampton and Jersey structures the reinforcement loads did not decrease at high tide as Archimedes Principle might suggest. Once the reinforcement becomes fully strained at low tide it is impossible for it to shed any significant load during high tide since this would require the reinforcement strain to decrease and the facing panels to move inwards. The design method for tidal conditions therefore needs some modification.

(4) Wilum, Z., and Staryewski K., "Soil Mechanics and Foundation Engineering, Volume 2, 2nd Edition, Surrey University Press, 1975, pp. 126-128.

Question:

Why were the 3 layers of gravel rejects separated by a geotextile?

Answer by N. W. John:

The geotextile layers reinforce the gravel rejects to form a raft foundation which is intended to reduce differential settlement due to the weak subsoil. It also provides a firm working base which makes construction easier.

Question:

What is the contribution of the Geotextile "Terram" in this design?

Answer by N. W. John:

The sheets of "Terram" in the Websol system are intended to control the forces induced by compaction. At low fill levels the compaction can cause strains of about 5% in the geotextiles (5). At this stage the weight of soil above the geotextiles is low and it is thought that the facing panels of paraweb strips move forward and thereby relieve themselves of the compaction load. Since the surface area of the Terram sheet is about four times greater than the Paraweb strips, the sheet material is unable to slip and therefore probably carries almost all the compaction induced loads.

(5) DuBois, D.D., Bell, A.L. and Snaith, M.S. "A fabric reinforced trial embankment," The Quarterly Journal of Engineering Geology, Vol. 15, No. 3, 1982, pp. 217-225.

Vol. 3 , p. 569

Question:

In light of the November 1981 decision by the High Courts of Patents in England which found that the use of Paraweb strips in retaining structures infringes on H. Vidal patents in Reinforced Earth, would you comment on:

1. The future of this system.

2. The purpose of the Terram sheet since it is placed directly over the Parawebs strip and is not connected to the facing.

Answer by N. W. John:

1. Fortunately my livelihood is not dependent upon the success of either system of soil reinforcement, and I therefore, hope that my comments are impartial. Since it has been implied that the Websol system has little future because of the patent situation, I should point out that there are many countries where H. Vidals patents have not been registered. A good example is the Channel Islands where the large wall referred to in my paper was built. Even in those countries where patents are registered some of these are now very close to their expiry date.

2. As indicated in my answer to an earlier question the fact that there is no mechanical bond between the sheet geotextile and either the facing or the Paraweb strip can be useful. It is also interesting to note that as the depth of fill increases the friction bond between these materials will effectively link them together. If the non-woven Terram were replaced by a high modulus geotextile this friction bond could ensure that the sheet geotextile carried a significant proportion of the earth pressure load.

FUKUOKA, M. and IMAMURA, Y.
Science University of Tokyo, Noda City 278, Japan

Fabric Retaining Walls

Mur de soutement de géotextile

Vol. 3, p. 575

Question:

Would it be possible to get a more detailed description of the Fabrics used. What analysis had lead you to the 5-10 years life expectancy of the exposed fabrics?

Answer by M. Fukuoka:

HDPE fabrics without weather resistance were used. Strengths in kN/m were as follows:

	1980 (initial)	1981	1982
Sheet type	73	60	(30)
Net type	61	56	(53)

Cohesive soil was used as backfill, and its cohesion height was more than 5 m. Expansion of the compacted cohesive soil seemed to be negligible small. Therefore, it would be sufficient for the fabrics to serve as a surface protection against erosion by rainfall. This was the reason why I guessed the life would be 5-10 years. Strengths after 2 years are listed in the table for reference.

JONES, C. J. F. P.
West Yorkshire Metropolitan County Council, Wakefield, West Yorks, UK

Practical Construction Techniques for Retaining Structures Using Fabrics and Geogrids

Techniques pratiques de construction pour les structures de soutement, utilisant des textiles et des géogrids

Vol. 3, p. 581

Question:

What fabric was used in Figure 5/6?

Answer by C.J.F.P. Jones:

Material used in Fig 5.6 of the paper was ICI TERRAM RF12.

INGOLD, T. S.
Geotextile Consultants Ltd., Boreham Wood, UK and Queens University, Belfast, UK
MILLER, K. S.
Geotextile Consultants Ltd., Boreham Wood, UK

The Behavior of Geotextile Reinforced Clay Subject to Undrained Loading

Le comportement de l'argile renforcée au géotextile soumise à une charge non asséchée

Vol. 3 , p. 593

Question:

What are the scaling effects of using full scale inclusion in small plane strain tests?

Answer by T. S. Ingold:

The most significant scaling effect relates to the apparent adhesion factor α . This inturn appears to be a function of the axial stiffness of the inclusion which is of course a function of length. This notion was borne out by the results of pull-out and shear box tests on the plastic geogrid referred to in the paper. In the pull-out test, which takes samples up to 500 mm long, the average apparent adhesion factor was 0.18. Direct shear tests using a 60 mm x 60 mm shear box rendered an average value of 0.89. In the case of the pull-out test this phenomenon almost certainly stems from interpretation of the test results which should take into account the fact that the distribution of relative strain and consequently mobilized adhesion along the comparatively extensible inclusion is far from uniform.

Question:

Since the reinforcing effect of a geotextile is dependent on the angle between reinforcement layer and principle stresses, the multiplier to get from c_u to c_u' must be different for each structure. Do you have an impression of the sensitivity of this multiplier with regard to the type of structure i.o.w. Are there many different multipliers of just a few?

Answer by T. S. Ingold:

The multiplier will be different for each genus of structure thus in principle one dimensionless multiplier might relate to foundations and another to walls. For a given genus the multiplier is only the same if the relative geometry of the system is the same. Thus in a foundation system the multiplier would be different if the depth to the top layer of reinforcement, as a fraction of the foundation breadth, varied from one system to another. If the equivalent undrained shear strength notion was to be adopted as a design aid then it would be desirable to first determine the most efficient geometry for each genus in which case a unique multiplier could be used for each genus. As regards sensitivity the same loaded width in a wall system and a foundation system would, for a small reinforcement spacing, render a wall multiplier twice as large as that for a comparable foundation system. One point that should be remembered is that the theory assumes the reinforcement to be inextensible. The verity of this assumption would diminish as the absolute size of the structure increases since for a given geotextile axial stiffness would tend to decrease.

GRAY, D. H.
University of Michigan, Ann Arbor, MI, USA
ATHANASOPOULOS, G.
University of Patras, Greece
OHASHI, H.
Honshu-Shikoku Bridge Authority, Japan

Internal/External Fabric Reinforcement of Sand

Renforcement interne et externe de sable avec les textiles

Vol. 3 , p. 611

Question:

How do you propose to install the encapsulated reinforced columns in soft clay?

Answer by D. H. Gray:

The encapsulated-reinforced columns could be installed in much the same way that sand drains have been installed in the past, i.e., by using a tubular, hollow mandrel. The fabric is woven in the shape of a sock which is slipped over the mandrel. The mandrel and fabric sock are then pushed into the soft clay and backfilled with sand. A one-way trap door in the bottom of the mandrel allows the mandrel to be withdrawn whilst leaving the sand behind. After the mandrel is withdrawn the sand remains in place surrounded by an encapsulating, fabric sock.

Question:

Would you be willing to scale up to larger size specimens or columns such as bridge piers or abutments?

Answer by D. H. Gray:

The load-deformation tests on encapsulated-internally reinforced sand were run on relatively small, cylindrical samples in the laboratory. Accordingly, the reported results are indicative only of possibilities for full size structural columns. As a practical matter one could not use such small spacings between horizontal, internal reinforcements in a full size prototype. It would be important, therefore, to conduct tests on larger size columns and determine, among other things, the minimum spacing and optimal placement of internal reinforcements for significant increase in load bearing capacity.

Vol. 3 , p. 611

Question:

1. What are the physical characteristics of the sand used?

2. Are the relationships given true for different types of sands?

3. If so, do you have any data on calcareous sands?

Answer by D. H. Gray:

The sand used in the load-deformation tests on encapsulated-internally reinforced samples was a uniform, medium fine beach sand in a dense condition. The results reported in the paper are probably representative for most sands provided they are in a dense condition. We have no data on calcareous sands, however, the mineralogy of the sand would not be as important a factor as differences in relative density, gradation, and particle shape.

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A Rapid Banking Method Using the Resinous Mesh on a Soft Reclaimed Land

Méthode de remblayage rapide utilisant le filet résineux sur le terrain mou remblayé

Vol. 3, p. 635

Question:

What are the mechanical and hydraulic characteristics of the Geotextile used?

Answer by K. Yasuhara:

Mechanical properties of net-mesh*

Width of net	2.0 m
Length of net	30 m
Pitch of mesh (Longitude)	66 mm
(Latitude)	41 mm
Tensile Strength	
(Longitudinal direction)	660 kg/m
(Latitudinal direction)	420 kg/m

(*The material is called Netlon, TRX-166MR.)

REMARK: In our country, we do not have a custom to open the name of the brand in the academic paper.

Question:

The author spoke little about selection criteria and dimensioning of the Geotextiles used. Please give explanations or details on this subject.

Answer by K. Yasuhara:

In our case, the fill material will close gas, settle and spread limitlessly without restraining layer between the mesh and the soils. The restraining layer gives rise to the frictional force and resultantly gives the more eminent tensile strength. We suppose the woven textile does not play this role so much compared with the unwoven textile. In other words, our conclusion is that for embankment on soft grounds the unwoven fabric is better than the woven fabric because the frictional force can work more efficiently between the mesh and the soils.

Vol. 3, p. 635

Question:

Was any attempt made to anchor the edges of the resinous web? Slides indicated that edges were left free to move since they were uncovered.

Also, what was mesh size and physical (mechanical) properties of the resinous web?

Answer by K. Yasuhara:

We did not make an attempt to anchor the edges of the resinous mesh. If the edges were anchored, the mesh had been torn off due to heavy weight of transported soils. We expected that the weight of soil mass should be balanced not with the tensile force of mesh itself but with the frictional force acting among the transported soil, the mesh and the soft soil.

WOLF, T. and CHRISTOPHER, B.
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Utilization of Geotextiles in Waste Management
Utilisation des géotextiles dans les décharges controlées

Vol. 3 , p. 641

Question:

How will you stabilize secondary type industrial sludge?

Answer by T. W. Wolf:

To date, our experience has been primarily associated with stabilization of primary sludges, and combinations of primary and secondary sludges. The concept of stabilizing the sludge by inducing consolidation, however, appears to have equal merit for stabiliaation of secondary industrial sludges. It is anticipated that the methods would be quite similar to those discussed in the paper. However, since secondary type industrial sludges generally have a very low consistency, it is anticipated that the consolidation inducing stress would need to be applied in several stages so as not to induce general failure of the sludge mass.

Question:

In large sludge ponds how will you install geotextiles?

Answer by T. W. Wolf:

It can be difficult to install geotextiles on large sludge ponds due to the lack of trafficability associated with the weak sludge mass. One method we have used to gain access to the sludge ponds is to perform the construction during the winter when a frozen crust has formed on the ponds or to place a lightweight fill material such as bark at the surface of the pond to produce a buoyant effect. Once the difficulty of access to the pond surface have been overcome, geotextile installation procedures are similar to other applications.

Question:

After installing the geotextile, when you dump soil by end dumping method, what will happen to sludge which will be shifted to the other end by mud wave action?

Answer by T. W. Wolf:

Mud waves do form below the fabric ahead of the progressing face of the soil fill. If one were to fill only from one direction, the problem could certainly be substantial. Therefore, it is necessary to fill from more than one of the perimeter dikes so that the mud wave can be "trapped" within the confines of the cell. In this regard, the soil filling procedure is directly related to the desired ultimate contours of the trench surface. It may be possible to use the mud wave action to one's benefit by causing a shift in the sludge, thereby imposing desirable surface drainage features.

Vol. 3 , p. 641

Question:

Is this cost effective?

Answer by T. W. Wolf:

In our opinion, the procedures utilizing geotextile overlain by soil fill to cover industrial sludge ponds is cost effective. The cost is certainly not inexpensive, but there are seemingly few alternatives. In the absence of alternatives, the proposed methods of stabilization are cost effective.

Question:

What happens to the gasses that come through pipes?

Answer by T. W. Wolf:

Through decomposition, industrial sludges release gases. The question refers to gas vents associated with landfill abandonment. Depending on the locale of the landfill, the final disposition of the gases may vary. In isolated and remote landfills, the gases may simply be vented to the atmosphere. Alternatively, the gases may be burned at the venting location, or collected and incinerated. The third alternative, which may be utilized in those instances in which the gases have offensive odors, is to pass the gas through a carbon absorption system prior to venting to the atmosphere or incineration.

Question:

In your last slide of interior dikes, there were black pipes protuding from the dike into a gravel ore trench. What is their purpose?

Answer by T. W. Wolf:

These pipes are corrugated plastic tubing. The pipes are a component of a leachate collection system unrelated to the geotextile usage on the project. After completion of placement of the geotextile and the overlying soil fill, through consolidation, leachate is brought to the surface of the pond. The pipes are a portion of a network of piping which allows collection of the leachate. The referenced "ore trench" is also a portion of the leachate collection system. The purpose of these components is to collect the leachate and transport it to a leachate treatment facility.

Question:

Have you used a knitted drainage filter geotextile sleeve on perforated drainage pipe and/or tubing?

Answer by T. W. Wolf:

The author has not used knitted drainage filter geotextile sleeve on perforated drainage pipe and/or tubing in landfill applications. However, the author is familiar with usage of these materials and has utilized the same for more routine construction projects such as underdrainage piping for building construction.

Vol. 3 , p. 641

Question:

The authors spoke little about selection criteria and dimensioning of the Geotextiles used. Please give explanations or details on this subject.

Answer by T. W. Wolf:

The question regarding selection criteria is unclear. If the questioner seeks information regarding design procedures for the application discussed in the paper, the authors have utilized a combination of procedures available in the published literature, some of which are contained in the references cited in the paper. Through these procedures, the engineer determines which properties the geotextile must possess to fulfill the project requirements. These are one part of the selection criteria. The second aspect of the selection process is to determine which geotextiles have the desired properties. It is the author's opinion that this can only be accomplished by specifically testing the candidate fabrics for the desired properties. The manufacturer's literature is useful in an initial screening process of fabrics. However, since the tests are not often standardized, and since the standard tests often are not tailored for engineering use, the designer must test the fabrics for each specific usage.

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Ontario Ministry of Transportation and Communication, Downsview, Canada
CRAGG, C. B. H.
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Instrumented Case Histories of Fabric Reinforced Embankments over Peat Deposits

Expérience pratique de la construction de talus à renfort géotextile sur des dépôts de tourbe

Vol. 3 , p. 647

Question:

The authors spoke little about selection criteria and dimensioning of the Geotextiles used. Please give explanations or details on the subject.

Answer by M. D. MacLean:

On the first project discussed, Manchester, the geotextiles were selected in order to compare the performance of various materials used as separators between a 1.8 m high roadway embankment and peat. The only criteria used in the selection was based on the survivability during construction dictating a minimum grab tensile strength of 330 N at a minimum elongation at break of 15%. The observations, as reported in the paper indicate fabric structure and pore characteristics did not have a significant effect on the performance of these separators.

On the second project, Bloomington Side Road, the reinforcing geotextiles were chosen on the basis of load-elongation properties. A total stress stability analysis using remolded shear strengths of the peat deposit was used to estimate the stresses in the geotextile. On this basis only woven high moduli fabrics with high tensile strengths were considered.

The authors note that none of the design methods available take into consideration the deformed geometry of the embankment due to settlements. Before geotextiles are routinely used for reinforcement, design methods must be developed that will address this aspect.

Question:

In the "bathtub" effect of construction, do you anticipate a future need to retrofit the highway with edge drains to remove the water that may gather in the embankment?

Answer by M. D. MacLean:

The embankments discussed in the paper are typically low embankments, 1.5 m high, constructed on peat that has a high ground water table. Given the relatively low permeabilities of the embankment fill and the granular base and subbase courses, it is expected the edge drains would have only a limited effect. Accordingly we do not foresee a need to retrofit the embankment with edge drains.

Vol. 3 , p. 647

Question:

On Figure 4 of the paper initially large geotextile extensions decrease during the months following construction. I would be most interested to know what the author attributes this behaviour to?

Answer by M. D. MacLean:

The occurrence of this phenomenon was observed at both sides, Manchester and Bloomington, as indicated by both types of strain gauges used. This behavior is not attributed to an elastic rebound of the geotextile membrane but is speculated to be a result of stress reorientation due to consolidation of the peat deposit.

While the cause of the behavior is purely speculative the fact of its occurrence cannot be dismissed. The behavior itself indicates that a reinforcing membrane as employed on these two projects is subjected to the greatest stresses during or immediately after construction.

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Fabrics Support Embankment Construction Over Bay Mud

Construction d'un remblai, supporté par des textiles, sur la vase de la Baie de San Francisco

Vol. 3, p. 653

Question:

You indicated "very high pore pressure." How high and how much drop was required before you resumed fill operations?

Answer by J. B. Hannon:

A general "rule of thumb" for control of embankment placement suggests critical stability when excess pore water pressures in the foundation soil exceed 50 percent of the applied loading. During construction at Dumbarton, excess pore water pressures were sometimes as high as 50 to 75 percent of the applied load. These high pore water pressures required four to seven weeks for sufficient pressure dissipation of 10 to 15 percent to occur.

Embankment placement was subject to controlled loading rates and some stability had to be sacrificed by providing additional loading to promote the functioning of the wick drains.

Question:

The author spoke little about selection criteria and dimensioning of the Geotextiles used. Please give explanations or details on this subject?

Answer by J. B. Hannon:

Reinforcing fabrics require the following three essential properties to provide embankment support over soft foundation soil:

1. High elastic modulus to prevent elongation of fabric and excessive foundation deformation.

2. High tensile strength to resist embankment failure.

3. Sufficient soil-fabric frictional resistance to prevent lateral movement of embankment.

The paper presents specification requirements for a reinforcing fabric that performed successfully over a soft foundation.

Fabric specifications should not be standardized but should be tailored to each specific application giving consideration to site conditions and fabric function.

Vol. 3, p. 653

Question:

By preventing mudwaves under dams in construction, you have to confine, at least, the undrained shear strength of the mud in contact with the fabric; my estimate in the order of 5 - 10 kN/m^2. Hence, over a base width of 20 meters the fabric is stressed in the centre line by 20/2 x 5 kN/m^2 = 50 kN/m. Is that more than the ultimate strength of the fabric used?

Answer by J. B. Hannon:

Your statement is generally correct and does suggest a probable failure of the fabric under excessive embankment loading. It also suggests a need for controlled rates of embankment loading and supplemental embankment struts to prevent lateral deformation of the main embankment.

The reinforcing fabric provides initial embankment support and prevents failure during marginal stability conditions. Controlled rates of embankment placement and waiting periods are generally necessary to achieve strength gain of the foundation soil before embankment placement can continue.

Question:

Was there any evidence of clogging of the vertical drains?

What was the general index properties of the Bay mud such as undrained shear strength, liquid limit, plastic limit, natural water content?

Answer by J. B. Hannon:

No effort was made to determine if clogging of vertical wick drains was occurring after installation. However, laboratory tests by Caltrans have provided a relative performance relationship between various wick drains. These laboratory tests have closely predicted the field performance of the three wick drain types used at Dumbarton.

The following are index properties for the bay mud soils at Dumbarton:

Soft, plastic, silty clay

Mimimum Undrained Shear Strength from 4.79 KPa (100 psf) at surfact to 14.37 KPa (300 psf) at greater depths.

Water Content, 30 to 100%

Liquid Limit, 25 to 76%

Plastic Limit, 22 to 57%

Coefficient of Consolidation (Cv) at 95.9 kN/m^2 (1 tsf) pressure, 0.02 to 0.065 ft^2/day

Vol. 3, p. 653

Question:

Few papers stress the increase in safety factor of embankments due to geofabric. The basic question is:

1. How are the properties of fabric to be used in the analysis determined?

2. Do the test methods reflect the in-situ behavior of the system?

Answer by J. B. Hannon:

1. Stability analysis can be performed for a geotextile reinforced embankment-foundation system using a series of slip circles. The tensile strength of the reinforcing fabric is converted to a unit cohesion value which is distributed along the critical failure arc.

The factor of safety for the embankment-foundation system at Dumbarton was increased about 10 percent by the inclusion of fabrics. The overall system included reinforcing fabric, filter fabric and vertical wick drains. The relative contribution of each was not determined.

2. Stability analysis closely modeled actual field conditions at Dumbarton. Critical stability was experienced during various phases of embankment placement. This was reflected by high pore water pressures in the soft foundation and some deformation of the embankment. Geotextiles maintained the integrity of the embankment and prevented failure.

In situ vane shear strength tests of the foundation soil indicated marginal stability during initial construction. Subsequent tests obtained during construction reflected strength gains commensurate with consolidation.

HUTCHINS, R. D.
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Behavior of Geotextiles in Embankment Reinforcement

Le comportement des géotextiles comme renforcement de remblai

Vol. 3, p. 617

Question:

How does the embankment contribute to eliminating the problems caused by freeze and thaw?

Answer by R. D. Hutchins:

If the embankment is constructed from frost, non-susceptible material, as this embankment was, there will be no frost heave.

Question:

The sub-soil is described as a "deep black marsh muck." Could the author amplify this to include:

1. Depth
2. Organic content (or ash content)
3. Degree of decomposition of organics. (Were fibers still recognizable from plants or was it completely decomposed?
4. Moisture content

As a general plea, when dealing with organic soils and peats it is necessary to describe and classify them as vast differences in behavior are found between a low decomposition fibrous peat and a highly decomposed peat or say a muck with 50% organic and 50% inorganic particles, etc.

Answer by R. D. Hutchins:

1. 11 to 12 meters.
2. Exact organic content unknown. Identified by smell.
3. Fibers were present only in the top 15 cm.
4. Moisture content 2-300%.

Vol. 3, p. 617

Question:

How deep was the soft soil deposit and how does he expect that the notes of deposit depth to embankment width would effect his elastic load limit and bearing capacity?

Answer by R. D. Hutchins:

The soft soil deposit consists of 35 to 40 feet (11 to 12M) of unconsolidated alluvial soils and organic river silts. These deposits had a shear value that increased with depth. The width of the embankment would only be important if this were not true.

Question:

1. The insitu soil data was apparently obtained by a third party, could these details be given?

2. Would one not expect that the soil beneath the embankment to be stronger than before construction because of the influence of overburden pressure and subsequent drainage?

3. Is it valid to compare plate test performance in the manner shown, i.e. in an excavation where there are apparently no corrections for the influence of the local restraints particularly over-burden?

Answer by R. D. Hutchins:

1. The area was explored prior to construction of a bridge approach.

2. Yes.

3. ASTM-D1196-7 calls for a pit three plate diameters in diameter to minimize overburden influence. The test shows the membrane effect of the geotextile.

Question:

The author spoke little about selection criteria and dimensioning of the Geotextiles used. Please give explanations or details on this subject.

Answer by R. D. Hutchins:

Typar was selected because it is the geotextile produced by my company.

The 136 $9/M^2$ product was chosen because experience had shown it was suitable.

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The Analysis of an Embankment Constructed on a Geotextile

Etude d'un remblai construit sur géotextile

Vol. 3, p. 677

Question:

I think I understood that the author used the test area 5 cm wide strip to characterize the reinforcing geotextile. Does he always use this method? Would it not be preferable to use the more recent, more applicable tests which give much different results?

Answer by R. K. Rowe:

In this paper, I am not advocating any particular test for the determination of fabric stiffness. The parameters quoted were obtained by Haliburton et al. (1979) using 152 mm wide samples and these results give the reader an indication of the relative stiffness of the fabrics used at Pinto Pass. The 152 mm width is considerably better than the 50 mm suggested by Mr. Leflaive but I would not suggest that it is optimal. In my own tests, sample widths of up to 500 mm are being examined.

Mr. Leflaive's question also raises the broader issue of whether we can ever precisely identify the fabric stiffness - and I think that the answer to this question is No. We know that in the laboratory, the fabric stiffness will depend upon the longitudinal strain conditions, possibly the out of plane stress conditions and the rate of testing. In the field, the matter is further complicated by surface irregularities and construction effects. Consequently, we can never expect to have a precise knowledge of the fabric properties just as we will never precisely know the soil properties in the field. However, what we can hope to know, is the likely range of stiffness values that might be anticipated for a particular geotextile and it is the knowledge combined with engineering judgement that must be used in any design calculations.

Question:

You mentioned the importance of the fabric-stiffness/E modulus, to confine lateral subsoil displacement. In the Pinto Pass experiment this stiffness was probably 50% less then established by Haliburton a.o. (in short time tests), due to creep of the fabric during the first month of loading.

How would this affect the outcome of your calculations?

Answer by R. K. Rowe:

Yes, I agree that the fabric properties are likely to change with time and that the values obtained by Haliburton, et al. (1979), only give an indication of the relative stiffness. This problem was recognized and the results presented were obtained for a wide range of fabric stiffnesses in an attempt to bracket the operative field stiffness. There is no question that the actual stiffness will lie within the range examined and as a consequence the conclusions reached in the paper are quite valid.

Vol. 3, p. 677

Question:

Several people have reported that the use of a geotextile can reduce fill indentation.

How does this relate to your study and is there any theoretical justification for this finding?

Answer by R. K. Rowe:

Yes, our studies have indicated that there is theoretical justification of this finding. There are, in fact, three ways that a geotextile can reduce settlement. Firstly, if the fabric is stiff with respect to the underlying soil then the geotextile may reduce local yield in the underlying soil with a consequent decrease in centerline settlement. This effect of the geotextile is greatest for very soft and compressible soils and diminishes as the stiffness ("modulus") of the soil increases with respect to the fabric stiffness. Secondly, the geotextile may reduce indentation due to construction technique. Poor construction technique involving placing of relatively high fill over a narrow width (e.g., careless dumping from the back of a truck) may cause local failure within the soil and will require the placement of additional fill. A geotextile may serve to spread this load over a larger area thereby reducing the likelihood of local failure and fill indentation particularly if the soil strength increases with depth. Thirdly, if there are local weak spots within the soil then, without a geotextile, fill placed on the softer spots will settle requiring greater fill placement. The presence of a geotextile tends to spread the load allowing some arching over the local weak spot. The magnitude of this effect will depend upon the size of the weak spot, the relative stiffness of the fabric and the amount of fill already placed on the fabric. It should be noted that although these latter two effects can be examined theoretically, typical analyses assume good construction procedures and a soil profile which is relatively homogeneous within the horizontal plane and hence may underestimate the beneficial effects of a geotextile.

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Queens University, Belfast, U.K.

An Analytical Study of Geotextile Reinforced Embankments
Une étude analytique de remblais renforcés au géotextile

Vol. 3, p. 683

Question:

How realistic are your design factors of safety? Have you ever checked out your design method by back-analysis of the factor of safety of an embankment, containing geotextiles, that failed?

If so, how did it check out?

Answer by T. Ingold:

In the analytical study presented two factors of safety are involved. The first is applied to the ultimate tensile strength of the geotextile and the second to a limiting equilibrium analysis of the soil-geotextile system. There is only compatability between these two factors when they are both at unity and clearly nothing is more real than a failure. The analytical approach as defined in equation (6) has been applied to the Almere slip to give a factor of safety of 0.86 assuming the geotextile to remain horizontal. In fact the geotextile deformed along the failure surface in which case the calculated geotextile restoring force is even larger as is the calculated factor of safety. This then introduces debate on the deformation of failure. Is it total collapse or excessive deformation? In considering service states rather than collapse states what is simply described as a factor of safety against tensile failure must take into account many factors. For example, if short term strain is to be limited then F_R might be defined as the ultimate tensile strength divided by that strength at which the desired limiting strain is achieved. If long term strain or creep rupture is to be considered then geotextile tensile loads compatible with acceptable strain rates might be a criterion. For simplicity this can still be expressed in terms of F_R. All of this of course presupposes that the tensile strength of the geotextile can be defined. This in turn is largely a case of agreeing test standards with everything else related to this datum. The second factor of safety which applies to the soil-geotextile composite confers the normally accepted limitation on soil shear stress relative to shear strength and an additional factor in the geotextile load. The analytical method could be amended to eradicate this conservatism. Like any other design approach one starts with an analytical model which is amended as necessary in the light of field performance. Clearly there have to be many more well documented failures before a reliable, non-conservative design method can be calibrated.

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Fabric Reinforcement of Embankments and Cuttings
Renforcement des remblais et des déblais à l'aide de textiles

Vol. 3, p. 707

Question:

Groundwater conditions and relic weakness in the soil often trigger slips in cuttings.

In the repair you described you installed drainage to control groundwater and recompacted the fill to eliminate relic weakness in the soil.

Do you think the reinforcement is also required, as you described at your site, where you are reinstating the slope at the same slope angle as before?

Is the reinforcement only giving additional security, e.g., should the drainage measures fail?

Answer by R. T. Murray:

As I have described in the paper, the failed section of cutting was reinstated employing the original foundered soil reinforced with Netlon mesh. This soil was in a very poor condition with a high water content and such low shear strength that construction plant could not operate effectively. To overcome this problem, unslaked lime was spread over the top of each layer to produce an adequate working surface. Beneath this surface, however, the soil was virtually at the original condition and further compaction only generated excess pore pressures without enhancing the soil properties. The reinforcement was therefore required to ensure the integrity of the slope by preventing slip planes from developing through this soft soil.

The drainage trenches installed at the base of the excavation and into the soil where the original collapse had initiated were intended to prevent a further, deeper seated, failure occurring. The presence of the reinforced soil above this region would not necessarily eliminate the possibility of such a failure as the whole reinstated section could have been carried on the slip plane.

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Strength Properties Measurement for Practical Applications

Le mesure des caractéristiques de traction en vue des appplications pratiques

Vol. 3, p. 733

Question:

For which reason you decided to use 500 mm in the direct traction test. Why not 1 m or 0.2 m?

What is the weight of the 500 mm steel clamp for this test? How does your laboratory staff like to perform our tests?

How much time is involved for one single test (mounting, straining, dismantling included)?

Answer by E. Leflaive:

Pour le premier point, les réponses se trouvent dans la communication écrite; il faut notamment examiner la figure 5 page 736 qui montre qu'un rapport b/h de l'ordre de 5 est nécessaire pour tester les différents matériaux géotextiles dans des conditions comparables.

Le poids des pinces dans la machine de traction est d'ure dizaine de kilogrammes, mais ce chiffre n'est pas particulièrement significatif car elles restent fixées sur la machine et on ne les manipule pas pendant les essais.

Le temps nécessaire pour un essai sur éprouvette de 500 mm est le même que pour un essai sur éprouvette de 200 mm.

En tout état de cause, c'est le comportement des matériaux qui doit nous dicter la dimension des éprouvettes et non pas des considérations arbitraires de commodité.

Question:

Would Dr. Leflaive agree with Prof. Bell that 200 x 100 mm specimens are adequate or does he maintain that the 500 x 100 mm specimen is necessary?

What type of grip does he use and how successful is it in testing high modulus/high strength fabric?

Answer by E. Leflaive:

Je maintiens que la largeur de 500 mm est hautement préférable pour tester différents géotextiles dans des conditions comparables (voir communication, figure 5, page 737) et obtenir des données valables sur la déformation des géotextiles en traction. Cette valeur a également été retenue, après examen et discussion, par l'Association Internationale des Congrès de la Route (A.I.P.C.R.) (communication de monsieur Delmarcelle page 327) et par la Réunion Internationale des Laboratoires d'Essai de Matériaux (R.I.L.E.M.) (communication de monsieur Gamski page 325); une largeur plus faible est acceptable pour les matériaux de haute résistance si des essais dans un laboratoire spécialisé ont montré que son adoption ne modifiait pas trop sensiblement la déformation à la rupture.

Les pinces utilisées sont des pinces en acier avec un profil ondulé s'opposant au glissement; pour certains matériaux l'interposition d'un autre géotextile entre la pince et celui qui est testé est utile pour éviter à la fois le glissement et la coupure; les pinces sont serrées par des boulons vissés par une clé pneumatique à effort contrôlé.

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Question:

D'après les essais que vous avez faits avec les fils continus, pensez-vous que le choix du polymere du fil sera important dans les différentes applications de cette nouvelle technique?

Answer by E. Leflaive:

Nous n'avons pas encore experimenté une variété suffisante de fils et de polymères pour répondre précisément à cette question. Nous avons l'intention de faire des essais de fluage pour comparer le comportement de différents fils.

Question:

I understand that your triaxial tests were run at cell pressure of 50 kPa and greater. In many practical or potential applications, the minor principal stress σ_3 will be less than 50 kPa. How would the influence of the fibres change at low confing stress?

Answer by E. Leflaive:

La communication indique que nous avons également fait des essais de compression simple, donc avec $\sigma 3 = 0$. Nous avons trouvé dans ces essais des valeurs de résistance correspondant aux cohésions mesurées dans les essais triaxiaux. Dans la mesure où cette cohésion est liée directement à la résistance à la traction des fils dans une section de surface unité, il n'y a pas de raison pour qu'elle change beaucoup en fonction de $\sigma 3$, au moins en première approximation.

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A Wide Strip Tensile Test of Geotextiles

Essai de traction des géotextiles sur eprouvette large

Vol. 3, p. 739

Question:

Have you had any success with grips for testing high modulus/high strength materials and if so what do you recommend?

Answer by J. R. Bell:

No. We have had very little experience with grips for high strength materials. The grips we have been using, which simply bolt together, are inadequate for fabric strengths greater than about 60 kN/m.

Question:

Is the ten percent strain per minute rate you recommended in agreement with the field conditions? Please elaborate.

Answer by J. R. Bell:

In the field some loads, traffic loads for example, are applied very rapidly. On the other hand some loads from normal construction are applied slowly. The 10% strain rate we recommend is not selected to duplicate field conditions. It is chosen to be (1) fast enough to keep testing economical, (2) fast enough not to include a large "creep" component, and (3) slow enough to limit the effects of very rapid loading.

Question:

The recommended test procedure includes a specimen width of 200 mm and a gauge length of 100 mm. How much time is required to properly clamp the specimen in the test jaws?

At the recommended strain rate this test could take up to 10 minutes for testing nonwovens. Could you comment on the effect of test results by increasing strain rates up to 2.5 inches/minute?

Answer by J. R. Bell:

It only takes three or four minutes to clamp the specimens in the test jaws and mount them in the test machine.

We did not perform tests at elongation rates as fast as 2.5 inches/minute. However, we do have tests that suggest the test results are affected at rates higher than 10 to 15% per minute which corresponds to elongation rates of 0.4 to 0.6 inches/minute in our test.

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Creep Behavior of Geotextiles Under Sustained Loads

Le fluage des géotextiles sous charges permanentes

Vol. 3, p. 769

Question:

The graphs seem to indicate that strain continues with time until it accelerates at some point before failure. The strain is obviously also a function of applied load, therefore

1. Can you predict the usable lifetime of the geotextile based on an upper acceptable limit to the creep?
2. Do you think creep will limit the usable life of the textile rather than chemical durability?
3. Have you or any committee members made recommendations for the working strength of the geotextile to be used by the practicing sales engineers based on a percentage of ultimate strength or in some way related to creep? Does this differ for the specific product or manufacturer?

Answer by J. R. Bell:

1. No. The state of knowledge does not allow me to do this. It would require being able to compensate for confinement in a soil, temperature effects, and changing loads. My current approach is to select a working stress low enough to protect against tertiary creep failure or limit creep deformations as required.
2. The importance of creep depends on load level. Either creep or chemical life can be longer.
3. I do not know of any published set of recommended working loads for creep. The working load as a percentage of the ultimate load depends on fabric structure, polymer, and fiber manufacture method. I currently use 60% for polyester fabrics and 40 to 50% for polypropylene geotextiles.

Question:

The author indicated that polypropylene geotextiles have significantly higher creep rates than those of polyester, yet the retaining walls he built with polypropylene fabrics are still standing.

1. Is this explained by confining pressure?
2. Has the test procedure practical relevance for design or selection tests?

Answer by J. R. Bell:

1. Confinement of a geotextile in the soil reduces creep rates and undoubtedly is a factor, but there are also other factors. The walls in question were designed for high live loads; therefore, the dead load is only a small percentage of the geotextile strength. Also, the analysis methods over-estimate the stresses in the geotextile. The actual sustained load is too low to cause large creep.
2. Yes. The creep test has practical value. It can be used to select a conservative working stress which will limit creep strains and protect against failure due to tertiary creep.

Vol. 3, p. 769

Question:

Was any attempt made to compensate loading for area reduction as the creep samples elongated?

If not, do you feel that true tertiary creep occurred, or was the accelerated creep rate due to an increase in tensile stress?

Answer by J. R. Bell:

No, we did not attempt to compensate for area reduction. I do not know if we have "true" tertiary creep in our tests, but I think what happens in the test is analogous to what happens in the field.

Question:

Les changements de comportement en fluage des geotextiles en polypropylene entre 22 c sont dus essentiellement a mon sens au fait qu'a basse temperature le polypropylene se trouve en-dessous de sa temperature de transition vitreose?

Pensez-vous que ce comportement puisse compromettre l'utilisation des geotextiles polypropylene en renforcement, dans des regions froides ou ils seraient mis en oeuvre a tres basse temperature, et pourraient ulterieurement lor du degel, fluer?

Answer by J. R. Bell:

I agree that the marked reductions of creep for the polypropylene geotextiles is due to the fact that the glass transition temperature for this polymer is between 22°C and -12°C.

The situation described should not cause failure of polypropylene reinforcement if it is tested and designed for creep at the higher temperature.

Question:

Has any of the creep tests performed at saturated moisture condition? If they have, then what is the effect of moisture content on the creep behavior of different types of fabrics?

Answer by J. R. Bell:

No. I do not know the effect of water content on creep. We have assumed it to be relatively small for conventional tensile strength tests.

Question:

What is your opinion concerning the importance of creep behavior in the practice applications?

Answer by J. R. Bell:

Creep behavior is very important in practical applications for reinforcement. The allowable working stresses for geotextiles for long term loadings are controlled by considerations to limit possible rupture in tertiary creep in reinforced structures. In heavy duty subgrade stabilization of aggregate surfaced roads on very soft soils, rupture of the geotextile may also result from a kind of tertiary creep resulting from cyclic creep under repeated wheel loads. Also, in haul roads, fabric prestress developed by rutting may be dissipated by relaxation which is a creep related phenomenon.

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Question:

1. If early walls were successful, using polypropylene, why not recommend again?

2. What field experience is there to indicate that lab tests are not realistic?

Answer by J. R. Bell:

1. I do recommend them again.

2. The evidence is the experience with the early walls which do not show the movement predicted from lab test data. Walls of different materials which show different lab results show similar movements in the field. Part of the problem may also be our inability to accurately know the stresses in the geotextiles in the field installations.

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Resistance to Area Change as a Measure of Fabric Performance
La résistance des tissus au changement de superficiel comme mesure de performance

Vol. 3, p. 781

Question:

In your last slide you compare the properties of different products.

Have all products the same weight?

Answer by A. Newton:

The question referred to a slide showing some additional information not given in the paper. It contained values of penetration energy of a variety of materials obtained from the test shown in Fig. 5 (a) and quoted from Reference 4 of the paper. The values of penetration energy are shown below together with the weight/unit area of each material.

	Type of Material	Penetration Energy Kg. cm/g	Wt. g/m^2
1.	Paper	0.167	77
2.	Polyethylene	1.24	50
3.	Woven steel	1.2	128
4.	Spunbonded polyester	2.2	106
5.	Bonded Polyester nonwoven	4.37	185
6.	Spunbonded polypropylene	6.1	110
7.	Bonded Nylon	6.17	157
8.	Woven cotton	8	133
9.	Needle felt, nylon	13.8	230
10.	Warp Knit nylon	17.4	85
11.	Woven nylon	28	116

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Load-Extension Testing of Geotextiles Confined In-Soil
Proprietés d'extension sous charge de géotextiles placés dans le sol

Vol. 3, p. 793

Question:

In the UK a major highway embankment failure occurred where a geotextile separated a clay fill from a horizontal granular drain later found to have a low permeability. Could the apparatus described in the paper be used to model different soils on each side of the specimen in undrained conditions.

Answer by A. McGown:

Yes, this apparatus or, at least, one very similar to it could be used.

Question:

It has been known since 1910 that the correct test for the conditions applying to geotextiles is biaxial L-E testing.

1. The interpretation of the effect of confining is made difficult because two effects are mixed up: the imposition of a cross-wise stress and the effect of compression. Why has no one considered this?

2. If you are confining the specimen can you justify the fabric dimensions which introduces boundary effects not occurring in practice?

Answer by A. McGown:

What we did was first of all to establish the size of test specimen which reduced the boundary effects to an acceptably low level then considered the influence of compression whilst subjecting the geotextile to external uniaxial loading. Of course, internally whether or not the geotextile is compressed in soil, biaxial stressing occurs. We believe our choice of sample size allows us to overcome the need for lateral restraint which the questioner is referring to.

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Effect of Confining Pressure on Performance of Geotextiles in Soils
L'effect de la pression de confinement sur la performance de géotextiles enterrés

Vol 3, p. 799

Question:

The authors have acknowledged the limitation of their test procedure and have noted many arbitrary assumptions made in interpreting their results.

1. What do they consider is the uncertainty of their results?

2. Would they be prepared to use the results from their tests directly in design?

Answer by E. Nowatzki:

1. Each data point plotted in Fig. 4 through 9 represents the average of three tests. The curves in figures 8 and 9 are "best fit" curves obtained from regression analyses. The results of the regression analyses yielded acceptable values for correlation coefficients. No other statistical analysis of the results was undertaken.

2. The authors feel confident in the results of this research. Indeed, they have already used them successfully in the analysis of a composite soil-geotextile beam. This structure was a design concept proposed for low level nuclear waste disposal trench capping; a prototype beam was built and its performance analyzed by using the moduli results reported here.

Question:

Was the anisotropy on woven materials considered in the analytical model? If so, was any test run on woven material on the bias?

Answer by E. Nowatzki:

As part of the overall research, stress-strain curves were obtained for two woven geotextiles (Mirafi 100X and Mirafi 500X) under the confinement of dry #30 Ottawa sand as cover and support materials. Two series of tests were performed, one with the horizontal force applied parallel to the direction of filaments (along the grain), the other with the horizontal force applied normal to the direction of the filaments (across the grain). The results of these tests were not presented at the conference because of space and time limitations. They indicate, however, that the across grain orientation results in a slightly larger equivalent tensile stress for a given level of strain for all magnitudes of confining stress investigated. However, we would not consider this increase to be significant. The authors would be pleased to send the results to the questioner upon request. No tests were run on woven materials cut on the bias.

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Question:

1. Do you really feel that test specimens will neck down the same amount under confinement as in the unconfined test? If you assume little or no neck down, how will that effect your data and conclusions?

2. Did the specimens fail at between the grips in the center of the test specimen between the grips in the center of the test specimen or at the grip?

3. Concerning the influence of moisure in the soil, did you try saturating the non-woven fabric and repeating the test?

Answer by E. Nowatzki:

1. The amount of neckdown is a function of horizontal deformation (refer to Fig. 2). The authors feel that for the same amount of horizontal displacement, the test specimens will neck down the same amount under confinement as in the unconfined test; however, the equivalent tensile stress in the former case will be larger than in the latter. If neck down is neglected, lower equivalent tensile stresses will be predicted for a given level of strain, therefore lower confined moduli will also be predicted.

2. None of the test specimens were observed to fail at the grips. As indicated in the paper, only the nonwoven geotextiles experienced neck down; for those materials failure occurred in the area of the neck down which was generally near the center of the specimen. No significant neck down was noted in the woven specimens; failure in them usually occurred transversely but off center toward the loading edge.

3. One set of results for saturated soil is shown in Fig. 5 for Mirafi 140S under 383.2 kPa (4 tsf) and a sand-sand interface. The authors do not know if this answers the question; we assume that when the sand is saturated, the geotextile is saturated also.

Question:

1. Would the author please comment on how the direct shear test proposed differs from a slightly modified pull-out test.

2. Has the author confirmed that the "necking" which occurs in isolation tests also occur when the specimen of geotextile is confined in soil, otherwise the equivalent tension calculated will be in error.

Answer by E. Nowatzki:

1. The test proposed uses a modified direct shear test device, but is not a direct shear test. Neither is it a pull-out test in the sense of pull-out tests performed to date since one edge of the geotextile is restricted from moving horizontally.

2. The authors confirmed that the "necking" which occurred in unconfined tests probably also occurred in confined tests by observing a number of geotextile specimens in the test device after a confined test. The amount of neck down for a given amount of horizontal displacement was approximately the same in both cases. One must realize, however, that prior to the observations, the confining stresses and the interface soils were removed in the process of dismantling the test apparatus. We did not measure neck down directly during tests on confined specimens.

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Question:

Both speakers describe the same property but with different apparatus.

Please discuss the differences between the two procedures and how these affect the results. Are there any serious limitations considered in the other approach?

Answer by E. Nowatzki:

The features of our (El-Fermaoui and Nowatzki) simple test appartus and test procedure are:

1. Constant normal stress is applied to the entire area of the specimen being tested throughout the test. Unless the normal stress is constant over the entire length of the specimen during application of horizontal load, failure will tend to occur at stress discontinuities and the results may not be applicable to the field conditions of interest.

2. Restraint of the specimen edge opposite to the edge being pulled. This boundary condition assured that the specimen would not be pulled out of the testing device completely during the test so that the entire specimen was subject to the applied normal stress.

3. The lateral edges of the specimen were free to move during the test in response to the geotextiles tendency to "neck down" under application of horizontal load.

4. The measured load-elongation data can be easily reduced to stress-strain curves from which values of moduli may be obtained as a function of applied normal (confining) stress.

   Our test boundary conditions appeared to suit our purposes best; we wished to use the results in an analytical model of a buried soil-geotextile beam. As it turns out, this simple test served our purposes well since the analytical model in which the moduli results were used, predicted the response of a prototype beam quite well. However, other applications having different boundary conditions might not be so well served by these test results.

If more data were available from full-scale field tests, perhaps better insight could be gained into the need for sophisticated testing devices and a judgment could be made as to just how far the results of simple tests could be extended to given applications.

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Creep Characteristics and Stress-Strain Behavior of a Geotextile-Reinforced Sand

La fluage et le comportement contrainte-déformation de sable renforcé par des géotextiles

Vol 3, p. 805

Question:

The comparison of geotextile reinforced samples with sand control samples shown on the diagram of creep strain versus LOG_{10} time seems to be in complete contradiction to the creep enhancing soil, confining effects described by Dr. K. Andrawes. Is this apparent contradiction equally applicable to the total strains that were developed by your geotextile reinforced samples?

Answer by R. D. Holtz:

It is not clear that our findings on confined soil-geotextile creep contradict those of McGown, Andrawes, and Kabir (this Conference, Vol. III, p. 798). They reported no tests at confining pressures other than 100 kPa, so I'm not sure how their tests indicate a "creep enhancement." The values of total strains reported in Table IV, Col. 4, are total strains after creep and rapid loading to failure. They did not report the results of any similar tests. Even though strictly comparable tests were not carried out, our data appears to be consistent with theirs for Mirafi 140S and Terram 1000. Our ε_c was 1.8% while theirs was about 1.5 to 2% (scaling from their Fig. 8 (a)), even though confining pressures and creep loads were slightly different.

Question:

Since sand as a boundary condition has a very significant effect on the stress development of the geotextile would you recommend that the fabric be bedded in and covered by a blanket of sand when installed in a clay embankment?

Has anyone studied the effect of gradation of the sand, say, from a fine to a coarse (100% passing 1/4")?

Answer by R. D. Holtz:

1. Yes, because the sand layer will definitely enhance drainage, consolidation and thus stability. I don't know how the creep characteristics would be affected, however.
2. As far as I know, not on the creep characteristics of the sand-geotextile system. Gradation (or at least grain size) and friction have been studied by Collios, et al. (1980), among others (Portland Meeting of ASCE).

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An Evaluation of Abrasion Tests for Geotextiles

Une évaluation de tests d'abrasion de géotextiles

Vol. 3, p. 811

Question:

In the repeated Loading abrasion test apparatus is the loading frequency and amplitude used relevant to that experienced in the rail track?

Answer by D. Van Dine:

The wear abrasion test that we conducted was a repeated loading test, not a dynamic loading test. In other words, the components of the system, for example the ballast-geotextile-ballast "sandwich," were always in contact with one another. Therefore, loading frequency and amplitude are of much less concern. The load of 290 kPa was chosen to simulate a wheel load. The number of cycles were chosen to represent gross total tonnage per time period on a rail line.

Question:

You have looked at the surface effects. But why did you not test the samples for loss of strength and for change in pore size distribution; as the main problem with failure of geotextiles under ballast for mud pumping.

Answer by D. Van Dine:

You are correct in stating that a major problem with failure of geotextiles in railroads is mud pumping. In the series of tests that were conducted for, and presented in this paper, we were interested in studying the resistance of the geotextile to impact and wear abrasion. In other tests carried out by G. P. Raymond, et. al., and soon to be published by the Canadian Institute of Guided Ground Transport (Queen's University, Kingston, Ontario, Canada), samples were tested for loss of strength and change in E.O.S.

Question:

Besides the application you presented in this paper (that's railroads) which other fields you think to be suitable for your abrasion tests?

Answer by D. Van Dine:

Impact abrasion of the geotextile occurs whenever aggregate or fill material is dropped onto the geotextile. This would occur in road building, constructing revetments or even backfilling of trench drains. Wear abrasion occurs whenever a repeated load (not necessarily a dynamic load) occurs. This would occur during the construction stages of most projects, but probably only continues in structures such as railroads and roadways that have a thin cover of material over the geotextile. The impact and wear abrasion tests outlined in this paper could easily be modified to test for the resistance under site specific conditions.

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Experimental and Theoretical Study of Tensile Behavior of Nonwoven Geotextiles

Etude expérimentale et théorique du comportement en traction des géotextiles nontissés

Vol. 3, p. 823

Question:

Please review the theoretical part of the paper which deals with the effects of fibers not terminating in the grips but mechanical coupled to the sample by entanglement, friction, and/or bonding.

Answer by J. P. Giroud:

In the theoretical analysis, no entanglement, friction, and bonding are considered. Therefore, fibers not terminating in the clamps do not contribute to the strength. In reality, there is entanglement, friction and/or bonding. These effects provide an additional strength, not taken into account in the theory.

Question:

It is my impression that your conclusions concerning the effect of sample size on ultimate strength, elongation and modulus are almost completely the opposite of those outlined by Prof. Bell (Vol. 3, p. 739) for similar tests. Could you comment?

Answer by J. P. Giroud:

In my paper with Baudonnel and Gourc, one of the conclusions is that the strength of nonwoven geotextiles measured in strip tests is affected by the size of the specimen. On the other hand, Shrestha and Bell (page 743 of the proceedings) write: "The ultimate strengths of geotextiles measured in strip tensile tests are not significantly affected by specimen size." There is obviously a discrepancy between these two conclusions and it is important that Peter Jarrett drew our attention to this point. A careful analysis of the two papers would be necessary to understand the reasons for these discrepancies and researchers are encouraged to do such analysis.

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Results of Permeameter Tests on Statically Loaded Geotextiles

Résultats d'essais de perméabilité sur des géotextiles chargés statiquement

Vol. 3, p. 845

Question:

In your talk you have used terms such as permittivity, permeability, and transmissivity. Do all of these terms not represent the same characteristic of the fabric? If so, why not use one term, namely permeability, with which geotechnical engineers are familiar.

Answer by F. Bucher:

The terms do not represent the same characteristics. The advantage of the term permittivity is that this value may rather be considered as a constant for a given fabric over a certain range of normal stresses. In comparing different products the terms permittivity and transmissivity may be more useful than the permeability where one has anyway to distinguish between the permeability normal to the plane of the geotextile and the permeability in the plane of the geotextile.

Question:

At what normal pressure do you think a geotextile should be tested for water flow or should some form of compressability index be used.

Are the geotextile 1-3 the same on the basis of 1) pore size, 2) weight/m^2, and 3) strength.

Answer by F. Bucher:

The normal pressure applied in determining the permeability of geotextiles should be in the range for which the geotextile will be used in practice. By performing tests at different stress levels the compressibility index will not be used for the determination of the permeability. Geotextile 1-3 are not too different in terms of the weight/m^2 (200-270 g/m^2) as indicated on page 846 of our paper. The differences in terms of strength and pore size are in a similar order of magnitude, but not the same.

Question:

Could the author please comment on the usefulness of measuring in-plane permeability of multiple layers of geotextiles, since, in practice, only one layer is used?

Could he list the constituent polymers in geotextiles 1, 2 and 3?

What would be the effect of wash-in soil particles on in-plane permeability?

Answer by F. Bucher:

With regard to the first part of the question, we are aware of the problems associated with using several sheets of geotextiles for in-plane measurements, mainly for spun-bonded geotextiles. We have therefore constructed meanwhile another permeameter where we can get around this problem. With regard to the polymers, they are given on page 846 of our paper. Concerning fine particles which are washed into the geotextiles we may certainly say that they decrease the in-plane permeability of the fabric and I like to refer to some interesting papers which have been presented on this aspect during this Conference.

Vol. 3, p. 845

Question:

1. With ingress of soil into the geotextile what would be its effect on the in-plane permeability of needle punches geotextiles?

2. Did you take into account the effect of the permeability of the porous pressure plate in the calculation of normal permeability of the geotextile?

3. When measuring permeabilities of soils in the laboratory coefficients of variation are normally of the order of 300%. What coefficient of variation for the geotextile permeabilities did you obtain in your laboratory tests?

Answer by F. Bucher:

The first question is covered by the preceeding answer. With respect to your second question I may say that the pressure heads were measured directly on top and below the geotextile and that therefore the loss of pressure heads in the porous plates have been considered. The coefficients of variation for the geotextile permeabilities are according to our test results rather less than that for soils. But due to the fact that we test several sheet of geotextile each test results in an average value of the permeabilities of the individual sheets.

Closing Reports

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Report on Sessions 2A, 3A, 4A, and 5A Drainage

Rapport des sessions 2A, 3A, 4A, et 5A Drainage

Thirty one papers originating from ten countries were presented in the four sessions on drainage. These papers can be grouped into three categories: reports on investigated in situ applications, analysis of results obtained in laboratory research programs and proposed flow models and filtration criteria. The property of non-woven geotextiles mostly investigated was the transmittivity of fabrics. It was well established that this property offers drainage capacity very useful in geotechnical applications even when the geotextiles are under compression. In this paper, I will report on the presented papers: the investigated in situ applications, the drainage capacity offered by non-woven geotextiles, results obtained in laboratory test programs. Finally I will underline fields into wich more investigations are needed in the future.

Trente-et-un articles ont été présentés lors des quatre sessions sur le drainage et ils peuvent être regroupés en trois catégories: des rapports sur l'examen de travaux accomplis, des analyses de données obtenues en laboratoire et enfin des critères de filtration proposés. Cependant, la transmittivité des géotextiles non-tissés a été certainement la propriété la plus étudiée et discutée. Il a été démontré que cette propriété peut être utilisée dans plusieurs applications en géotechnique même lorsque les géotextiles sont soumis à des compressions importantes. Dans cet article, je présenterai les articles en fonction des applications examinées, des mesures de la capacité de drainage des géotextiles, des résultats obtenus en laboratoire et finalement je soulignerai les domaines qui devront être étudiés dans l'avenir.

From all geotextile functions, filtration and drainage capacities of synthetic fabrics to be used in geotechnical applications are certainly the most utilized. But more important for the engineers is the guarantee that following the installation of a geotextile in a work their properties can be conserved and that blocking or clogging of the fabrics do not alter their original values to such an extent as to offer unsafe conditions. Because of the lack of knowledge in geotextiles' behaviour in contact with soils of different nature, scientific investigations were performed in laboratories to simulate filtration and drainage applications and in situ examinations of installed geotextiles were done. Innovative designs were developed to optimise their utilization in many applications.

Most of the presented papers in the drainage sessions fall into one of these categories. As shown on Figure-1, 31 papers were presented originating from 10 countries. Eight of the papers were reporting in situ investigations, twenty papers were analysing laboratory test results while the other papers presented empirical and theoritical analysis to obtain filtration criteria and flow models. Finally, a paper indicated psychological hang-ups to overcome.

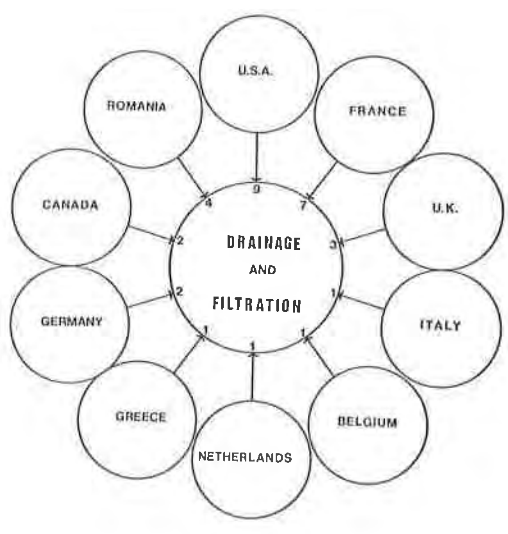

Fig. 1 Number of papers presented in the drainage sessions.

IN SITU APPLICATIONS INVESTIGATED

As shown on Figure-2, eight papers were presented on the analysis of geotextiles' behaviour in applications. These investigations covered most of well known applications such as vertical drainage, trench drains, ponds, depth drains, fin drains, protection of geomembranes and seadikes and inland waterways.

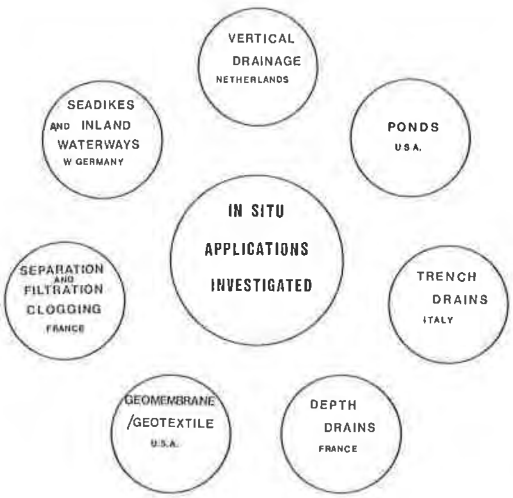

Fig. 2 In situ applications investigated.

The behaviour of five types of prefabricated vertical drains have been compared and the results indicated large differences of behaviour amongst the functionning of the drain in the tested areas (DE JAGER). Further analysis of these results is needed to indicate wich properties of the drain materials have resulted to the observed differences in behaviour.

Withdrawn samples of geotextile and surrounding soil from a trench drain was performed after 30 months of utilization (CANCELLI). The analysed samples indicated only slight variation in the permittivity and transmittivity of the contaminated geotextiles and that a cake was formed upstream of the fabric with the coarser soil particles adjacent to the geotextile.

Both field and laboratory tests were conducted on geotextiles used as an underlining of geomembranes in pond construction to insure puncture protection, gas release and abrasion resistance (COLLINS). It was found that thick non-woven geotextiles in a weight range of 400 to 600 g/m^2 provided a satisfactory behaviour.

From field experiences, it was shown that design of geotextiles associated with geomembranes used for ponds, canals and dams do not have the drawbacks of granular materials (GIROUD). Several examples are presented to illustrate how to use properly geotextiles to alleviate effects of underpressures and mechanical actions

A ten meters depth drain along a motorway was achieved in saturated clays and technical checks were performed after four months of performance (PUIG). The results indicated good efficiency of the drain and no disturbance in the protected slopes.

Laboratory testings were performed to examine the hydraulic flow and filtration properties of fin drains to be used in ground and structure drainage (HUNT). It was found that fin drains, constituted of a plastic core and a synthetic fabric, were capable of meeting the demands for structure drainage.

Intensive field investigations on long-term behaviour of geotextiles in coastal structures were carried out and geotextiles were dug out after many years in function (HEERTEN). Their filtration properties were measured. It was found that a great amount of soil particles were incorporated into the non-woven fabrics and that the thickness of the fabrics increased when compared to the thickness of these fabrics after the soil particles were washed out (90% of soil particles could be removed from the washing procedure).

Clogging of several geotextiles dug out of geotechnical works after 12 years of installation was analysed using a morphological approach (SOTTON). Three types of clogging have been defined and it was found that agglomeration of fibres in a geotextile structure constituted clogging sites. More important are the obtained photographs (shown in the paper) that suggest different soil particles level of attraction to fibres depending on the polymer and their surface treatment.

RESULTS OF LABORATORY TEST PROGRAMS

Results of laboratory investigations were analysed and presented in twenty papers. These research programs can be grouped under many filtration and drainage behaviours, as shown on Figure-3, such as permittivity and transmittivity measurements, fin drain flow behaviour, clogging mecanisms, pore size characterization, filtration of suspended soil particles, siphon ability, hydric properties and others.

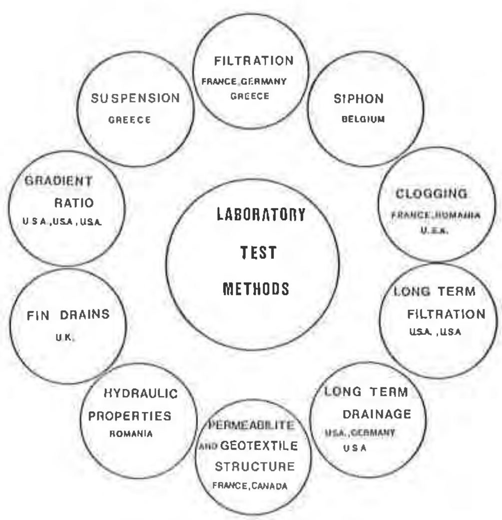

Fig. 3 Laboratory test programs

An experimental investigation indicated that in some circumstances, some types of geotextiles can perform in a siphon function (GAMSKI). The conditions are discussed and laboratory test apparatus are described. More work is needed.

Experiments were conducted in a design laboratory apparatus to assess the solids removal efficiency of geotextiles in filtration of suspension of Kaolinite and Grundite particles (ATMATZIDIS). A filterability index, used to measure mass removal efficiency, indicated poor behaviour as fabrics for filter systems.

A second study of use of fabrics as filtering elements was presented (KELLNER). It was found that many of the geotextiles used were manifesting a susceptibility to retain fines due to the action of electrochemical forces.

Long term filtration tests were performed in many laboratories (KOERNER, LOUBINOUX, RYCROFT). It was found that the initial flow of water through the svstem soil/fabric is governed by the soil but that the final range is governed by the soil/fabric interaction. The minimum time required for such tests varies from few hours to hundred of hours depending on the nature of the soil. Also, depending on the experiments performed, the permittivity deacreases were negligeable to very important as a function on the silt and clay content of soils. But more important is that the permittivity decrases rapidly in the initial stages of the filtration to level off with filtration time.

Studies of clogging of geotextiles using gradient ratio test method were performed (HALIBURTON, KOERNER). Gradient ratio values were found to increase rapidly with increasing soil silt content such that there is no unique value of gradient ratio.

The mostly investigated properties of non-woven geotextiles were the transmittivity and the permittivity under compression (RAUMANN,LOUDIERE, ANDREI, IONESCU, GOURC, ROLLIN,McGOWN, KOERNER). It was found that these properties vary slightly with an increase of compression ranging from values of 0 to 1000 kPa and that the load influence becomes negligeable for values greater than 200 kPa. As shown on Figure-4, the transmittivity varies more than the permittivity such that both properties are reaching similar values at high compression level. On the other hand, structure analysis of compressed non-woven fabrics indicated that the distance between fibres decreases dramatically from average value of 140 µm at 25 kPa to value of 40µm at 800 kPa. These findings are suggesting different filtration behaviour of geotextiles under compression (clogging levels, particle size retention, ...).

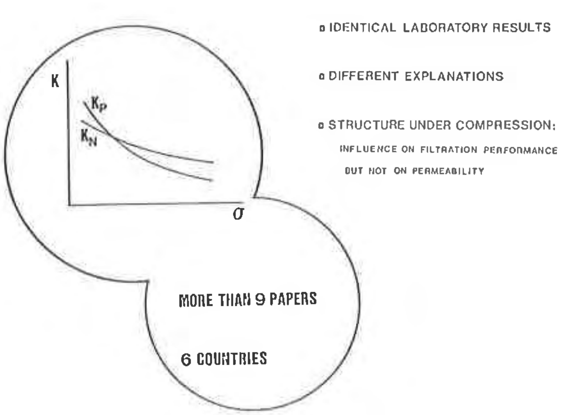

Fig. 4 The field the most intensively investigated

Pore size characterization was also a field investigated by many researchers (FAYOUX, ROLLIN, HALIBURTON, ANDREI, LOUDIERE, GOURC, McGOWN). Except for direct measurement obtained with the Image Analyser, various wet and dry sieving techniques have been developped. The data obtained " are not entirely consistent but can provide an approximate estimate of largest pore sizes of geotextiles in uncompressed state " (McGown).

PROPOSED FLOW MODELS AND FILTRATION CRITERIA

Models to represent the flow of water through geotextiles and filtration criterion were proposed by many authors (PUIG, GOURC, ROLLIN, FAYOUX, WITTMAN, GIROUD, TAN, IONESCU, RAUMANN, McGOWN).

The mostly used filtration criteria were basically a comparision between the O_{95} of the pore histogram of a geotextile to the uniformity coefficient and density of the soil as a retention criterion and also to insure that the permeability of a geotextile is greater than the permeability of the soil. It should be noted that on the other hand many other criteria were used through out the presented papers such that it becomes clear that more work is needed in this field.

The influence of size of fibres on their hydraulic characteristics was determined from analysed models and corraborate from laboratory experiments (FAYOUX).

It was also shown, that the permittivity and the transmittivity of geotextiles could be calculated from the fabric porosity, the fibres' shape, the fibres' density and the polymer specific mass (RAUMANN, GOURC, ROLLIN).

It was also indicated that the flow through a geotextile should not be assimilated to flow through a soil both materials being structuraly different.

Finally, a model was developped to predict earth fills consolidation using fabrics (AURIAULT).

FIELDS TO BE INVESTIGATED IN THE FUTURE

Following the study of the presented papers in the drainage sessions, it becomes apparent that in some areas more investigations are needed to understand the soil/fabric behaviours.

FILTRATION CRITERIA

Because of the many filtration criteria proposed and used, it is believed that the cake formation as well as the retention of particles are resulting from mecanisms that are influenced by the geotextile structure and the soil granulometry and nature. Also, it is now well recognised that geotextiles retention of particles and their resulting clogging level vary greatly upon the state of the system soil/fabric . A system sollicitated by dynamic hydraulic conditions (cyclic systems, suspended particles, flow in both direction,...) behaves differently than one where a stabilisation of the soil has happened and where the hydraulic sollicitaions are continuous. All these situations suggest that filtration criteria must be different.

PORE SIZE CHARACTERIZATION

Decrepencies observed between results obtained from wet and dry sieving techniques utilised suggest that an investigation should be done to gather and analyse these results to try to corrolate them with actual geotextiles' structures. This is a very complex problem because the ability of particles to pass through thick non-woven geotextiles for example is function of the distance between fibres and the thickness of the fabrics, the electrochemical forces between the soil particles and the fibres, the fibres' surface treatment, the nature of the soils, and many other factors. It should be noted also, that these sieving methods are applicable only to geotextiles under uncompressed state. To what extent can the obtained results be applied to fabrics installed in geotechnical works where the fabrics are under appreciable loads?

COMPOSITE GEOTEXTILES

In recent years, more and more composite fabrics are used for specific needs in civil engineering works. They are constructed of a) two or more layers of different fibres with varying diameter, b) woven fabric incorporated in non-woven structure, c) non-woven fabric thermically treated or strenthed with chemical bounding agents and d) others. Very few investigations were performed on these geotextiles to forecast their filtration behaviours.

OTHERS PROPERTIES TO INVESTIGATE

The wettability of geotextiles should be investigated to determine the critical water head needed to insure flow of water through fabrics. This property is critical in many drainage works.

The siphoning ability to transport water within the plane of some geotextiles from capillary effect is very important in some applications.

Long term filtration and drainage experiments must be done with soils very hazardeous in order to establish " le bien fondé " of using geotextiles in these cases. Also in many applications, the installation procedures must be performed with great care to protect the work such that more experiments must be performed on filtration of suspended particles.

Finally, as indicated in CEDERGREN paper, emphasis should be given to overcome psychological hang-ups by publicizing the necessity of using fabrics to increase drainage capacity of civil engineering works using well documented case studies.

These fields to investigate and others are schematically represented on Figure-5.

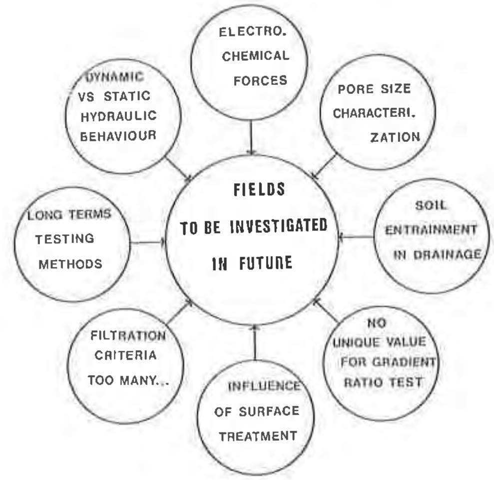

Fig. 5 Fields to be investigated in the future.

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Report on Sessions 6A, 7A, and 8A Dams and Erosion Control

Rapport des sessions 6A, 7A, et 8A Barrages et contrôle de l'érosion

INTRODUCTION

One session at this Conference was devoted to dams and two sessions to erosion control, with a total of 18 papers in these. Of these, 12 papers described various aspects of major permanent works whilst 6 papers were devoted to field trials. The types of projects covered were dams, tailing dykes, temporary dykes in the form of "super sandbags", canal bank linings, scour protection in harbours and land reclamation works, control of wind erosion of sand and lastly, surface water erosion of slopes. This exemplifies the large range of problems in which geotextiles are now being used and the wide geographical distribution of the various works described also illustrates the growing awareness of the potential for geotextile usage all over the world.

In the following sections of this report, I shall briefly summarise the papers in the various sessions and identify some of the main points made in them. On the bases of these points, I shall attempt to draw some overall conclusions from this part of the Conference and make some recommendations for future work in these areas of geotextile application.

SESSION 6A - DAMS

The papers by BENTAL et al and SCHEURENBERG described the use of geotextiles within drains incorporated in tailing dykes in South Africa. Both highlighted the problem of ferrous compounds in the groundwater becoming oxidised near the surface and ferric precipitates then forming on the geotextiles with consequent reduction in their hydraulic conductivity. Apart from this one problem, both reported satisfaction with the use of geotextiles and indicated that drains which included them were chosen as they represented the cheapest and simplest method of construction. Much of the paper by HAAS indeed dealt with the subject of selection and use of geotextiles in tailing dykes. At the end of it he sets out a very useful check list for this in which he emphasises the need to modify construction procedures and choose appropriate construction plant for dykes in which geotextiles are employed.

The papers by LIST and LEDEUIL both dealt with dam projects. In the cases they reported, the use of geotextiles allowed various local fill materials to be incorporated which otherwise might not have been. The role of the geotextiles was primarily to prevent piping of water through the dams. Both authors carried out their designs in the mid-1970's and it is perhaps surprising that geotextiles were considered for use in such important structures so early in their development.

BOGASSIAN et al dealt with the use of geotextiles in the form of large cylinders to form low cost temporary dyke constructions in Brazil. They described in detail the problems of filling them and the solution they developed for this.

SESSIONS 7A and 8A - EROSION CONTROL

The use of geotextiles to prevent surface erosion by wind was described in the paper by AURAIT. He stressed the importance of following up the initial stabilisation of the eroding surface with a vegetation planting programme in order to ensure the long term benefits of the works. Such a combination of geotextiles and vegetation for long term slope stabilisation was also recommended by SMOLTCZYK and MALCHAREK but using different geotextiles and plants and in situations where steep slopes were required. SIMON et al also dealt with stabilising the surface of relatively steep soil slopes. In their paper they type of geotextile used was of a relatively new honeycomb construction.

River and canal bank protection was dealt with in four papers. In the paper by COUCH and that by VELDHUIJZEN VAN ZANTEN and THABET the approach to major protection works was described. This involves full scale tests and detailed consideration of the influence of soil types and construction techniques on long term performance. These papers contrast sharply with those of BOTZAN et al and STEPHENSON which both look at the use of sand bag type protection techniques. These are essentially low cost and relatively short term solutions but nevertheless serve a useful function.

GYSSELS discussed in his paper the use of geotextile within conventional willow fascine mattresses to protect the sea-bed against scour on the seaward side of Zeebrugge Harbour in Belgium. This project lies immediately south of the mouth of the River Schelde. The remaining four papers in this section in fact all dealt with the massive Oosterschelde Storm-Barrier Project and in particular with the design, development and application of the scour protection mats used in this project. VISSER and MOUW first gave an overview of the project and the functions of the scour protection mat used within it. DORR and DE HAAN then described in detail the construction of the mat and the gravel bags used to protect the mortar between the massive storm barrier piers and the mat. Next VAN HARTEN described the analysis and testing associated with the mat whilst WISSE and BIRKENFELD dealt with the long term stability of the polymers in the geotextiles used.

DISCUSSION

Emerging from the papers on major projects within these sessions is the fact that they were generally conceived and designed in the mid to late 1970's. Considering that the Paris Conference on geotextiles was not held until 1977 it is surprising that the decisions to use them in such large projects could be made. The amount of technical data on the geotextiles and the availability of design techniques was extremely limited at that time. Perhaps this is why field trials prior to final construction were commonly employed. These field trials also assisted in the development of suitable construction techniques and the identification of the importance of these is one of the major contributions of this session.

Once appropriate construction procedures had been developed many of the authors commented on the ease of construction with geotextiles. A few of the papers on major projects and trials did not go beyond this and did not therefore consider the long term behaviour of the structures. Of those that did consider long term behaviour few identified any major problems. One recurring factor influencing long term filtration/hydraulic conductivity of the geotextiles did, however, emerge and that was the clogging of pores with iron compounds emerging as precipitates on the geotextile close to groundwater surfaces at shallow depths in soil.

The trials reported appeared to have two functions. The first function was to prove the use of geotextiles of fairly standard type in sections of major works prior to the adoption of this form of construction for the entire project. The development of construction techniques appears to be the principal benefit of this. The second function is to demonstrate the use of new geotextiles. Again the development of construction techniques appears to be the main objective of these. In all the cases reported one common feature was the lack of instrumentation to monitor performance.

Taking these various points together suggests that the papers in this section are almost exclusively dealing with the practical aspects of the inclusion of geotextiles in soil structures. Since the projects reported were conceived in the mid to late 1970's, there was perhaps little else to be reported. Few theoretical studies or testing data were available at that time. Thus there is a distinct time gap between the practical work in these sessions and the research and testing work reported elsewhere in the Conference. This is inevitable, but the fact that only in a few of the projects was any attempt made to measure the long-term performance of the soil-geotextile systems, was not inevitable and is certainly not desirable. In view of the novel nature of the form of construction used, the systems' performance should have been monitored. Thus in most of the cases reported it is likely that little warning of possible failures will be available and the causes of such failures will be very difficult to diagnose.

CONCLUSIONS

1. Some massive and high risk projects have now employed geotextiles to perform functions which must be sustained long term. Those projects reported to this Conference date back to the mid to late 1970's and there is therefore little or no use made in these of the research work reported to this Conference. The judgements on which these goetextiles were employed long term were thus based on very limited data.

2. The sessions were dominated by descriptions of construction techniques developed during field trials prior to major works or developed while proving new types of geotextile materials or applications. These all showed that once appropriate construction techniques were identified, construction with geotextiles was usually free from major difficulties. However, few of the trials and full scale works contained instrumentation to monitor the long term performance of the systems built. No major failures were reported and few problems were anticipated, the latter perhaps due to the lack of instrumentation.

3. On the bases of the data presented in the papers and the discussions on these at the Conference a number of recommendations for future development in these areas can be made. These recommendations are as follows:

a) That practising engineers continue to incorporate new construction ideas and materials in their projects after carrying out field trials as so many of the authors to these sessions have done. Only in this way can progress in construction technology be made.

b) International and national groups and associations should promote educational and communications links between researchers and practising engineers to provide the latest technology to the practising engineer at the earliest date possible and thus increase the bases on which practical judgements are made.

c) Failures should be sought out and widely reported. This might be encouraged by having a session on failures at the next conference on geotextiles.

d) That every effort be made to encourage the development of instrumentation to monitor the performance of geotextile systems over long periods of time. The performance of soil-geotextile systems should always be measured against alternative systems and performance assessments should be made in terms of technical performance, capital costs and maintenance costs.

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Report on Sessions 2C, 3C, 4C, and 5C Walls and Foundations, Slopes and Embankments

Rapport des sessions 2C, 3C, 4C, et 5C Murs et fondations, Pentes et remblais

This report covers the papers submitted to and discussed at Session 2C through 5C of the Conference. Session 2C focused on the use of geotextiles in walls and foundations, while the others covered various aspects of slopes and embankments. Several papers reported on properties of geotextile-reinforced soils. In almost every case, the primary function of the geotextile was that of reinforcement, although secondary functions of especially separation and drainage were apparent in some instances.

Of the 28 papers published in Volume III of the Proceedings of the Conference, 23 were presented orally in the technical sessions. In the period following formal presentation, many excellent questions were asked by members of the audience, and in most sessions, the discussion was quite lively.

I have classified the 28 papers into the following four categories. The number of papers primarily in each category is also indicated.

Case Histories (actual working construction or full scale tests): 11

Field Tests: 1

Laboratory Tests (including Model Tests): 7

Primarily Theoretical/Analytical: 9

CASE HISTORIES

In my opinion, the value of a case history to the profession is in the detailed description of (1) a non-conventional approach to the problem, either in terms of the design or the construction techniques utilized; (2) the design assumptions, and (3) verification of the design assumptions. Verification can be either quantitative, that is from field measurements, or it can be qualitative, that is, based on a description of the performance. It was disappointing that a number of case histories presented at the Conference did not adequately consider these three requirements.

Two of the ten case histories were, in my opinion, excellent in all respects, and both involved embankments. The first was the Almere embankment test in Holland (Brakel and co-authors, p. 727), which is a particularly well-documented report of a full scale loading test carried to failure. A companion unreinforced embankment was similarly instrumented and tested. A simple method for calculating the stability was proposed and good agreement with observations was found.

The second well-documented case history was from Mexico (Olivera, p. 625), where two embankment test sections were constructed on peats and other highly compressible soils. In each case, a non-reinforced test section was also constructed as a control. Although specific design calculations are not given, the construction and instrumentation are particularly well described. Results of the measurements and the overall behavior of the test sections suggest that (1) the fabric may have acted more as a separation layer and construction aid rather than strictly as reinforcement; (2) the stress distribution under the embankment was altered by the reinforcement; and (3) the settlements were made more uniform by the geotextile.

Space does not permit a detailed summary of the other case histories, so only a few pertinent comments will be made. Murray's presentation (p. 707) of an innovative slope stabilization technique was an excellent description of the practical execution of the project. He also provided a simple design procedure.

Papers with innovative construction techniques for embankments on soft ground included those by Barsvary, McClean, and Cragg (p. 647), Hannon (p. 653), and Yasuhara and Tsukamoto (p. 635). These authors also gave some useful performance data, while the latter paper described a design method based on model laboratory tests. One anomaly that calls for additional investigation is the decrease in fabric elongations in both directions with time observed by Barsvary, et al. (Fig. 4, p. 649). This phenomenon would seem to be appropriate for a detailed analysis by the finite element method.

The Wolfe and Christopher paper (p. 641), although primarily descriptive, deserves careful consideration by engineers faced with all phases of waste disposal. The subject is of increasing importance in the industrialized world. The paper could have been placed in other conference sessions, as it deals with several geotextile applications besides embankment stabilization.

Most papers on geotextile-reinforced retaining walls were disappointing, because the design assumptions were missing and good follow-up measurements and observations were often lacking. The paper by John and his co-workers (p. 569) described the rather impressive instrumentation of a Websol-reinforced wall. The Websol system appears to be similar to the reinforcement systems of the Reinforced Earth Co. and the York method of construction. It is not clear what function the geotextile itself performs, since it is not attached to the wall face. Coincidentally, the inventor of the York method was Jones (p. 581), who describes the construction of walls reinforced with both geotextiles and geogrids. The large scale retaining wall tests described by Fukuoka and Imamura (p. 575) were well-instrumented and utilized cohesive backfills--but the fabric was used only as the facing. The primary reinforcing elements were multiple anchor "tie backs". Walls with fabric facings were also described by Schwantes (p. 605), but since no design details or measurements were given, the value of the paper to future designers is unfortunately minimal.

FIELD TESTS

It was disappointing that there were not more papers like Hutchins' (p. 617) who used an in situ test, a plate load test, to evaluate the field performance of a geotextile-reinforced embankment. In situ tests are becoming increasingly important in geotechnical engineering practice, and their use to assist in both the design and the evaluation of geotextile performance should also increase.

LABORATORY TESTS

Valuable data on certain properties of cohesive soils reinforced by sands was reported in the papers by Christie (p. 659) and Ingold and Miller (p. 587 and 593). Such data is essential if fills are to be constructed of compacted cohesive soils. Akinmusuru, Akinbolade, and Odigie (p. 599) utilized a natural fiber rope material instead of man-made fibers in model footing tests on reinforced sands. These tests were very similar to some reported by the same authors in the Journal of the Geotechnical Engineering Division, ASCE, June, 1981, and by Binquet and Lee, same journal, Dec. 1975. Because of problems with durability, there is considerable question as to the practicality of using natural fibers as reinforcement for permanent foundation construction.

A rather fundamental study of sands reinforced externally (encapsulation) and internally by layers or sheets was reported by Gray, Athanasopoulos, and Ohashi (p. 611). Encapsulated soils have been of interest to the U.S. military for expedient construction of roads, bridge abutments, etc. It is not clear how encapsulation and layering of foundation elements can be practically carried out in the field, although Gray suggested that mandrels might be employed in soft clays.

The laboratory study by Leflaive (p. 721) will be discussed separately later in this report.

Much can be learned by laboratory-scale model tests. Already mentioned was the design method developed by Yasuhara and Tsukamoto (p. 635) for embankments on very soft ground. Papers reporting on the use of model tests to verify theoretical computations include those by Petrik, Baslik, and Leitner (p. 631) and by Andrawes, et al. (p. 695).

THEORETICAL/ANALYTICAL WORK

Papers on theoretical/analytical work are mentioned last, since it is here that I feel the most progress has been made since the Paris Conference in 1977. There have been some exciting developments in this area.

Four papers report on research utilizing analytical techniques such as the finite element method (FEM) or finite differences. These techniques, especially the finite element method, require large digital computers for solutions. From the results of these analyses, behavioral models of the performance of reinforced embankments, slopes, and walls are obtained. Blancier and Gielly (p. 621) used finite differences to investigate the stability of reinforced embankment slopes. Rowe (p. 677) and Petrick and his co-workers (p. 631) used the FEM to investigate the behavior of geotextile-reinforced embankments on soft foundations. Jeyapalan and Lytton (p. 701) in a study very similar to that carried out on circular culverts by Nowatzki, Sanan, and Sogge (1980, Portland ASCE Convention), used the FEM to investigate the effect of geotextile layers over flexible metal box culverts. In both papers, the geotextile overlying the crown of the culvert tended to reduce such stresses and crown deflections. The final FEM paper was by the "Strathclyde Mafia" of Andrawes, McGown, and their students. The response of a footing resting on a dense sand reinforced at different depths with a single layer of geotextile was investigated. Predictions were excellent up to about 85% of maximum load, provided the stress-strain behavior of the sand could be characterized accurately.

Several authors proposed design methods and procedures based on rather conventional limiting equilibrium principles for, primarily, embankments on soft foundations. Noteworthy in this regard are the papers by Fowler (p. 665), Jewell (p. 671), and Ingold (p. 683). Similar techniques were applied by Murray (p. 707) to the slopes of embankments and excavations and by Christie (p. 659) to the slopes of dams and embankments. These methods are simple and easy to understand, and they require only "back of the envelope" type calculations, or calculators or small computers which today are found in every CE design office.

As Peter Jarrett, Leader for Session 4C, remarked: we now have a sufficient number of design methods available so that we can begin to check some of the case histories we've collected since the Paris Conference.

Finally Bell, Green, and Laverty (p. 689) give a convenient check list for the properties and other important considerations affecting the choice of geotextiles for most reinforcement applications.

MOST INNOVATIVE IDEA

Finally, for the most innovative paper presented at these sessions of the Conference, I would like to nominate the paper by Leflaive entitled "Reinforcement of Granular Materials with Continuous Fibres and Filaments" (p. 721). This paper reports on developments since the suggestion by Hoare (1977) at the Paris Conference. The idea is "far out" but promising, and the paper is highly recommended.

CONCLUSIONS

We have made significant progress in the years since the Paris Conference. Our analytical techniques and capabilities have markedly increased, and even relatively simple design procedures have been proposed with apparent success. We still lack reliable geotextile-soil property data for our design calculations. How to appropriately characterize the stress-strain relationships for the soil-geotextile system is not fully established.

The introduction of reinforcing elements in cohesive soils is still in its infancy, but important work on this aspect has been reported at this conference. Normally, we aren't used to being very concerned with the rheological properties of sands (as we are with clay soils), but, such behavior becomes important when granular materials are reinforced with geotextiles.

The durability of especially geotextile wall facings is still a problem, but it is hoped that designers can turn to papers in Session 8B for some assistance. Some of the durability problems with geotextile wall facings could be ameliorated by the development of simple repair procedures for such installations.

Other still-to-be-solved problems remain. Scale effects between laboratory tests and the field plague conventional geotechnical engineering practice, and the introduction of geotextiles into the system doesn't simplify matters any. Additional research in this area is greatly needed.

The continuing publication of full scale field trials and other case histories is encouraging. However, greater attention must be paid to the characteristics of a good case history given earlier in this report. Better design information and detailed reports on how well the design assumptions checked field observations are crucial to the proper evaluation of a case history.

It is hoped that a greater use of in situ tests will take place in the future to assist in site characterization as well as the evaluation of geotextile performance.

Finally, geotextiles as reinforcing and stabilization materials for increasing the stability of especially embankments on soft ground, earth retaining structures, cut slopes in natural soils, and high embankment slopes are significant additions to the techniques usually available to geotechnical engineers for such problems. This point was established only tentatively at the time of the Paris Conference. Today, only five years later at the completion of the Second International Conference, geotextiles as reinforcement are firmly established in civil engineering practice.

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Report on Sessions 3B, 4B, and 5B Unpaved Roads

Rapport des sessions, 3B, 4B, et 5B Routes non revêtues

This subject has been dealt with in three sessions comprising 18 contributions from manufacturers, engineering companies, authorities and research establishments.

The system treated is largely granular layer on top of a soft subsoil. This top layer could be an embankement on a soft soil, the embankement being protected by geotextile, or an unpaved road layer which is separated from the subsoil by a geotextile. Application of geotextiles for stabilization of retarding walls has also been demonstrated.

The basic technical effects of geotextiles are:

- drainage
- separation
- filtration
- reeinforcement

The drainage aspect has not been given much attention in the present contributions, although it has been established that there must be drainage. The geotextile can for instance not be replaced by an impermeable membrane, although consideration of mechanical properties may suggest it.

It is agreed by the contributors that there are two reinforcing effects

- membrane or catenaria effect
- lateral restraint of aggregate

The latter implies that under the influence of wheel loads the granular material outside the action area of the vertical forces from the wheel are restricted from lateral motion and hence engage a wider area of the granular layer to participate in load spreading. Such invasion of soil will be prevented in the presence of geotextiles, i.e. the separation effect will in a sense protect fatigue life.

The membrane effect is a result of the forces at right angles to the membrane, resulting from membrane tension. They are according to elementary laws of mechanics in in the first approximation proportional to the tension and inversely proportional to the radius of curvature) of the fabric, which means that an undeformed fabric (i.e. lying on a flat subgrade surface) would yield no membrane effect of this kind.

An additional effect of the membrane is protection against certain mechanisms, which are suggested to occur in repeated loading. These mechanisms are

- tensile cracking of the granular layer
- punching out of the loaded section of the granular layer

At each load repetition a small amount of soil material will enter the tensile cracks and the result will be reduced load spreading, and finally the load spreading function of the granular layer will cease. Punching of the granular layer section adjacent to the loading wheels, which occurs at heavy loads, will also allow soil material to enter the cracks in the granular layer with the same result as indicated above.

Mathematical models of the load spreading and membrane effects following a mechanistic approach according to the fundamental properties indicated above have been derived by several authors. Assumptions then have to be made regarding the mechanical behaviour of the soil as well as the fabric, and the fit of such models is strongly related to the assumptions made. Non-linear behaviour of soil has to be considered, as well as timedependent behaviour of the fabric. FEM-models are also included. Model experiments as well as full scale field tests have given excellent agreement between computed and observed behaviour.

A model implying an additional fabric halfway down the granular layer has also been considered and found successful in improvement of bearing capacity.

From the models it is possible to find the parameters, which are required in road design. The importance of proper design in application of geotextiles has been emphasized by several authors.

Fabric parameters of primary importance are:

- tensile modulus
- tensile elongation at break
- tensile stress at break.

There is not full agreement on the relative importance of these parameters. For the catenaria effect a high modulus is required but also a high breaking length, since the catenaria effect is favoured by high deflection. The question of woven or non-woven fabrics has a bearing on the relative importance of these parameters. A medium level modulus was stated as preferable, and wovens are ruled out by some authors.

The importence of anchoring the fabric has been emphasized by some authors and also supported by theory. The catenaria effect actually means that the granular layer partly hangs in the fabric.

One author has in a model experiment tried circular fabrics of different diameters without anchoring and found a considerable load spreading effect when the diameter was greater than that of the bottom of the "load spreading cone".

In one paper an approach based on energy consideration was suggested but not proved and in another paper a probabilistic approach was worked out, which considers failure of a fabric system due to fabric failure and fabric pull-out. Experimental verification of the theory was not reported.

Contribution to bearing capacity based on the catenaria effect requires rather heavy rutting, since the forces at right angles to the fabric require zero curvature. This effect can therefore not be utilized in permanent roads where heavy rutting is not tolerated. This could however be overcome by allowing for some rutting in an early stage followed by an early overlay.

Considerable benificial effect of rut repair has been reported. The most important bearing capacity effect in permanent roads - if there is one - will however be derived from the protection against soil invasion in fatigue cracks mentioned above. Additional indirect effects come from separation and filtration.

Papers dealing with separation and filtration properties confirm quantitatively the existence of such properties and their relation to fabric properties. Filtration of much smaller particles than the effective characteristic pore size occurs, and it is established that penetration of soil particles occurs at contact points between fabric and aggregate. Considerable penetration has been observed only when fabrics have been torn.

There has also been established a relation between contamination and pore pressure dissipation time.

It is quite possible to have a geotextile replace the granular separation layer on the subgrade, specified by some authorities. For this purpose a thick relatively incompressible fabric with appropriate filter properties has been recommended.

It is agreed that the use of fabric is meaningless at CBR > 1 %. Bearing capacity therefore should be measured by some other test such as the vane test.

Experiments with freeze-thaw cycles indicate that near the liquid limit small changes in water content may be critical for the functioning of the geotextile.

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Report on Session 5B Special Applications

Rapport de la session 5B Applications spéciales

This session presented to the audience a great diversity of topics but one theme found in three of the papers was the use of geotextiles as flexible forming systems. When made as an enclosed form, the geotextile can be filled with neat cement grout, cement rich mortar, plasticized concrete, resinous grout, flyash/cement mixes, asphaltic grout, sand or, even, bentonite clay. The Lupton, Brandl and Welsh and Dominske papers all treated this general topic. Lupton presented three case histories, the major one being the use of a double layer geotextile to form an underwater strut between two sheet piles. Brandl illustrated the use of geotextiles to form insitu piles which could eliminate negative skin friction and for use as sand filled "stockings" to accept radial drainage as in conventional sand drains. In this latter case, the vertical drain now has considerable tensile resistance with which to resist a possible shear failure. The Welsh and Dominske paper traces patents on flexible forming systems and then proceeds to summarize their extensive experience via a number of case histories. These are the following:

1. As mattresses for erosion control structures whereby a double layer of fabric is inflated by a cement grout conforming to the soil slope on which it is formed. Pore pressure dissipation can be made by the inclusion of filter points within the mattresses.

2. As a pile restoration technique, fabrics have been wrapped around deterioriated piles in the form of a jacket and pumped up with cement grout (even reinforced cement grout). The finished product often has a strength greater than the original pile.

3. As in-situ columns in abandoned mines and limestone cavities, fabric tubes have been inserted in predrilled holes and inflated with cement grout. The tubes yield where there is no resistance, i.e. the void, and the final deflected shape gives support where it is required. They can be placed on as close a spacing as deemed necessary.

4. As tubes or bags for concrete placement underwater. Such areas where scour has occurred beneath bridge piers, or beneath pipelines, or for good load distribution of piers or footings, have been areas of particularly successful applications.

The Clough and French paper is a departure from the above in that a geotextile/plastic insert/geotextile composite material was used as a capillary moisture break. Its use was illustrated in arid areas of Saudi Arabia where the near surface salt laden groundwater raises via capillary force to the ground surface. Here it destroys vegetation, agriculture and even road aggregate and building foundations. The authors conducted a series of small scale, then large scale tests to see if their capillary break composite geotextile would eliminate the problem. Based on the positive results of these tests, field trials are now ongoing.

Purdon and Resal present their experience with the use of geotextiles in athletic fields. Their technique is to encapsulate a free draining gravel between two geotextiles with a 6 cm soil cap above. The primary function of the system is drainage, but proper flexibility is also significant. In this latter regard, a "sportest" has been devised whereby a displacement transducer is deployed and attached to a storage oscilloscope to give a real time trace of both the deformation and rebound as an athlete traverses the surface. The interface with biomechanics and biomedical engineering in the proper tuning of an athletic field is of great significance.

This reporter found the session most exciting and stimulating as far as the incredible range of geotextile applications that exists. Those major items which come to mind are:

- Flexible form techniques for new construction activities.
- Flexible formwork techniques for all kinds of remedial construction problems.
- Use of a wide range of composite geotextile systems, designed and manufactured for specific applications.
- Development and deployment of insitu tests to instantaneously assess and monitor the results of a specific geotextile installation.

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Report on Session 6B
Railroads

Rapport de la session 6B
Chemins de fer

This session produced six papers, of which three were laboratory oriented and three were field oriented.

Regarding the laboratory oriented papers, the Saxena and Chiu and Friedli and Anderson papers both used large size triaxial specimens to assess the reinforcement ability of geotextiles in soft soils. Each found a pronounced improvement in the aggregate/geotextile/soil system over a standard aggregate/soil system. This membrane type reinforcement effect was evidenced by an increase in deviator stress, an increase in resilient modulus and a decrease in elongation at failure. At a larger, but still laboratory, scale, Saxena and Wang evaluated a simulated railroad track, finding that the deflection of the ties were reduced and that lower strains in the subgrade were observed when using a geotextile. A major finding was that the ballast/geotextile/soil system can be modeled using elastic theory. Thus existing numerical techniques, e.g., finite element models, can be utilized or appropriately modified for use.

Regarding the field oriented papers, Raymond's provides excellent detail in regard to abrasion resistance versus depth of ballast beneath the railroad tie when the geotextile is placed in the system. Here an optimum depth of ballast of 12", with a minimum value of 10", is recommended. Primary functions of a geotextile in railroad work (according to Raymond) are listed as:

- lateral drainage
- abrasion resistance
- separation
- degradation resistance

Clearly stated is that tensile reinforcement is "not a primary requirement in track rehabilitation work". Raymond further recommends that the geotextile should be needle punched, acrylic resin dipped and force air dried with an equivalent opening size (EOS) of 200 or, as small as, 400.

On the basis of past field experience, Newby comes to the conclusion that the ideal railroad geotextile is a needled polyester nonwoven fabric of nine denier filament, 6" long fibers with a minimum tenacity of 4 grams per denier. Lateral drainage is emphasized in the text of the paper in addition to filtration, separation and tensile strength. In his field oriented paper, Fluet lists a variety of geotextile properties as being critical. These are abrasion resistance, lateral drainage, burst strength and puncture resistance. His paper is basically a checklist, or guideline on how to assess a geotextiles' performance in railroad stabilization. It presents excellent comments on the proper setup of a field monitoring program.

In reviewing these papers, and listening to their presentations, a number of generalized comments can be offered.

1. Laboratory work is emphasizing membrane type reinforcement, whereas field work relegates this function to lesser importance than drainage, abrasion resistance, and other functions.

2. Laboratory work emphasizes woven slit film polypropylene geotextiles, whereas field work seems to be using rather thick and heavy (as high as 29 oz./yd^2, as per Raymond) needled polyester or polypropylene fabrics.

3. Field work places drainage, and in particular lateral drainage (thereby using the geotextiles' transmissivity characteristics), as the major geotextile function. This obviously requires a thick or bulky geotextile, hence the needled nonwoven recommendation by the authors of the field oriented papers.

4. Neither laboratory nor field oriented papers make note of the lateral restraint of the ballast given by the geotextile. This reinforcement type of action (albeit not of the membrane type), has appeared in the literature by authors other than those represented at this session.

5. The general area seems to be in desperate need of quantified field data. Fluet makes note of this need in his paper and of the need to publish the information in the open literature. These comments are heartily endorsed by this reporter.

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Report on Session 7B Paved Roads

Rapport de la sessions 7B Routes revêtues

This session contained seven papers, of which five concentrated on the use of geotextiles in crack reflection prevention. In this application an existing pavement, which is obviously distressed, is covered with a suitably treated geotextile and then paved over with a bituminous layer. Benefits are hoped to be realized by either using an equivalent thickness of overlay pavement as with no geotextile, for longer lifetime; or to use a thinner thickness of overlay pavement, for an equivalent lifetime. In this regard, the design technique offered by Majidzadeh, et. al., is of considerable interest. The technique is based on fatigue life enhancement by the fabric (as per laboratory tests) and both fatigue and rutting distress are incorporated. The authors of this paper state that the method must be field evaluated.

Regarding laboratory testing of geotextiles used to prevent crack reflection, papers by Button et. al., (who evaluated 8 fabrics in 6 different types of laboratory tests) and Murray (who evaluated 5 fabrics in 2 different tests) were presented. Both found a decided improvement in flexural fatigue by factors of up to 12 and 15 times, respectively, over nonreinforced control specimens. The most relevant fabric property seems to be the geotextiles' stiffness which in one case, i.e., Murray's, is best estimated by its 5% secant modulus.

The field assessment papers, there were two of them, both showed improvement in retarding crack reflection when using geotextiles. Colombier, et. al., have observed five such sections which were constructed as early as 1977, and found that crack propogation is retarded by about two years over nonreinforced sections. Furthermore the cracks, when they do appear, are more diffused and branched when a geotextile is used. Caution is expressed regarding poorly fit fabrics and fabrics that are too compressible. Both situations lead to accelerated crack damage. The Hugo, et. al., paper presents a ten year study on a major freeway on the use of a very open mesh woven polyester fabric with a high modulus. The work was based on earlier laboratory and small pilot scale tests which illustrated the advisability of a "bond breaker" over the initial crack. This bond breaker was best realized by placing a narrow 5 mm thick fine sand over the crack before placement of the geotextile. Presumably, high stress concentrations are avoided by this technique. The crack pattern has been nicely quantified and, after eight years of service, is now accelerating in its deterioration. Adjacent nonreinforced sections have long since cracked and failed. Deflections were critically evaluated and their conclusion is that the fabric worked this long because of the prevention of ingress of water to the structure and not due to increasing the structural capability of the system. Therefore, Hugo, et. al., concluded that fabric in crack reflection work is a waterproofing mechanism, not a reinforcement one.

Two other papers in this session dealt with low volume roads and, although appropriately placed in this session on paved roads, they function quite differently than in the above described crack reflection papers. Leflaive, et. al., describe a surface coating method whereby a composite system of subgrade soil/oil treatment/geotextile/oil treatment/bituminous wearing course is utilized. The geotextile imparted a cohesiveness to the system (also confirmed by simulated laboratory tests) and is definitely used for its reinforcement capabilities. It will be field tested under actual traffic conditions in the near future. The Lawson and Ingles paper presented two case histories involving membrane encapsulated soil layers (MESL). Here an unsuitable soil, e.g., high water content clay, expansive clay, frost susceptible soil, etc., is completely wrapped in a geotextile. The geotextile is treated to reduce its permeability to the point where only minor amounts of water can pass (or better, where only water vapor can pass) thus assuring controlled moisture in the MESL. Basic functions are separation, reinforcement and moisture control. Both case histories presented in this paper are excellently documented and evaluated. The first site had no failures in either the MESL or control sections. It is important to note, however, that the MESL section was tapered down to as little as 100 mm in thickness and still performed as good as the control section. In the second site the MESL performed excellently, but the control sections (on each side of the MESL) were completely failed. The mechanisms involved are due to separation and reinforcement since moisture was not involved.

By way of general comments, this reporter feels that in the use of geotextiles in reflective cracking prevention, the basic mechanism is still uncertain. Do geotextile reinforce or not? Laboratory tests all show a definite reinforcement effect, while field tests generally do not show reinforcement. Thus, in the field, moisture prevention, and its subsequent negative effects, seem to be the prime mechanism contributing to improved performance. Obviously, controlled field tests are definitely needed, as basic insight into the mechanisms taking place is still not available.

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Report on Session 8B Durability

Rapport de la session 8B Durabilité

To every owner, public agency, or responsible party of a geotextile involved construction project, the question of durability is essential. Basically, one demands to know if the geotextile will deteriorate with age, and, if so, what is the nature of this deterioration. Thus, it is our duty, to evaluate the phenomena involved and to report the findings. This session was addressed to this situation. The fundamental phenomena involved are the following:

- direct and indirect ultraviolet light degradation
- chemical attack and aging
- biological attack and aging
- long term burial degradation

Although the number of variables involved is tremendous, this session provided considerable insight into the various aspects of the problem.

On UV degradation, Raumann reported on eleven fabrics tested in outdoor exposure in Florida, Arizona and North Carolina. Samples were withdrawn for up to 52 weeks (or failure) and tested for grab strength and elongation. Almost all types of fabrics were represented with weights ranging from 100 to 360 gm/m^2. Results were categorized into the following groups: (a) poor performers (severe strength loss in 24 weeks or less), which were the bonded polypropylenes, (b) intermediate performers (severe strength loss in 24 to 40 weeks or less), which were the woven slit films, and (c) long term performers (negligible strength loss after 32 weeks or longer), which were the needled polyesters and the woven polypropylene monofiliment fabrics. In general, low strength was usually accompanied by a loss of elongation, i.e. the fabrics became brittle, which was excellently illustrated by means of scanning electron micrographs. Raumann cautions that in no case should a geotextile be left exposed to light for permanent installations.

On polymer aging, Sutton and Leclercq evaluated a wide range of conditions and compared these by means of tensile tests and scanning electron micrographs. Effects are shown on the basis of fundamental polymer type and exposure time. Outdoor exposure tests were conducted which showed weakening of the fabric, but burying the fabrics in basic, acid and sea water had no negative effects.

On biological degradation, Ionescu, et. al., evaluated six fabrics (four polypropylenes, a polyester and a composite) in eight media for times of 5-17 months. The media were distilled water (the control test), sea water, compost, soil, iron bacteria, levansynthesizing bacteria, desulfavibrios bacteria, and a liquid mineral. Evaluation was by permeability, wide width tensile strength and infrared spectroscopy. Results show no measurable effect on permeability (although some roots were growing in the fabric), only small variations in tensile strength and no structural changes visible on the infrared spectroscopy.

On burial degradation, Sutton, et. al., looked at exhumed samples which has been in place for up to twelve years. Mechanical properties were emphasized in the paper (hydraulic properties are published elsewhere) and these were found to loose less than 30% of their strength over the time period involved. Considerable detail, on a wide range of geotextiles, in a wide variety of applications, is presented in this important paper.

By way of summary, this reporter feels that with required lifetimes of up to 100 years, the long term functioning of geotextiles is mandatory. The group of papers in this session was essential in spreading the word that chemical, biological and burial deterioration are not particularly detrimental to the typical construction geotextile. Conversely, ultraviolet light degradation deteriorates almost all types of polymers, and demands that construction activity be controlled to place the fabric in its final, and buried, position as soon as possible. For those cases where exposure of the geotextile is necessary, e.g., silt fences, polyester fabrics or woven monofiliment polypropylenes seem more resistant to light than others, which would give them the longest lifetime.

By no means, however, do the papers in this session close the door on this type of testing. The number of variables involved, the types of test, and the suitability of accelerated laboratory aging tests all need further inquiry. Future research efforts should be aimed in these directions.

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Report on Sessions 2B, 5C, 6C, and 7C International Standards, Properties and Tests

Rapport des sessions 2B, 5C, 6C, and 7C Standards internationaux, Propriétés et essais

In a general way are reported in this text :

- International and national Association's positions regardins tests, standards and properties of geotextiles.
- Progress made since the First International Conference on Geotextile in Paris, relating to research work on testing geotextiles. Two great tendancies have been ascertained :
  . tests on larges samples (up to 500 x 100 mm) ; tensile test, tearing, punching (practical and theorical approaches).
  . tests able to reveal geotextiles fitness to support long term stresses (creep, fatigue with or without presence of soil).

Sont rapportés dans ce texte d'une manière générale :

- Les positions des Associations nationales et internationales en ce qui concerne les tests, normes et propriétés des géotextiles.
- Les progrès effectués depuis le Colloque de Paris en ce qui concerne les recherches de tests. Deux grandes tendances ont pu être dégagées :
  . essais sur éprouvettes de géotextiles de grandes dimensions : test de traction, de déchirure, poinçonnement (aspects pratiques et théoriques).
  . tests susceptibles de révéler l'aptitude des géotextiles à supporter des contraintes à long terme (fluage, fatigue en présence de sol ou non).

Les tests sur géotextiles sont restés l'un des thèmes majeurs abordé lors de ce colloque de LAS VEGAS. On peut être tenté de faire des comparaisons par rapport au premier colloque de PARIS afin d'essayer d'apprécier, au terme de cette rencontre américaine, les efforts et progrès accomplis dans le chapitre des propriétés et essais.

Le colloque de Paris avait posé en termes très clairs un certain nombre de questions et laissé transparaître un certain nombre de préoccupations :

- souci de précision dans la dénomination et la description des géotextiles ;

- souci de caractérisation et qualification ensuite ;

- souci de recommandations ou spécifications enfin.

On peut affirmer aujourd'hui que des éléments concrets sont désormais apportés en réponse aux préoccupations exprimées à Paris. Ces remarquables résultats sont les fruits légitimes des rapprochements qui se sont produits entre géotechniciens et textiliens. Ces groupements qui, selon les pays, ont pris nom : Comité, Commission, Association, etc... sont venus s'exprimer en tant que tels à la tribune pour témoigner de leur existence, de leur dynamisme, mais aussi pour faire part des résultats de leurs travaux, de leurs décisions, de leurs ambitions et également parfois de leurs difficultés et de leurs interrogations.

D'une manière générale, les démarches réalisées par ces groupes apparaissent exemplaires :

- exemplaire déjà par la qualité des rassemblements qui se sont constitués un peu partout autour de l'idée.

- exemplaire ensuite par le profil des travaux qui ont été réalisés et qui ont permis de proposer dans certains cas :

  . un langage de travail, une terminologie ;
  . des méthodes d'essais pour la détermination des principales caractéristiques des géotextiles ;
  . des recommandations et spécifications pour les utilisations les plus courantes.

En ce qui concerne le langage d'abord, un certain nombre de définitions semblent désormais bien acquises et même si le néologisme "géotextile" fait encore l'objet de certaines réticences et interrogations, le large emploi qui en a été fait pendant la durée de ce colloque devrait définitivement faire tomber toute résistance. Ce langage a déjà permis, dans certains pays, de proposer des "fiches standards" de désignation et de description des géotextiles qui devraient permettre de lever toute confusion et ambiguïté au niveau des utilisateurs.

En ce qui concerne le problème général des tests et spécifications, il convient de souligner que, même si les différents auteurs qui se sont exprimés à ce sujet ont pu sembler diverger dans leurs prises de position, l'analyse plus fine des textes laisse néanmoins apparaître un "tronc commun" important :

. Tout le monde semble d'accord sur la nécessité de choisir des tests simples pour l'identification des géotextiles et une quasi unanimité s'est faite sur la sélection de tels tests, qu'il s'agisse de la mesure de la masse surfacique ou de celle de l'épaisseur...

. Tout le monde semble d'accord sur la nécessité de choisir des tests de caractérisation des géotextiles et, là encore, une tendance se dégage en faveur d'essais simples, reproductibles en traction et déchirure sur bande large, des mesures de perméabilité et filtration. Mais, par contre, des désaccords sérieux existent sur l'utilisation finale qui peut ou doit être faite des résultats de ces tests :

- pour certains, ces tests constituent essentiellement des méthodes de qualification des géotextiles et permettent simplement de suivre les différentes productions des fabricants en vue de comparaisons (aide à la décision de l'Ingénieur ou du Prescripteur).

- pour d'autres, ces tests apparaissent déjà suffisamment représentatifs, c'est-à-dire permettent une simulation suffisamment satisfaisante du comportement des géotextiles dans leurs utilisations les plus courantes, pour que les valeurs numériques obtenues aient une signification physique claire et utile pour l'Ingénieur, mais puissent aussi servir à l'établissement de recommandations et de spécifications. Dans la mesure où la restriction est posée, à savoir, que de telles recommandations pour les utilisations communes des géotextiles sont établies en se référant d'une part, à l'expérience acquise, et, d'autre part, aux résultats de ces tests, le désaccord semble déjà moins grand... Et, ces utilisations courantes représentent pour les producteurs plus de 90 % du marché !...

Enfin, et bien sûr, tout le monde est d'accord sur le fait qu'il est nécessaire d'établir des spécifications sur les bases d'essais particuliers, chaque fois qu'un géotextile est appelé à jouer un rôle majeur dans un ouvrage et, notamment, chaque fois qu'il doit assurer des fonctions à long terme. Evidemment, les essais particuliers doivent simuler au mieux les contraintes et l'environnement. Ce point a été bien senti par exemple par les équipes hollandaises, pour les problèmes de drainage vertical, pour lesquels, un essai standard s'avère difficile à proposer : les spécifications de drainage doivent être établies pour chaque type d'ouvrage. Par contre, dans beaucoup de cas, il n'existe pas encore de méthodologie acceptable :

- l'exemple du fluage est à retenir : aucune méthode ne permet d'étudier et prévoir le fluage tel qu'il se produit sur le terrain ;

- la prévision de la tenue au vieillissement chimique, photochimique, pose également les mêmes problèmes.

Il convient toutefois de souligner qu'au-delà de toutes ces discussions et réflexions sur les tests, certaines associations nationales et internationales sont très avancées dans leurs démarches d'établissement de méthodes d'essais et de recommandations d'usage des produits (cas de la Suisse, de la France, de la Belgique, de la Finlande, de l'Association Permanente Internationale des Congrès de la Route, de la RILEM...).

Pour en venir plus précisément aux tests, le Colloque s'est révélé fertile puisque 22 communications ont été présentées dans les sessions réservées à ce thème.

Sans rentrer dans le détail de ces communications, on peut rapporter ici deux tendances qui sont apparues lors de ce Colloque et qui correspondent à une deuxième génération de publications révélatrice à coup sûr de la maturité de la recherche sur les géotextiles :

- La première tendance est relative aux essais mécaniques sur éprouvettes larges, qui s'écartent donc délibérément des tests traditionnels textiles :

essais de traction tout d'abord sur des éprouvettes de grandes dimensions pouvant atteindre 500 x 100 mm. Test simple, relativement rapide et qui peut sans difficultés majeures être utilisé pour les géotextiles actuels aussi bien tissés que nontissés. Les notions matérielles de coût et temps d'essai n'ont pas échappé aux conférenciers qui ont pu constater que les nontissés, du fait de la dispersion de masse surfacique dans les différentes directions de l'éprouvette, exigeaient plus d'essais que les tissés, pour obtenir un coefficient de variation donné.
Néanmoins, ce type d'essai fournit des valeurs présentant une signification physique claire pour l'Ingénieur qui a besoin de faire des calculs de dimensionnement. Certains conférenciers se sont interrogés sur l'intérêt de choisir des caractéristiques mécaniques ultimes du géotextile, c'est-à-dire aux limites de rupture, caractéristiques qui sont dépendantes des formes et dimensions d'éprouvettes... La mesure du module à 10-20 % de la déformation de rupture, qui semble indépendant de la dimension d'éprouvette, serait peut-être judicieuse ?... Il convient néanmoins d'être prudent sur la valeur pratique d'un tel module mesuré sur un géotextile neuf, lorsque l'on sait que le module peut changer (amélioration) fortement à mesure que l'intimité sol-fibres devient plus grande à l'usage.
Plusieurs conférenciers sont allés bien au-delà de la simple affirmation de leur choix pour ce type d'essai "bande large" en en justifiant théoriquement la validité. L'essai de traction sur bandes larges permet de reproduire ce qui se passe en réalité dans l'association sol et textile, à savoir : les effets de particules de sol sur les fibres qui empêchent ces dernières de se déformer trop et qui limitent la réorganisation du géotextile. Les résultats de cet essai se situent donc très proches de ceux fournis par le test référence de ST BRIEUC, ou par ceux de tests conduits en présence de sol, ainsi qu'ils se pratiquent à l'Université de GLASGOW.
Toutes ces réflexions théoriques et pratiques sur le comportement en traction des géotextiles ont permis d'introduire un concept qui devrait devenir essentiel : celui de la plus ou moins grande aptitude ou résistance d'un textile à changer de "masse surfacique" sous l'effet d'une contrainte (résistance à la déformation surfacique). En passant, avec les modélisations proposées, on peut dire qu'à LAS VEGAS, la Mécanique des géotextiles a fait un bond en avant, qui devrait servir la cause de la Mécanique textile en général et aider l'Ingénieur dans la conception de produits très spécifiques.

essais de résistance à la déchirure ou à la perforation qui prennent également en compte des éprouvettes larges. Signalons à ce propos l'intérêt de machines d'essais dynamiques, telles celles construites au Laboratoire des Ponts et Chaussées en France qui permettent d'appliquer aux éprouvettes des déformations à des vitesses pouvant atteindre 3,5 m/seconde (outil précieux pour l'Ingénieur et susceptible de lui fournir une bonne évaluation des risques de déchirure dans les travaux où ce type d'endommagement est à craindre).

Le test CBR étant toujours très utilisé dans les laboratoires de Mécanique des Sols, des essais ont été faits pour essayer de corréler la résistance à la déchirure et au poinçonnement des géotextiles mesurée selon cette méthode et celle mesurée par dynamométrie sur bande large. Les résultats sont assez identiques, mais le test CBR, outre le fait qu'il ne fournit aucune valeur numérique de déformation, donne des résultats très hétérogènes compte tenu de l'inhomogénéité des géotextiles.

- La seconde tendance concerne toutes les communications nombreuses, qui ont traité des tests suceptibles de révéler l'aptitude des géotextiles à supporter des contraintes à long terme.
Il faut rapporter les études sur la tenue au fluage et à la fatigue, conduites aussi bien par les équipes américaines qu'européennes, avec ou sans présence de sol. Ces essais ont même été réalisés sur des éprouvettes de grandes dimensions en essayant de simuler au mieux les conditions d'environnement rencontrées en situation réelle. Il a été parfaitement démontré que la tenue au fluage des géotextiles est fortement améliorée, surtout dans le cas des nontissés, en situation de confinement, lorsque s'établissent de fortes intéractions sol-fibres. Il a été proposé par certains, d'étudier l'effet de la pression de confinement des géotextiles sur le fluage, en choisissant 3 sols typiques. Par contre, en ce qui concerne la mesure des coefficients de frottement sol-géotextiles, qui rejoint les mêmes préoccupations, il a été proposé de développer et d'appliquer une méthode normalisée en mettant en oeuvre cette fois le sol à stabiliser.

Il faut également évoquer les études sur la tenue des géotextiles aux basses températures, lumière, abrasion (application : ballast de voie ferrée), aux agents chimiques et biologiques. Ces travaux ont d'ores et déjà permis d'accumuler des données fort intéressantes, mais il reste à trouver le bon niveau de sévérité pour de tels tests et à mieux interpréter les résultats qu'ils fournissent en développant des corrélations entre modifications structurales et modifications de propriétés, corrélations qui pourraient être utilisables sur le plan pratique.

Nous terminerons cette analyse en citant des travaux qui permettent de mesurer des propriétés particulières, telle que la flexibilité des géotextiles et dont la connaissance peut être très utile pour apprécier a priori la plus ou moins grande facilité de mise en oeuvre sur le chantier (aptitude du géotextile à se dérouler, à épouser les irrégularités du sol support...).

Il est certain que tout ce travail devrait contribuer à rendre la tâche de l'utilisateur de géotextiles plus facile, à développer la réputation des géotextiles et le marché qu'ils représentent et à susciter enfin de nouvelles utilisations encore plus originales et encore plus audacieuses.

Il n'est pas sûr néanmoins que toutes les initiatives nationales ne compliquent pas quelque peu la tâche des producteurs dans leurs entreprises technico-commerciales... Espérons, que la coordination internationale qui se met en place oeuvrera pour le développement harmonieux de la recherche et du commerce international des géotextiles.

GIROUD, Jean-Pierre
Woodward-Clyde Consultants, Chicago, IL, USA
Conference Chairman

Closure Address
Allocution de clôture

Ladies and gentlemen

The first steps towards the formation of an international society on geotextiles have been taken. These steps were taken as a result of the meeting held on Wednesday evening.

The meeting was a big success. It was attended by approximately one hundred and fifty people from many different countries. A large number of the attendees contributed valuable ideas. A straw vote taken at the meeting showed that an overwhelming majority of attendees were in favor of the creation of some sort of an international society, few abstained, none voted against.

The attendees voted in favor of motions which can be summarized as follows:

- An Interim Committee will be formed with a lifetime of two years.
- The Interim Committee will prepare a set of bylaws for the operation of the International Society and will take appropriate steps to assure the start of the International Society.
- The Interim Committee will entertain submissions from organizations desiring to sponsor and organize a Third International Conference on Geotextiles and it will select the site after a careful review of proposals.

During the Wednesday meeting, nineteen persons volunteered to work in the Interim Committee, two more volunteered shortly afterwards. The twenty one members of the Interim Committee come from eleven countries and four continents. During the meeting professor Charles Schaerer was appointed chairman of the Interim Committee.

The Interim Committee met twice yesterday and decided to organize into three groups: one coordinating group, and two task groups. The coordinating group includes:

- professor Schaerer, chairman;
- Mr Fluet, secretary-treasurer;
- Mr Rankilor, vice secretary-vice treasurer;
- Dr Massenaux, vice chairman and leader of task group 1; and
- Dr Giroud, vice chairman and leader of task group 2.

Task group no. 1 includes twelve members. Its task is to prepare the bylaws and to take appropriate steps to assure the start of the International Society. Task group no. 2 includes six members from four continents. Its task is to prepare guidelines for the selection of a location for the Third International Conference, to entertain and review submissions from potential sponsors of the Third International Conference, and to select one of them.

The Interim Committee needs to be in contact with people interested in geotextiles worldwide. There are eleven countries represented in the Interim Committee. In addition, corresponding members from other countries or regions of the world are most welcome. In some countries there are already national committees on geotextiles and they are the natural corresponding members. In other countries, existing organizations or persons can be corresponding members. Those desiring to act as correspondents please contact professor Schaerer by mail.

The Interim Committee will work essentially by correspondence. Members of the Interim Committee will take the opportunity of various conferences to meet and, therefore, expenses will be kept as low as possible. However, expenses will be incurred for items such as correspondence and stationary. Several people have already voluntarily contributed, and any of you who also wish to contribute may see Chairman Charles Schaerer and Treasurer Joe Fluet after this meeting.

In conclusion, I can say that historical steps have been taken this week.

The fact that members from eleven different countries volunteered to put their time and effort into the organization of the International Society is clear evidence of the cooperative spirit of the international community. I pray that the society we are trying to form will enjoy the same cooperative spirit throughout its life.

It is almost time now to say goodbye, but, before that, I would like to thank on behalf of the Executive Committee those who contributed to the success of this conference. Once again, I would like to thank all the sponsors, especially the IFAI, and I extend special thanks to the IFAI staff who was very helpful during this conference and ensured its success.

I would also like to congratulate the translators for their highly professional work. I was even told that some presentations were improved through the translation!

Congratulations also to the audio-visual technicians. Slide projections were absolutely perfect. The first conference ever with no slide upside down!

And of course the technical success of this conference is the result of the impressive work done by authors, speakers, session leaders and general reporters.

Finally I would like to thank again the members of the organizing committee for the considerable amount of work they accomplished in two years. During the past month, I overwhelmed them with work and work and work, and they found the time to prepare the very thoughtful presentation they made last night at the banquet. I am very grateful and I will never forget this.

I tried as much as I could to delay the moment when we would have to say goodbye, but, as all the speakers at the technical sessions, I have limited time to deliver my speech and I have to abide by the rule of the yellow light, especially since Dick Bell, the chairman of our Technical Program Committee, is now in charge of the timer.

So goodbye and see you all at the Third International Conference.

Erratum
of
Conference Papers

Volume I
Erratum

Session 2A: Drainage I

Author: A. Cancelli

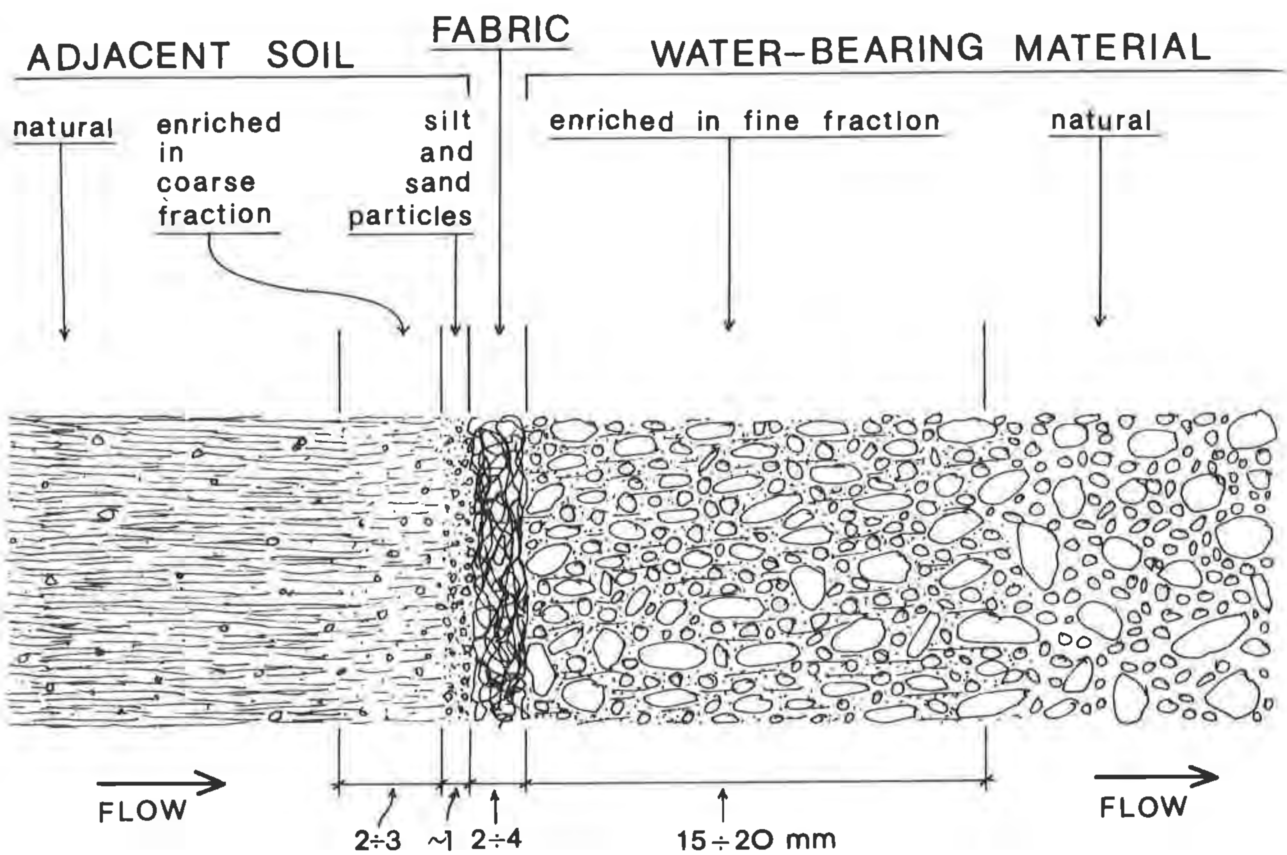
Title: Performance of Geotextiles in Stabilization of Clay Slopes in Italy

Comportement des géotextiles dans la stabilization de pentes argileuses en Italie

PP. 7-12

1) p. 12 COMMENTS AND CONCLUSIONS, Paragraph 1, last line should read "stop any further piping of fine particles, (see also fig. 8)"

Fig. 8 Representative section of the contact "soil/geotextile/sand" (all the measures are in mm).



Session 2A: Drainage I

Authors: T. G. Collins and D. D. Newkirk

Title: The Use of Geotextile Fabrics in Pond Construction Beneath an Impermeable Membrane (Geomembrane)

Utilisation de géotextiles sous une membrane imperméable dans la construction des bassins

pp. 13-18

1) p. 13 Column 2 of introduction

line 5 substitute "gases" for "gasses"

next paragraph, line 6 , substitute "gases" for "gasses"

2) p. 16 Column 1, lines 10 and 11

substitute "CPER 0.87 mm (35 mil)."

for "CPDE 0.76 mm (30 mil)."

Column 1, line 17

substitute "Geotextile B"

for "Geotextile C"

Column 1, paragraph 3, second to last line

substitute "Geotextile C"

for "Geotextile A"

Column 2, paragraph 2, line 16 should read:

"area 31.7 cm^2 (4.91 in^2) the indicated air flow"

Session 3A: Drainage II

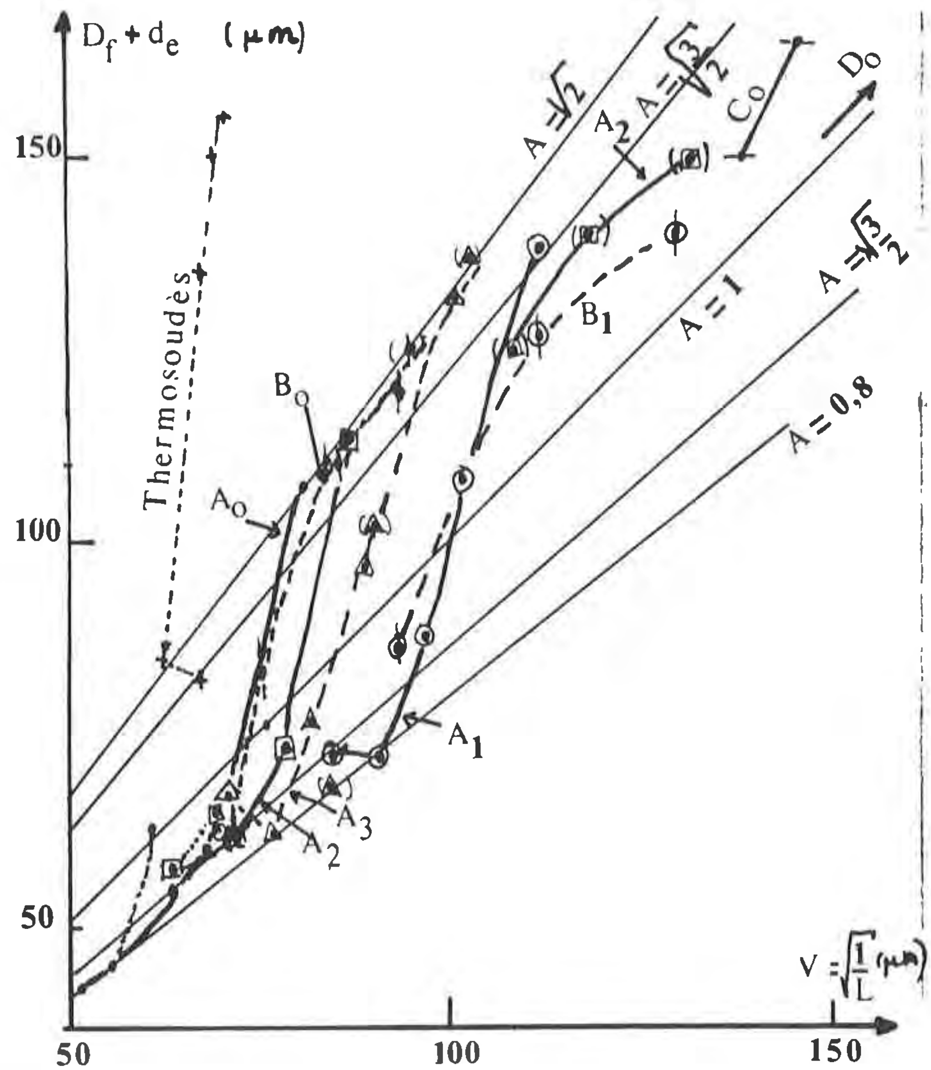
Authors: D. Fayoux and E. Evon

Title: Influence of the Fiber Size on the Filtration Characteristics of Needled-Punched Geotextiles

Influence de la fibrometrie sur les caracteristiques de filtration des geotextiles aiguilletes

pp. 49-54

1) p. 53 Corrected fig. 9



Author: G. Raumann

Title: Inplane Permeability of Compressed Geotextiles

La perméabilité dans le plan des géotextiles comprimés

Pages 56-60

1) p. 56

Two equations need correcting:

$$f(n)_H = [4 \ln(1-n)^{-1} - n(8-4n + n^3)/(2-2n + n^2)] (1-n)$$

$$f(n)_k = 2 [2 \ln(1-n)^{-1} - n(2 + n)] / (1-n)$$

Session No. 4A: Drainage III

Author: J. P. Giroud

Title: Filter Criteria for Geotextiles

Criteres de filtre pour les géotextiles

Pages 103-107

1) p. 104, Section 2, in the paragraph entitled "Comments related to Eq. 2", the first formula should read $\sqrt{3}/(2-\sqrt{3})$.

2) p. 106, in Eq. 8, H_f should be replaced by T_f.

Authors: W. F. J. De Jager and A. C. Maagdenberg

Title: Test Areas with Vertical Drainage Systems

Champs d'essai aux systèmes de drainage vertical

Pages 133-138

1) p. 138 6 Conclusions should read

Based on the results of in situ measurements combined with soil mechanical data and calculations, it may be concluded that the established differences in behaviour in practice of the different test-sites may be explained by the better or worse functioning of the different drain-types. Further analyses of the different measurements will lead to conclusions and have to indicate which properties of the drainmaterials or drains have resulted to the observed differences in behaviour in practice.

Session 5A: Drainage IV

Authors: J. P. Gourc and H. Hussain and M. Sotton

Title: Standard Test of Permittivity and Application of Darcy's Formula

Essai standard de permittivité et respect de la "loi" de Darcy

This paper begins on page 149 instead of page 139.

Authors: J. P. Gourc, Y. Faure, A. Rollin and J. Lafleur

Title: Structural Permeability Law of Geotextiles

Loi structurale de perméabilité pour les géotextiles

This paper begins on page 139 instead of page 149.

Authors: A. Rollin, J. Masounave, and J. Lafleur

Title: Pressure Drop Through Non-Woven Geotextiles:
A New Analytical Model
La perte de charge au travers les géotextiles nont
un nouveau modèle

1) p.166
Equation number 24 should be written

$$K = \frac{d_f [d_f + d] n \rho g}{A \mu}$$

Session 6A: Dams

Authors: F. Bogossian, R. T. Smith, J. C. Vertematti and O. Yazbek

Title: Continuous Retaining Dikes

Digues continues de retention au moyen de geotextiles

Pages 211-216

1) p. 215

On table one in all figures please replace the commas by points.

Example: 4,20m must be replaced for 4.20m

2) p. 216

On table two, please add a remark: the numbers 16, 32 and 33 on the fifth column are α_f values for samples 5 cm wide and 20 cm long, according to AFNOR G.07001, expressed in KN/m.

Session 6A: Dams

Author: R. J. Scheurenberg

Title: Experiences in the Use of Geofabrics in Underdrainage of Residue Deposits

Experimentation dans l'usage de géotextiles pour le drainage sous les residus

Pages 199-204

1) p. 199, 3rd last pasragraph, 1st sentence should read

"The East Rand Gold & Uranium (ERGO) project has been established . . . . to extract gold, uranium and pyrites (for the production of sulphuric acid)".

2) p. 201, 1st paragraph, 2nd last sentence should read:

"However, when certain items of the plant are out of commission, the pH cannot easily be controlled and it is noticeable how quickly the ferric percipitate . . . . . where oxidation can take place"

— Caption to Photo 3: "Laboratory simulation of filter drain performance".

Session 8A: Erosion Control II

Authors: R. Veldhuijzen van Zanten and R. A. H. Thabet

Title: Investigation on Long-Term Behaviour of Geotextiles in Bank Protection Works

Recherches sur le comportement à long terme des géotextiles utilisés dans la protection des berges

Pages 259-264

1) Page 263, right column, line 6:

$F_4 = \Delta h_l / \Delta h_b \times 100\%$

2) Page 263, Table 1:

F_3 (%) instead of F_3

F_4 (%) instead of F_4

Volume II
Erratum

Session 4B: Unpaved Roads II

Author: G. Raumann

Title: Geotextiles in Unpaved Roads: Design Considerations

L'usage des géotextiles dans les pistes de chantier

pp. 417-422

1) p. 421 First paragraph, last line:

For "breaking" substitute "braking."

Volume III
Erratum

Session 6C: Properties and Tests II

Authors: J. W. S. Hearle, A. Newton, J. Amirbayat, F. Alsawaf and M. A. Elgazar

Title: Resistance to Area Change as a Measure of Fabric Performance

La résistance des tissus au changement de superficiel comme mesure de performance

pp. 781-786

1) p. 783, Column 1, Line 32,

4(c) should be 3(c)

Line 35: 4(c) should be 3(c)

2) p. 784, Column 1, Line 36:

"Circumferential" should be "radial"

Session 7C: Properties and Tests III

Authors: R. D. Holtz, W. R. Tobin and W. W. Burke

Title: Creep Characteristics and Stress-Strain Behavior of a Geotextile-Reinforced Sand

Le fluage et le comportement contrainte-déformation de sable renforcé par des géotextiles

pp. 805-809

1) p. 807: The paragraphs beginning . . .

"Although the geotextiles . . ."

and

"The effect of soil density . . ."

should be transposed in their entirety to be read before the subheading "Long Term Tests."

Session 7C: Properties and Tests III

Authors: D. VanDine, S. Williams and G. Raymond

Title: An Evaluation of Abrasion Tests for Geotextiles

Une evaluation de tests d'abrasion de géotextiles

pp. 811-816

1) p. 814

Table 1b: Test Results

Rotary Abrasion Test (1000 cycles)

2) p. 815

Geotextile D

rounded Pe^2
flat Pe^1

AUTHOR INDEX
INDEX DES AUTEURS