INTERNATIONAL CONFERENCE ON GEOMEMBRANES

Proceedings
Volume I

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USA

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INTERNATIONAL CONFERENCE ON GEOMEMBRANES

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GIROUD, JEAN PIERRE
GeoServices, Inc. Consulting Engineers, Boca Raton, Florida, USA
Conference Co-Chairman

Opening Address

Ladies and Gentlemen:

On behalf of my co-chairman, Ron Frobel, and the Organizing Committee, I declare the International Conference on Geomembranes open, and I am pleased to extend a warm welcome to all of the participants. I am also very pleased to see that the audience is truly international.

Sponsorship

This conference is sponsored by IFAI, the Industrial Fabrics Association International. The fact that a technical conference of this importance is sponsored by a trade association demonstrates the vitality of the industry and we must congratulate the IFAI. Their representative on the Organizing Committee is Stuart Waugh, who served as Secretary General. Stu has been in charge of the many practical aspects of this conference, and the outstanding quality of his efforts will be quite evident to us all as the conference proceeds.

Organizing Committee

In addition, a number of cooperating organizations have contributed to the promotion of the conference, and their representatives comprised the Organizing Committee. I would like to extend my thanks to these cooperating organizations which are: ASTM, the American Society for Testing and Materials represented by Ron Frobel, co-chairman of the Organizing Committee and chairman of the ASTM task force on geomembranes and by Ivan Johnson, chairman of the ASTM sub-committee on impermeable barriers; USBR, the United States Bureau of Reclamation, represented by Ron Frobel and Lloyd Timblin, chairman of the Technical Program Committee, who certainly had the most difficult task in the Organizing Committee and was assisted by Professor Robert Koerner and other members of the Organizing Committee; USCOLD, the United States Committee on Large Dams, also represented by Lloyd Timblin; USEPA, the United States Environmental Protection Agency, represented by Marshall Dick; NSWMA, the National Solid Wastes Management Association, along with the Institute of Chemical Waste Management and the Sanitary Landfill Council; ASCE, the American Society of Civil Engineers, represented by Professor Ara Arman; ASAE, the American Society of Agricultural Engineers, represented by Dr. Allen Dedrick; AWWA, the American Water Works Association, represented by David Kittredge; IGS, the International Geotextile Society, represented by Joe Fluet and myself; and RILEM, the International Union of Testing and Research Laboratories for Material and Structures, represented by our European corresponding member, Professor Gamski, Chairman of the RILEM Commission on Synthetic Membranes.

The Geomembrane Community

The fact that we have a member from Europe in the Organizing Committee illustrates the international character of the geomembrane community. We hope that many other international conferences on geomembranes will be organized and that this conference will be remembered as the First International Conference on Geomembranes. To achieve this goal, it is important to unify the international geomembrane community which includes people involved in manufacturing, fabrication, design, installation, quality control, operation, monitoring and maintenance of structures built with geomembranes, as well as owners and representatives of universities, research organizations, governmental agencies, regulatory bodies, committees, trade associations and professional societies.

This conference brings together a maximum number of these people, and therefore offers a unique opportunity to coordinate their activities. Accordingly, a meeting has been scheduled to discuss the creation of an International Geomembrane Society. This society could promote the exchange of information, organize conferences (including a second international conference on geomembranes) and perform many other tasks. This is a very important matter for the future of geomembranes and I urge all interested people to attend the meeting.

A similar meeting was held two years ago at the Second International Conference on Geotextiles, and it led to the formation of the International Geotextile Society. Geotextiles and geomembranes have a lot in common, and maybe one day we will have an International Geotextile and Geomembrane Society and an International Conference on Geotextiles and Geomembranes. A major step toward the unification of geotextiles and geomembranes is the publication of the first issue of the Geotextiles and Geomembranes International Journal. Also, the Geotechnical Fabrics Report, published by the IFAI , is devoted to geotextiles and geomembranes.

Background of Geomembrane Development

One of the goals of the conference is to review the background of geomembrane development. The four special lectures to be presented during the opening session will set the stage for the conference. The first two lectures will provide an historical background from the point of view of the manufacturer by Charlie Staff, and from the point of view of the designer by Yves Lacroix. The next lecture, presented by Gordon Hawkins, will give us a taste of the international background of geomembrane applications. The last special lecture, presented by Robert Landreth, will illustrate the regulatory background that governs the use of geomembranes in waste disposal facilities in the United States.

Utilization of Geomembranes

Another important goal of the conference will be to review the state of the art of geomembrane utilization.

The first question that comes to mind is: do we use geomembranes? The answer is yes because geomembranes are used in a wide variety of applications demonstrated by the variety of technical sessions of the conference. The answer is yes because the market for geomembranes is increasing, a subject which will be discussed in a paper presented by Stuart Waugh, the Secretary General of our Organizing Committee. But the answer to the question "do we use geomembranes" is not as positive as it should be, for at least two reasons. One reason is that many designers and users lack knowledge on geomembranes. For instance, most of the case histories presented at this conference are related to geomembranes used as liners while geomembranes may be used in many other types of applications in geotechnical engineering. Another reason for not using geomembranes more often is that, for some designers, geomembranes lack credibility. This conference will certainly help increase knowledge of geomembranes, but a conference is simply not sufficient to establish credibility. It is critical for credibility that geomembranes be properly used.

This leads us to a second question: are geomembranes properly used? It is likely that this question will not be sufficiently addressed at this conference because many people do not like to discuss problems. Speakers who will present papers discussing problems and even failures should be commended, but there are too few of these courageous papers. Therefore it is appropriate that we devote a few minutes during this opening session to discuss why geomembranes are not always as successful as they could be. The main reason, I believe, is that many people, when they use geomembranes, tend to neglect two important aspects, design and quality assurance, because they believe that geomembranes are absolutely impermeable in all circumstances. The fact the there is no such thing as an absolutely impermeable geomembrane cannot be emphasized enough. Misconceptions about impermeability will be discussed in several papers presented at this conference. Of course, impermeability is a legitimate goal, especially for conservation of resources and protection of the environment which are two of the most important challenges of our times, but this goal cannot be achieved by just putting a geomembrane on the ground - proper design and quality assurance are required. I believe that no one is more qualified than Ron Frobel, the co-chairman of the Organizing Committee, to discuss in his opening address this topic which is so vital for the profession.

It is time for the profession to realize that a major effort should be made to ensure by all means that geomembranes perform satisfactorily. At present, the profession's attention seems essentially directed toward competition, which has the good effect of lowering prices, but too often only results in a low quality of installed geomembranes. Competition should not be the only concern of the profession. Cooperation between all members of the profession is urgently needed to select minimum standards that will ensure quality of installed geomembranes. The accomplishment of this task is necessary to pave the way for expanded uses of geomembranes and to establish the credibility of the profession. This task is a challenge placed before the profession. No time can be wasted. Together, let us all rise to meet this challenge. I do hope that when we meet again, in a few years, for the Second International Conference on Geomembranes, we will be proud to see the progress inspired by this first International Conference on Geomembranes.

FROBEL, RONALD K.
U.S. Bureau of Reclamation, Denver, Colorado, USA
Conference Co-Chairman

Opening Address

I am pleased to see that so many people from so many different countries have sacrificed the time and expense to participate in a conference devoted to synthetic barrier materials used in geotechnical design and construction.

The use of geomembranes has increased significantly in just the past 8 years. According to a recent survey, in 1976 approximately 70 million square feet of synthetic linings and covers were installed in the U.S. and Canada alone. In 1980, 122 million square feet were installed and it is projected that 185 million square feet will be installed by the end of this year. If this trend continues, by 1990 over 400 million square feet per year will be included in geotechnical construction for the control of liquid migration. These estimates are for the U.S. and Canada only. However, it is apparent that geomembrane use is becoming big business.

Since 1976, a total of approximately 25,000 acres of synthetic barriers have been manufactured and installed. Of this total, 60 to 70 percent of the impoundments involved containment of liquid chemicals, wastewater and solid waste. The remainder (30 to 40 percent) is devoted to clean water containment. But what percentage of this total represents successful installations where the appropriate geomembrane was chosen and where correct design and construction proceedures were utilized? I venture to say only a small percentage. Why?

The many types of geomembranes in use today are as diverse in their manufacture as in their applications.

They vary in base polymer type, color, texture, thickness, type of scrim reinforcement (or no reinforcement at all), method of production, method of seaming, method of application or installation and above all, cost. Their applications can range from a simple landscaping pool in a private backyard to an enormous potable water or waste containment reservoir. But, the end product, the installation, is dependent on many variables all of which are directly associated with product quality.

What is an acceptible geomembrane installation and how is quality germane to its achievement? The final installation will be greatly affected by the attention given to quality and the sincerity with which a program for its assurance is pursued. But what is quality and how can it be assurred in a geomembrane installation? Webster defines quality as excellence; the degree of excellence; relative goodness or grade. The verb assure means to insure (as against loss); to make certain; to guarantee; to establish firmly. Quality Assurance!

Quality Assurance for a project must start at the very beginning with the design engineer. It is apparent that in too many projects, proper design is used **only** as a last resort when all else fails. Proper design must marry the site parameters and geomembrane selection to the end use of the project. This marriage must last many years and should not be subject to failure because of incompatibility. Proper design procedures must include a thorough investigation of all parameters associated with the project before a final geomembrane system is selected.

Once the goemembrane system is known, a thorough quality control program at the point of manufacture is a must. From base resin selection, polymer formulation and mixing through sheet manufacture, all production control points must be monitored in an effort to produce the best quality sheet material or spray applied system possible. After production, the material must comply with minimum design specifications and the manufacturer must at least acknowledge this fact through testing and certification. A thorough sampling and testing program must be evident at point of manufacture.

When fabrication of a product into large panels is necessary, the point of fabrication must also comply with minimum specifications as to type and strength of bonded seam. Quality Assurance at the fabrication plant is also a must. Every foot of seam must be checked for integrity in a manner acceptible to all parties involved.

At the site, a thorough quality assurance program must include inspection during all phases of installation from subgrade preparation and liner placement through field seaming to final cover and future monitoring of the geomembrane system if required. The quality assurance program at the site could very well be the most critical because here the installer must contend with the elements in piecing together the final product. The best geomembrane system in the world (if there is one) can be delivered to the site, but without thorough control of its final placement, the installation could very well be a failure.

A failure of a geomembrane installation for whatever reason is not just a failure but another loss in credibility for the entire geomembrane community and for those of us who desire the best in quality for a product used in protecting our environment and conserving our precious resources.

We hope that this conference will achieve all of the goals we have set, but in particular, we hope that all of you will have a better understanding of the proper utilization of geomembranes.

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The Foundation and Growth of the Geomembrane Industry in the United States

The use of Geomembranes for Water storage and conveyance started in the 1950's with experimental work at Utah State University and the U.S. Bureau of Reclamation in Denver. The market was slow to develop but is now rapidly expanding with many different materials being used.

The foundation of the geomembrane industry in the United States is based upon the need for water conservation, particularly in the Western states. Farmers and ranchers have had the losses of water in conveyance and storage with resultant crop and animal losses since the opening of the west. The need for irrigation was recognized by the early settlers and the supply of water for irrigation was always a critical problem. Efforts were made to prevent the loss of the available water by proper earth selection so that the water could be diverted to the crops and storage facilities so as to be made available for when needed. Much of the water came from streams fed from snow melt and so was available only at various times of the year depending upon the snow depth and the weather conditions. However, as agriculture developed and industry increased it became necessary to store the snow melt and transmit the water to locations for use more economically.

Early canal and reservoirs were frequently lined with low permeability soil to reduce seepage. However, seepage losses in many canals amounted to 20 to 50 percent of the inflow. In many cases this seepage recharged the water table and in later years this was a source of water which could be pumped from the ground onto areas which were not served by gravity irrigation but additional cost were involved.

The Bureau of Reclamation was started in 1902 to provide water for the Western states for agricultural uses. The Bureau has been instrumental in building many miles of canals and also dams for major supplies of water. The greater length of canals, however smaller in sizes have been provided by various local irrigation companies frequently assisted, particularly in the design, by the Bureau of Reclamation.

The Bureau of Reclamation canals were originally of earth and most still are. However, many are also concrete lined, especially where soils are poor and land expensive. Higher velocities and smaller cross-sections can be used with concrete linings thereby reducing the land required for canal construction. The Bureau investigated various means of reducing seepage from canals, including sprayed asphalt, sprayed concrete, as well as soil treatments. These techniques are still used. For a number of years, about this time the Bureau of Reclamation had a "Lower Cost Canal Lining Committee" to investigate less expensive lining methods. I was privileged to be invited to the committee as a representative of the plastic industry. Later the name of the Committee was changed to "Open and Closed Conduit Systems Program" and the work continued.

No one knows who first used synthetic plastic materials for water storage and conveyance. It is possible that rubber sheeting was used early but we have no record. Polyvinyl chloride sheeting became available in the early 1930's and sheeting was used for swimming pool liners in the late 1930's or early 1940's. Today this is still a large application for PVC sheeting. My first containment use was for a three foot by twelve foot by one foot deep trough liner for my children's use for swimming which I made from vinyl-coated fabric in 1937 while I was still a research chemist. Since sheeting was being used for swimming pools no doubt there were other applications for water storage which were not recorded.

In 1952 Union Carbide became interested in the potential uses of polyvinyl chloride and polyethylene films in agriculture as a possible means of expanding the plastic markets. Many potential applications were considered. The potential use of liners in preventing seepage in irrigation water conveyance and storage was based upon contact established with the Soil Conservation Service who gave information regarding the large losses from the thousands of farm ponds as well as canals and ditches. The Soil Conservation Service suggested that the investigative arm of the U.S. Department of Agriculture, the Agriculture Research Service might be interested in undertaking an evaluation of plastic linings for seepage reduction. Contact was established with a Dr. C. W. Lauritzen (1) at Utah State University who was the Agricultural Research Service specialist on canal linings and a small grant to cover some of the costs involved was made for a study of the use of plastic linings. Dr. Lauritzen had worked on the use of various soils, soil cement, asphalts and concrete for linings for a number of years and had accumulated a lot of information on the requirements and materials. He also had a small grant from the Enjay Chemical Company, now Exxon, to investigate the use of Butyl rubber tubing in irrigation applications. Later he also had a grant from the Indian Jute Mills to investigate the feasibility of using jute in connection with asphalt for canal linings. Dr. Lauritzen had several assistants working with him on various projects including collection and storage of rain water for ranch animal use, items now called "rain traps".

Since we at Union Carbide were concerned regarding the economics of the lining of ponds, small test linings about 40 foot square were supplied for evaluation using both four and eight mil polyvinyl chloride and polyethylene. These were installed using a technique similar to that for other lining materials in which the impervious layer would be covered with earth to protect them from the heat of the sun and mechanical damage. Dr. Lauritzen also ran tests on the coefficient of friction between linings and soils for holding the cover on the lining, for root penetration covering techniques, microbiological resistance, etc. For durability opaque films were used from both materials and the compounding was made of materials which are inherently resistant to microbiological attack. In the test work it was found that roots would not penetrate the linings, that side slopes of three to one were desirable for earth coverings and that linings should be protected from mechnical damage and from animals. It was found that polyvinyl chloride linings were much tougher and puncture resistant than the polyethylene based liners upon the amount of damage occurred when gravels were shoveled onto the linings from the perimeter and the films examined. Subsequent tests by the Bureau of Reclamation also bore out this information.

Some test work on canal linings was also initiated in Hawaii and it was found that considerable damage was caused on the thin gauge films from nut grass which is indigenous to Hawaii. We later confirmed that nut grass would puncture thin films in the southern states. However, in South Carolina it was observed that if there was pressure on the film, the nut grass was turned under the liner and would not puncture the lining. Puncture occurred only slightly above and below the water line of exposed liners.

The seepage tests on the small installations in Utah were very encouraging even though there was some puncturing from the covering operation. The results indicated a leakage of only a few thousandths of an inch per day, a considerable improvement over other materials.

The ARS headquarters in Washington who approved Dr. Lauritzen's work soon became interested in initiating tests in the eastern part of the United States and a few installations were made including the one in South Carolina which was of eight mil PVC and from which samples are still being removed after a 25 year period. These samples show that properly compounded PVC has good durability when properly installed.

Since the U.S. Bureau of Reclamation is the largest canal operator in the United States contact was made with Mr. Lou Ellsperman and Mike Hickey (2) of that organization and they initiated tests in the laboratory in Denver and also elsewhere. Some of the installations were reported in special reports by the Bureau of Reclamation and generally satisfactory results were obtained. Based upon the success of the initial installations the Bureau is making considerably larger installations at the current time. Some of the initial installations also included a section of Butyl rubber approximately 1,000 feet by 30 feet. Originally the Bureau of Reclamation used 8 mil films the same as Dr. Lauritzen had used in his test work. However after a few years the thickness was increased to 10 mils and still later to 20 mils for greater puncture resistance and durability at little increase in cost for the total canal as the lining was only a small portion of the whole cost.

Because there was a large potential for linings of farm ponds as well as canals and reservoirs, Union Carbide installed a fabricating unit in their plant in Bound Brook based upon solvent bonding of a number of strips of vinyl together simultaneously. However, the market did not develop fast enough and Union Carbide discontinued their fabrication operation in about 1962.

Dr. Lauritzen had had contacts in Canada also and with the Prairie Farm Rehabilitation Service installations were made of polyethylene in a couple of canals as well as reservoirs in Alberta and Saskatchewan. A number of farmers and canal companies also installed liners after reading of the preliminary reports of Dr. Lauritzen.

The interest of the Agriculture Research Service and the Bureau of Reclamation and various resin and sheeting producers, led to the establishment of a committee to set up specifications for materials and installations of linings under the sponsorship of the American Society of Agricultural Engineers. As usual the proceeding took a number of years and along the way it was decided that it would be better to have the materials specifications under the American Society for Testing Materials rather than the Agricultural Engineers and the Agricultural Engineers would only have the specifications for installation. As a result the Agricultural Engineers wrote a specification number AP340.1 on installation and the ASTM has several specifications covering polyvinyl chloride, polyethylene and Butyl sheeting for linings. These are numbered ASTM 3020, ASTM 3083-72, ASTM 3253 and ASTM 3254.

The applications for liners began to grow about 1960 as more publicity was given to the earlier successful installations of liners. The original Water Pollution Control Act was written in 1948 but implementation was slow. With liners a technique for preventing seepage of pollutants from industry and cities became available and began to become accepted. A number of years ago a meeting was held in Vancouver, British Columbia to discuss the problems of landfill pollution and possible cures. The installations which had been made of liners for pollution prevention were discussed. Later the U.S. Environmental Protection Agency asked for assistance on a means of establishing specification on liners for landfills and it was suggested that possibly the National Sanitation Foundation in Ann Arbor, Michigan might be an appropriate group. This organization was interested and a contract established for the writing of a standard for Liners for Landfills. This effort was supported by about 40 suppliers of resins, sheeting producers, fabricators, uses and government agencies. After many years a standard has recently been issued by the NSF giving specifications for many current materials. The EPA in turn is using this data for their specifications for use in landfills. The specifications of the standard, will, of course, be used for many other applications of liners in water management and pollution control.

I have talked principally about the foundation of the geomembrane industry and would like to say only a few words about the growth of the business which will be covered more by other papers. Like all ideas the growth of the business was quite slow initially but has been increasing very nicely in the past few years. Original interest was on confinement of pure water in canals and reservoirs to prevent seepage losses. We are currently involved in the prevention of seepage from not only water but more frequently of toxic or hazardous materials.

As mentioned Union Carbide discontinued the fabrication of linings and I left the company and with my brother established Agricultural Plastics Company to develop the potential markets of plastic films in various applications for agriculture in which I had been interested (3). Soon after I began calling I made a visit on Claud Fetzer of the Corps of Engineers in Los Angeles and he asked if he could make a suggestion regarding our company name. He indicated that our market was not going to be in agricultural but in civil engineering applications and I accepted his suggestion and subsequently we changed our company name to Staff Industries so we could produce any item we wished. We concentrated on the fabrication of large vinyl liners again initially for lining of canals and farm reservoirs. However, since our contact was now going to be with the engineering field we began getting inquiries for a different application for linings and with the engineers were able to develop a number of markets which have been in need of means of preventing seepage. Pollution prevention has become the biggest requirement as many industries were storing their polluted waters in large ponds or reservoirs which were leaking severely and people in the neighboring area were complaining about the odorous or toxic compounds which had been introduced into their drinking water. The major inquiries were from the chemical and paper industry with a smaller number from the petroleum industry who have need for storing their waste brines. Inquiries were received regarding potential use of linings for repairing existing concrete structures which had failed severely by cracking due to settlement of the ground beneath. Some of these installations became quite complicated to fit around the existing structures. Some inquiries we received were regarding covering of portable water supplies to prevent pollution of the water from airborne contaminants and bird droppings. Some work had previously been done by the Agricultural Research Service including Dr. Lauritzen on covers mainly to prevent evaporation but the economics for this purpose were not suitable. But the economics for pollution control are good.

The future for Geomembranes looks very promising. Not only are the older applications for membranes expanding but many more applications are being developed. Linings have insignificant permeability but are generally used in thin gauges to be economical and so precautions must always be taken on their installation and use. The producers of linings count on the engineers who specify liners to see that they are properly specified and installed. With properly selected and installed membranes, we can make the world a better place in which to live.

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The Geomembrane Liner at Terzaghi Dam

Portions of the clay blanket on the upstream face of Terzaghi Dam were subjected to tension and possible cracking. A polyvinyl geomembrane was installed over these portions of the clay blanket to apply the full reservoir pressure to the plastic clay and reduce the probability of cracking. Impermeability, stretchability, strength, and adhesion to soils were considered in selecting the geomembrane and the other materials in contact with it. Design studies included stability of the geomembrane with respect to sliding, and jointing of the geomembrane sheets in the shop and in the field. Installation of the geomembrane was done with considerable care.

The geomembrane was installed in 1960; some sections of it were inspected in 1962, 1969, and 1974. The geomembrane was found to have performed as intended, although it ruptured after having stretched at the locations of sinkholes.

FUNCTION OF THE GEOMEMBRANE LINER AT TERZAGHI DAM

Terzaghi Dam, a 52-m-high earth and rockfill dam located in British Columbia, Canada, and completed in 1960, is underlain by a very compressible tongue-shaped clay lens. The dam has been described by Terzaghi and Lacroix (1). The upstream slope of the dam is covered by a 1.5-m-thick clay blanket, itself covered by a protective rubble fill about 2 m thick. Because of the variable thickness of the highly compressible clay lens, considerable differential settlement (up to about 5 m) was expected to take place within horizontal distances of approximately 15 m; and this in spite of the small thickness of the earthfill, which does not exceed 30 m in the major portion of the upstream face of the dam. Therefore, a clay blanket with a plane surface, even of low consistency, would sag and fail by tension.

Throughout the main portion of the clay blanket, the designer prevented excessive stretching and subsequent cracking and hydraulic fracturing of the clay by constructing the surface of the clay blanket at an elevation above its final position which was equal to the expected ultimate settlement of the clay blanket. The surface of the clay blanket could have a convex shape in the direction parallel to the contour lines and remain in compression during settlement. However, in the area of transition from the sloping portion to the horizontal portion of the clay blanket, it was impossible to eliminate a concave shape of the clay blanket surface in the direction perpendicular to the contour lines. Thus, intense stretching could not be prevented in that transition area.

The consequences of stretching are illustrated in Figure 1A, which represents a vertical section through the clay blanket perpendicular to the contour lines of the dam. As long as the clay blanket is intact, water percolates through the clay at right angles to its top surface. However, if the tensile strain in the clay blanket were to exceed a certain small value, the top layer of clay would fail by tension. Cracks would form and would propagate toward the base of the clay blanket as a result of water pressure acting on both sides of the cracks (splitting action). To reduce the probability of such an occurrence, the designer decided to cover the portion of the clay blanket that is subject to tension with a stretchable geomembrane. Without a geomembrane, the seepage pressure exerted on the clay by the water percolating through the clay blanket into the drained and relatively pervious underlying fill increases from zero at the top of the clay blanket to about 0.3 MPa at its base. Due to the presence of the geomembrane, the entire thickness of the clay layer is acted upon by the full water pressure, and since the unconfined compressive strength of the blanket clay does not exceed 0.1 MPa, incipient cracks leading to splitting action cannot form (Figure 1B).

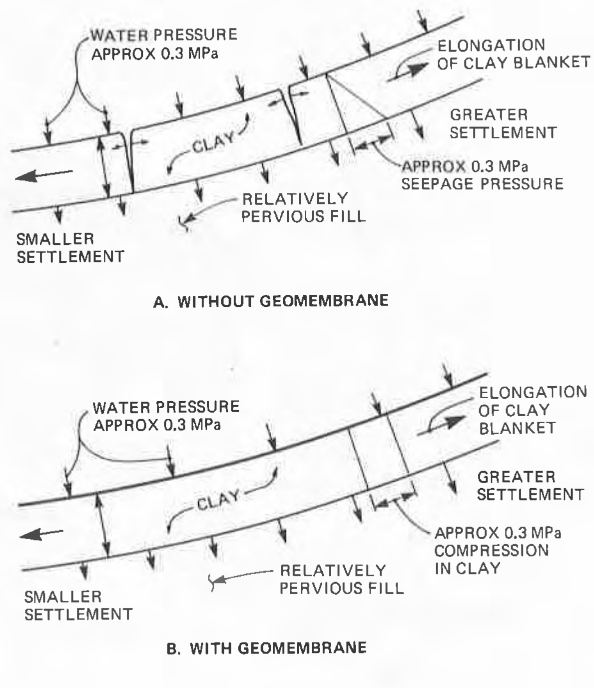


Figure 1. EFFECT OF SETTLEMENT ON CLAY BLANKET

The portions of the upstream face of Terzaghi Dam covered with a geomembrane have a total area of 10,000 m^2 and are shown in Figure 2.

FACTORS WHICH DETERMINED GEOMEMBRANE SELECTION

The factors taken into account in evaluating the relative merits of different kinds of geomembrane were impermeability, stretchability, durability, stability with respect to sliding, and cost.

Asphalt coatings were considered; they were rejected because of lack of knowledge about their durability under variable temperatures, and because of their flow properties.

Vinyl plastics that can be sprayed would have had the great economic advantage of requiring a minimum amount of preparation of the clay blanket prior to spraying. Tests conducted with two products (cold and hot spraying) showed that the plastic filled the depressions in the clay surface but that as a result of subsequent shrinkage, the geomembrane was lifted off the depressions.

Investigation of prefabricated plastic sheets showed that they were preferable because the manufacturer could guarantee the absence of pinholes. On the negative side, there was the necessity of smoothing the surface of the clay blanket prior to placing the geomembrane sheets. Polyvinyl chloride geomembranes appeared to be the most satisfactory.

The following properties were considered in the selection of the geomembrane.

Impermeability. Geomembranes were tested with a specially built permeameter under hydraulic gradients of about 10^6. For practical purposes, the geomembranes were found to be either permeable or quasi-impermeable, depending on the presence or absence of pinholes. The coefficient of permeability was found to be smaller than 10^{-20} cm/s.

Stretchability. To prevent formation of cracks in the blanket when deformation takes place, the geomembrane should be able to exhibit a large strain before breaking. A failure strain of 200% at room temperature was considered satisfactory. The geomembrane should not become brittle at temperatures as low as -30°C.

Strength. The strength of the geomembrane had to be great enough to avoid damage during installation. Field placing tests led to the selection of a thickness of 0.75 mm for the geomembrane.

Adhesion to Soils. The adhesion between geomembrane and soil depends on the degree of roughness of the geomembrane and the properties of the soil. During manufacturing, the surface of the geomembrane could be made smooth, or it could be embossed with a particular pattern. The geomembrane could be coated with adhesive and sprinkled with sand.

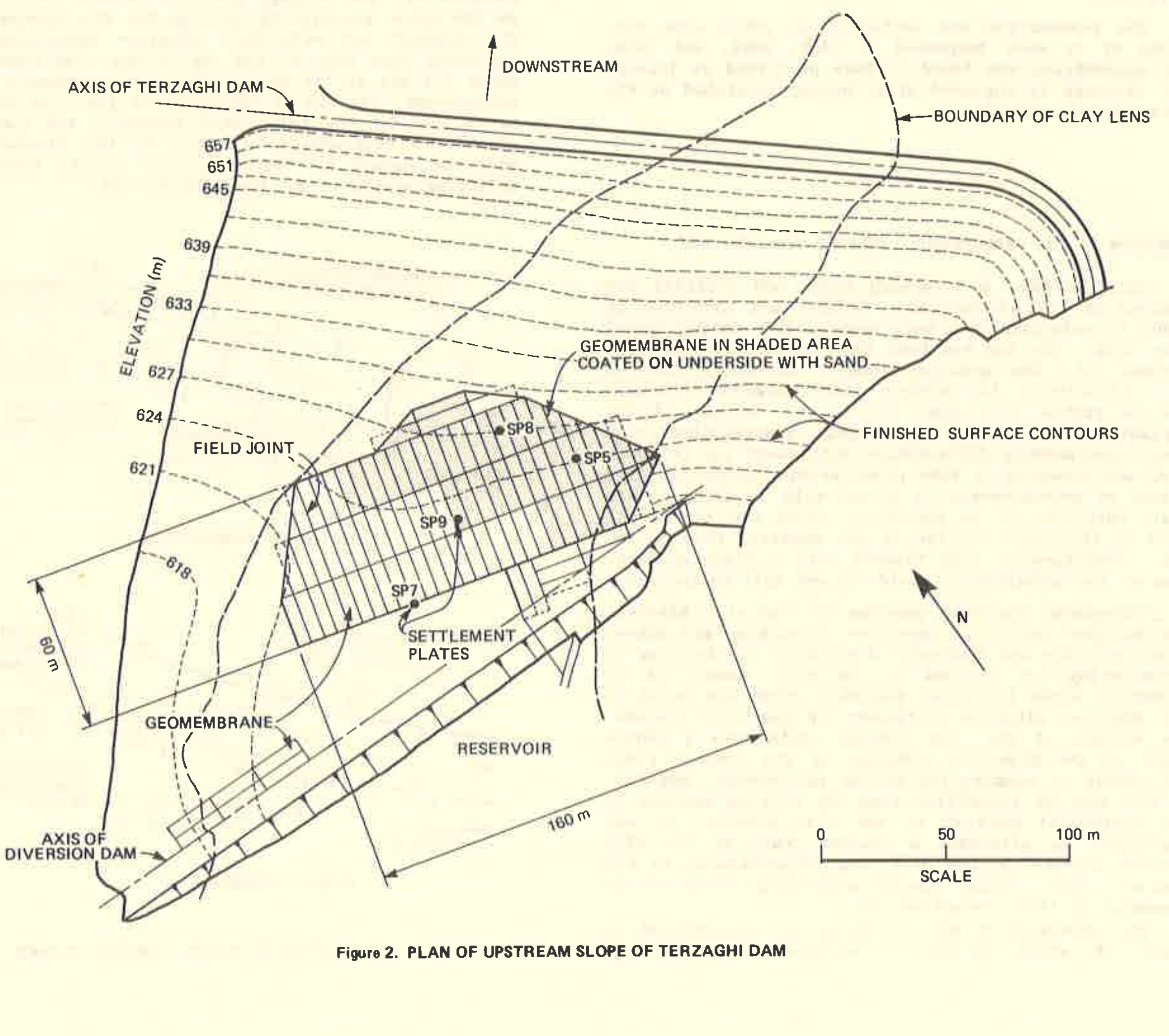


Figure 2. PLAN OF UPSTREAM SLOPE OF TERZAGHI DAM

Adhesion between geomembrane and soil was characterized by the shearing resistance against sliding, which was obtained from a laboratory and field testing program. The shearing resistance was found to vary linearly with the normal pressure on the geomembrane, with cohesive and frictional parameters depending on the geomembrane and the soil.

Under a normal pressure of 25 kPa, the following adhesions were measured in the laboratory:

Smooth geomembrane vs. plastic clay	10 kPa,
Embossed geomembrane vs. silt	15 kPa, and
Sanded geomembrane vs. plastic clay	25 kPa.

The test program showed that the adhesion to clay decreases when the water content of the clay increases, and that the adhesion to clay is weaker than the adhesion to silt, itself weaker than the adhesion to sand. The roughness of the surface of the geomembrane is significant. A smooth surface adheres less than an embossed surface or a surface covered with a coating sprinkled with sand. The type of sand is irrelevant as far as adhesion is concerned; however, fine silty sand adheres better to the geomembrane than coarse sand which, in addition, creates an unwanted drainage layer underneath the geomembrane. The adhesion decreases when the surfaces in contact are wet, and falls to a very low value if the geomembrane is covered with fungus.

SELECTED PLASTICS AND SOILS

Geomembrane. The geomembrane was a polyvinyl chloride manufactured by Canadian Resins and Chemicals, a division of Shawinigan Chemicals Limited of Montreal, under the brand identification No. KF-367. The geomembrane consisted of two plies heat-laminated (calendered) in the factory to eliminate pinholes. One face of the geomembrane was smooth, the other was embossed. The impressions on the embossed face had the shape of squares 0.75 mm by 0.75 mm that were about 0.08 mm deep. The thickness of the geomembrane was 0.75 mm ± 0.05 mm.

Adhesive. A vinyl adhesive with the consistency of a heavy syrup was used to join the sheets of geomembrane. It was manufactured by Minnesota Mining and Manufacturing Co., Ltd., under the brand identification No. EC-800 High Velocity Duct Sealer. The rate of consumption of the adhesive was about 1 liter for 4 m of joints.

Coating. A coating with the consistency of a paint was used to bond sand to the smooth face of some geomembrane sheets. It was manufactured by Amercoat Corporation of California under the brand identification Amercoat No. 33HB. The consumption of coating was about 1 liter for 10 m^2.

Ribbon Seal. A paraplastic ribbon seal was used in the field joints along with the adhesive. The ribbon was 25 mm wide and 3 mm thick, came in rolls, and was manufactured by Minnesota Mining and Manufacturing Co., Ltd., under the brand identification No. LC105 Ribbon Seal. The ribbon seal provided a deformable section in the joints.

Sealant. A polybutene sealant with the consistency of heavy grease was used in some locations (e.g., where settlement pipes penetrated through the geomembrane). It was manufactured by Tremco under the brand identification Mono-Lasto-Meric Buttering Compound Type A, Code No. 93-500.

Tensile Properties. Tensile tests were made at room temperature on geomembrane specimens that were either plain or that had various types of joints.

Type of Joint	Tensile Strength (MPa^2)	Failure Strain (%)	Duration of Test (min)	Days Cured	Remarks
None	10	250	30		Plain and sanded geomembranes gave similar results
None	23	170	5		Tested at -15°C
Shop joint	0.7	220	30	12	Failed at joint
Field joint	4	110	10	9	Failed at joint

Clay Blanket. The clay blanket consisted of glacial lake clay. It was a brown silty clay containing about 50% clay and 50% silt. The Atterberg limits were: liquid limit, 52; plastic limit, 19; shrinkage limit, 17; and sticky limit, 37.

As placed in the clay blanket, the clay had a water content of between 30% and 32%. Its unconfined compressive strength at 20% strain was about 80 kPa.

Silt. Light gray, fine sandy silt consisted of 2% clay, 70% silt, and 28% very fine sand (smaller than 0.2 mm). The silt was nonplastic, cohesionless, and moist in a natural state, although it absorbed water readily. Its natural water content was about 14%, whereas its water content at standard compaction was 18%. The silt was used over the geomembrane.

Sand. Granodiorite gray sand, a by-product of a screening plant, consisted of 25% particles from 5 to 12 mm, 40% from 2 to 0.6 mm, 33% from 0.6 to 0.06 mm, and 2% of silt finer than 0.06 mm. The sand was sprinkled on the coating of some geomembrane sheets.

Fine Sand. Brown fine sand consisted of 5% particles from 2 to 0.6 mm, 85% from 0.6 to 0.06 mm, and 10% of silt finer than 0.06 mm. The fine sand was used over the silt.

Rubble Fill. Well-graded rubble fill containing sand, gravel, and stones, for the most part smaller than 15 cm, was used for protection. The rubble fill was used over the fine sand.

STABILITY CALCULATIONS

The factor of safety with respect to sliding of the geomembrane on the clay blanket and that of the overlying soils on the geomembrane depends on the ratio of two forces: (1) the driving force, which is a function of the slope of the surface (the slope angle varies from 1° to 15°), and (2) the resisting force, which depends on the adhesion between geomembrane and soil or the shearing resistance of the soil itself, whichever is smaller. Both forces depend on the thickness of the protective fill.

For a given slope of the geomembrane, the driving force is at a maximum when the reservoir is empty. The adhesion tests between geomembranes and soils showed that the adhesion is at a minimum when the normal force is at a minimum (i.e., when the reservoir is empty). Consequently, the minimum value of the factor of safety occurs during construction or after drawdown.

When the smooth face of the geomembrane is placed on plastic clay, it was calculated that the factor of safety is greater than 2 if the slope of the geomembrane is flatter than 6°. If the slope is between 6° and 14.5°, the geomembrane has to be sanded to achieve a factor of safety greater than 2. The shaded surface in Figure 2 represents the portion of the geomembrane that was sanded on its underside.

When protective silt was compacted on the embossed face of the membrane, it was calculated that the factor of safety is greater than 2 for any portion of the geomembrane area. The factor of safety decreased to 1 or slightly less if the clay of the clay blanket became soft (water content 36%) or if the silt became loose and wet.

FABRICATION OF THE GEOMEMBRANE

The manufacturer provided the geomembrane in rolls 90 m long and 1.35 m wide. Four strips of geomembrane 1.35 m wide and about 20 m long were heat-welded by the vendor to form sheets about 5 m by 20 m. The heat necessary for welding was generated by a high-frequency electric current directed to electrodes between which the geomembrane was placed (dielectric method). Double electrodes 30 cm long and 3 mm wide were used. They made welded segments that had to be overlapped to produce a continuous joint. This procedure tended to create wrinkles in the sheet. The welds themselves were only partly satisfactory: weakness at the end of the segments, gap or offset between segments, and overheated welds occurred frequently. It was decided to strengthen all welded seams with adhesive joints.

The adhesive previously identified was manually placed under the lap of the smooth side (underside) of the seams with a pressure extruder. Adhesive was smeared over the lap and a capstrip of geomembrane 12 cm wide was placed on the adhesive (Figure 3A). The capstrip was omitted when the lap of the seam was wider than 2 cm.

To prepare the sanded sheets, the coating previously identified was sprayed on the smooth side of the sheet to form a coat 0.025 to 0.05 mm thick. Care was taken to keep a 20-cm strip around the edges of the sheets free from coating to prevent impairing field jointing. Within 1 min of application of the coating, dry sand was sprinkled on it. After 30 min of curing, the sheets were placed on racks for at least 24 h before they were installed on the dam.

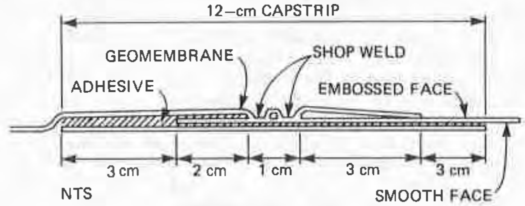
INSTALLATION OF THE GEOMEMBRANE

Clay Blanket Preparation. The finished surface of the clay blanket was rutted (bulldozer tracks) and nonuniform in consistency. A few days before installation of geomembrane in a predetermined area, the surface of the clay was kneaded and smoothed by a tractor with smooth tracks. The clay was then covered with a sheet of transparent polyethylene. Water condensing on the inner face of the polyethylene kept the top few millimeters of the clay soft. During the morning preceding installation of the geomembrane sheets, the clay surface was further flattened by pneumatic hand tampers. Flat discs 25 cm in diameter had been welded to the heads of the tampers. The area was again covered with polyethylene sheets until the time the geomembrane was placed. Just before unrolling a sheet, the consistency of the clay was adjusted and its surface was tamped. The small, local depressions were filled with powdered, dry clay. The finished surface was very smooth, although it was not plane.

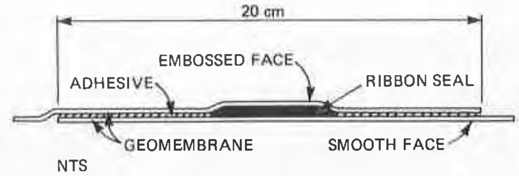
Placement of the Geomembrane. Immediately after final preparation of the clay surface, each sheet was unrolled with the embossed side up. The formation of wrinkles was avoided by pulling gently at the edges of the sheet, taking care to avoid lifting the geomembrane from contact with the bottom of any depression, for this would have trapped air beneath the geomembrane. Polyethylene sheets were temporarily spread over the geomembrane to provide some protection from heat prior to placing the overlying materials. All sheets were secured to the ground with sand-filled plastic bags.

Field Jointing. Field joints were made with the adhesive and the ribbon seal previously identified (Figure 3B). The sheets were overlapped a minimum of 20 cm. After cleaning the overlap, ribbon seal was unrolled approximately 10 cm from the edge of the embossed face of the sheet. The adhesive was smeared on both sides of the ribbon seal. The smooth face of the other sheet was then immediately pressed over. Geomembrane patches 1 m by 1 m were glued over the meeting area of four sheets.

Three pipes connected to foundation settlement plates were located within the geomembrane area. The detail of the junction between geomembrane and pipe is shown in Figure 4.

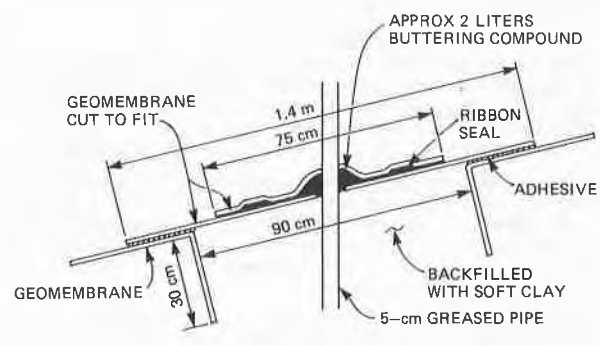


A. SHOP JOINT DETAIL



B. FIELD JOINT DETAIL

Figure 3. GEOMEMBRANE JOINTS



DETAIL Y

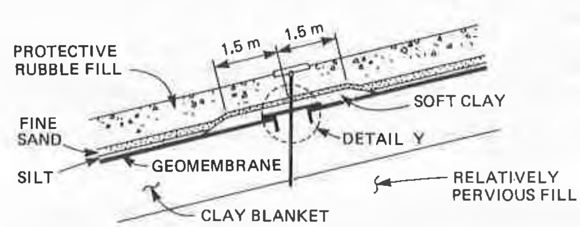


Figure 4. TYPICAL DETAIL AT SETTLEMENT PLATES

Covering of the Geomembrane. The geomembrane was covered with three types of material: silt, fine sand, and protective rubble fill. A clay liner was placed on and around the perimeter of the geomembrane to prevent the reservoir water from seeping between geomembrane and clay blanket (see Figure 5).

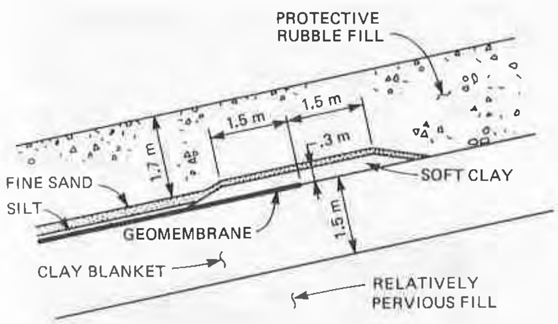


Figure 5. TYPICAL DETAIL AT OUTER EDGE OF GEOMEMBRANE

To construct the clay liner (Figure 5), lumps weighing a few kilograms were placed side by side to form a layer 10 to 15 cm thick. The lumps were of medium consistency: 50 to 100 kPa. Water was then added to obtain a consistency slightly softer than that of the plastic clay used for the blanket. The lumps were compacted with pneumatic tampers. Air pockets which were formed in the process were punctured with a stick. The bond between clay blanket and clay liner was improved by scarifying and watering the clay blanket just before placing the clay liner.

The silt was spread by hand on the geomembrane to form a loose layer 15 to 20 cm thick. It was watered with hoses while in place to increase its water content to optimum and compacted by laborers walking on its surface, and by four passes of a heavy, smooth garden-roller. The thickness of the compacted layer varied between 10 and 15 cm.

The fine sand, carried in wheelbarrows, was spread on the silt to form a layer about 25 cm thick. The sand, when spread, was watered with hoses and compacted with two or three passes of a vibrating plate measuring about 30 cm by 60 cm. Approximately 1.7 m of protective rubble fill was placed over the sand. It was spread by bulldozers in two or three layers.

The geomembrane was installed in 4 weeks; heavy rain impaired the work for 1 week. The various stages of installation of the geomembrane are illustrated in Figure 6.

Precautions and Difficulties. Precautions were taken to protect the geomembrane. Smoking was forbidden. Walking on the geomembrane was restricted to personnel wearing rubber shoes. No heavy equipment was used on the silt and sand; bulldozers were allowed on the protective rubble fill. In spite of these precautions, holes were found in the geomembrane. They resulted from various causes, such as dropping of tools. Geomembrane patches with field joints were placed over these holes.

Water condensed between the clay blanket and geomembrane before placing of the silt. The clay surface was softened, reducing adhesion of the geomembrane.

Air bubbles formed under the geomembrane. Air was trapped when the geomembrane was bridged over undulations of the clay surface. Most of the air was forced ahead of the advancing edge of the silt layer, which was in most cases built in the uphill direction. Small air bubbles were easily concentrated in large bubbles

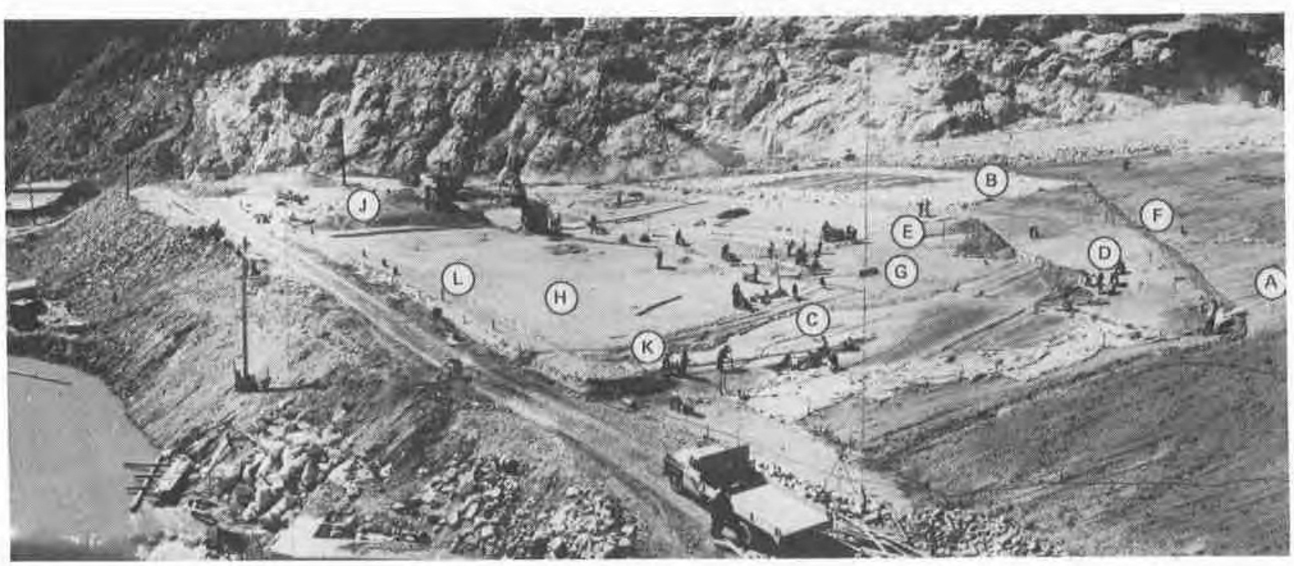


Figure 6. INSTALLATION OF THE GEOMEMBRANE IN 1960

A. CLAY BLANKET
B. POLYETHYLENE SHEETS PREVENTING THE CLAY FROM DRYING OUT
C. GEOMEMBRANE SHEETS BEING JOINTED
D. GEOMEMBRANE SHEETS BEING FOLDED BACK TO ALLOW CONDENSED WATER BETWEEN CLAY AND GEOMEMBRANE TO EVAPORATE
E. SETTLEMENT PIPE CROSSING THE GEOMEMBRANE
F. DITCH PREVENTING RUNOFF WATER FROM FLOWING UNDER THE GEOMEMBRANE
G. SILT
H. SAND
J. PROTECTIVE RUBBLE FILL
K. CLAY LINER BEING PLACED
L. LOCATION OF SINKHOLE AREA IN 1962, 1969, AND 1974

which were punctured. Field patches were placed over the punctures.

Besides the condensation water, rain and construction water percolated under the geomembrane during installation. Most water bubbles had the appearance of air bubbles. These bubbles were punctured and then left open to let the clay dry. Water was sometimes bailed out in varying quantities, up to 30 liters in one instance.

PERFORMANCE OF THE GEOMEMBRANE

In 1962, 1969, and 1974 (i.e., 2, 9, and 14 years after completion of Terzaghi Dam), sinkholes formed on the upstream slope, some of them in the area covered by the geomembrane. In some cases, sections of the geomembrane were found to be about 3 m below their installed elevations. The geomembrane followed the deformations of the clay blanket and strained as it had been intended to. The geomembrane ruptured when the strains were very large. The rupture shown in Figure 7 occurred after the geomembrane strained 160%.

Tests made in 1962, on samples recovered 2 years after installation, showed that the failure strain was still very large (150% to 200%). Observation of the geomembrane during the 1974 sinkhole repairs showed that it was stretchable and appeared to have retained most of its original properties.

Although the geomembrane is no longer essential since most of the deformations of the clay blanket have now occurred, the geomembrane is still functioning 24 years after installation.

ACKNOWLEDGEMENTS

Karl Terzaghi developed new design methods and innovated several construction techniques for the dam which bears his name. He was the author of the concepts and most of the significant details reported in this paper. The writer was head of the soil section of the engineering company owned by the owner of the dam. Terzaghi Dam is owned by British Columbia Hydro and Power Authority.

Figure 7. THE GEOMEMBRANE STRETCHED BEFORE RUPTURING IN ONE OF THE 1962 SINKHOLES

REFERENCE

(1) Terzaghi, Karl, and Lacroix, Yves, "Mission Dam - An Earth and Rockfill Dam on a Highly Compressible Foundation," Geotechnique, The Institution of Civil Engineers (London, March 1964).

HAWKINS, GORDON
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Development & Use of Membrane Linings for Canal Construction Worldwide

SUMMARY

Two decades of research and development following the introduction of plastic membranes have proven the merits of such lining for watertight canal construction. Construction techniques have been refined during this period to meet present day needs. Mechanization is now available for the construction of membrane-lined canals for higher production rates and for better quality installation. Membrane lining is now accepted universally as an approved method of seepage control.

INTRODUCTION

Geomembranes, in the form of thin plastic films, have been used for canal construction for the past 20 years. Many countries have experimented with plastic membranes to determine life expectancy under field conditions, but despite extensive laboratory testing, there has always been a fear that field conditions would limit the life of the thin plastic films. For this reason, much of the early construction work was done on small pilot projects. Once these systems were installed, they were monitored yearly to determine seepage losses, aging and deterioration of the plastic. Canals are normally lined to prevent seepage losses. With highly porous soils, such losses can amount to 50 percent or more of the total flow of the canal. Such losses are costly from an operational standpoint, and limit the water needed for successful farming operations. Canals constructed on poor soils (loess or gypsum), without watertight membranes have experienced extreme deterioration in a very short period of time. In such cases it has been found necessary to make the canals totally water tight, allowing no leakage of water through cracks, joints, etc. Plastic membranes provide a good solution where watertight construction is required. This review is based on personal observations made in the countries being evaluated.

Figure 1 shows a typical canal system under construction. This canal, part of the California Aqueduct, was more than 400 miles long and was constructed to transfer water from Central California to Southern California.

Figure 1 California Aqueduct Under Construction

HISTORICAL

Major canal construction can be found world wide and yearly more and more lands are being reclaimed for irrigation purposes. Those countries now doing the biggest job of land reclamation are:

Figure 2

Area in Irrigated Land - 1983
(Estimated)

	Acres	Hectares
China	120 million	50 million
India	108 million	45 million
USSR	84 million	35 million
United States	62 million	26 million
Middle East	50 million	21 million

Prior to World War II, low energy costs and an apparent abundance of low cost water resulted in low operational efficiencies on many projects and the loss of water due to seepage was not feared. With increasing costs in all areas and with dwindling water supplies, the need to prevent water losses became apparent. Earth lining with a clay type soil was used in earlier years. This was followed by lining of concrete. With the advent of the geomembrane industry, many countries began experimenting with thin plastic membranes as an economical means of sealing the canals against water losses. Testing of such materials started in the late 1950's and by the middle of the 1960's, there were a large number of short canals utilizing this new material. At this time, there was skepticism concerning the durability of thin plastic films for this application and most engineers were willing to wait to see how such materials survived the forces of nature and the environment before endorsing their use wholeheartedly.

CONVENTIONAL CANAL LININGS

Where on-farm canals do not present a serious problem for water losses, larger distribution canals or aqueducts normally require a protective lining to reduce or eliminate seepage losses. Today, conventional lining material can be grouped into four categories:

* 1. Earth lining
* 2. Concrete lining
3. Asphaltic type lining
* 4. Rock or clay brick lining

* These linings may be used alone or in conjunction with membrane linings.

HEAVY EXPOSED MEMBRANES NOT USED

The heavy exposed membranes used for mining and industrial applications, are not appropriate for canal construction. Traffic from people and animals is common in canal areas. Such traffic can do severe damage to exposed linings. Most membranes used for canal construction are thin (10 to 20 mil) and require an overlayment of a foot of soil or two to four inches of concrete. This type of design has been followed consistently by most nations. Sufficient projects have been operating for 10 to 20 years to give confidence for future application of this construction technique. The economics realized in the use of plastic membranes has proven to be good.

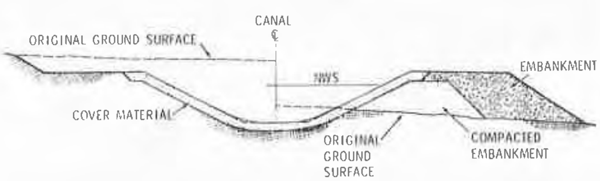

Figure 3 - Profile of a Typical Earth Lined Canal With Underlayment of Plastic.

MEMBRANE MATERIAL EVALUATED FOR CANAL CONSTRUCTION

LDPE - PVC - Hypalon - Polyolefin - Hypofors - High Density Polyethylene (HDPE)

Although polyethylene (PE) and Polyvinyl Chloride (PVC) have maintained a consistent lead in usage for canal construction, many membrane linings have been evaluated for this application. In the early 1950's, a new polymer was developed called Hypalon. It was superior in some ways to even butyl but generally more expensive. In the mid 1960's, a bitumen membrane reinforced with nylon called hypofors was introduced. In the mid 1970's a new product called polyolefin was introduced. Most recently, high density polyethylene (HDPE) has been introduced and blended for specific use applications. South Africa has introduced a new HDPE/alloy polyethylene impregnated with rubber that has excellent characteristics in cold weather application and is used as an exposed lining. This material is available in 20 mil to 80 mil thicknesses. West Germany has, likewise, introduced new high density polyethylene in thicknesses from 60 mil to 100 mil with comparable characteristics. Both materials have been used in pond construction rather than canal construction.

Membrane Lining Materials

Of the wide range of materials that have been proposed for lining canals, material evaluations, field testing and economics have reduced the potential list of candidate materials to the following:

1. Polyethylene (PE) Film: The lowest cost of the membrane materials. Most frequently, this material is referred to as LDPE (Low Density Polyethylene). Material thickness is 0.20mm to 0.30mm (8 to 12 mils). Polyethylene cannot be exposed for long periods to the sunlight and must be covered with concrete or soil. This is the most economical of all membrane lining material.

2. Polyvinyl Chloride (PVC): The next lowest cost of the membrane materials. This material has a price higher than PE film, but it also has some characteristics of strength and durability that are better. Material thickness is 0.25mm to 0.50mm (10 to 20 mils). This material must be covered to be protected from solar radiation with soil or concrete. The percent and type of plasticiser is important as to the grade of PVc that is obtained and as to its longevity characteristics.

3. Polyolefin: A newer development of the plastic membranes that is resistant to solar radiation. Although it can be used exposed, it is normally used the same as PE or PVC. Material thickness is 0.25mm to 0.50mm (10 to 20 mils).

UNITED STATES

The earliest U.S. history or experience regarding plastic membranes dates back to 1953 when the U.S. Bureau of Reclamation installed a polyethylene plastic membrane in a canal on an irrigation project in the state of Montana. Four years later, a polyvinyl chloride (PVC) lining was installed on another project in the state of Wyoming. This canal work had been preceded by work by the U.S. Department of Agriculture in Utah when butyl coated fiberglass panels were installed in canals as a test for lining purposes. Eventually 31 projects were undertaken utilizing plastic membranes. All work before 1981 involved the use of PVC with a 10 mil thickness. After 1981, the thickness of the material was increased to 20 mil. The results of the work done by the Bureau with plastic canal linings has now been summarized in a book entitled, "Performance of Plastic Canal Lining" - REC-ERC-84-1. W.R. Morrison and J.G. Starbuck have completed a 20 year review of the subject and published their findings this year. This book is available through the U.S. Bureau of Reclamation printing office in Denver.

Figure 4

RELATIVE COSTS OF MEMBRANE LININGS
Installed in the United States

Membrane Lining Installed With 1 Foot (30cm) of Overburden

Mat'l.	Membrane Thickness	Furnishing & Placing Membrane	*Cost of Total Install.
Low Density Polyethylene (LDPE)	10 mils	$1.00/yd2	$4.00/yd2
Poly-vinyl Chloride (PVC)	20 mils	$2.00/yd2	$5.00/yd2
Poly-olefin	20 mils	$2.50/yd2	$6.00/yd2
Butyl Rubber	60 mils	$4.50/yd2	$8.00/yd2
Hypalon	30 mils	$4.50/yd2	$8.00/yd2
High Density Polyethylene	80 mils	$6.00/yd2	$10.00/yd2

Canal Construction Machines

Modern-day construction machines have been developed to prepare sub-bases before the laydown of cover materials as well as place the soil, concrete, or plastic. The R.A. Hanson Company, the only U.S. company presently involved in this work, has done much in the past five years to further the advancement of mechanization for base preparation and placement of membrane lining. Figure 5 shows full section machinery and Figure 6 half section machinery used to trim and place covering materials.

Earth lined canals are designed with slopes of 2:1 or 3:1; however, concrete lined canals have slopes of 1.5:1. Early research in this field showed that any attempt to place concrete over plastic on slopes of 1.5:1 resulted in the concrete sloughing or flowing to the bottom of the canal. Originally, it was thought that the smoothness of the plastic was responsible for this action but continued monitoring of the work showed that concrete would bleed water after being placed and the water film combined with the plastic provided a surface too slick for the concrete to adhere to. It was then necessary to effectively remove the moisture to complete the concrete pouring job. Machines are now operating overseas effectively placing concrete over plastic on slopes as steep as 1.5:1.

Figure 5 - Full Section Canal Construction Machinery Courtesy - R.A. Hanson Co., Inc.

Figure 6 - Half Section Canal Construction Machinery Courtesy - R.A. Hanson Co., Inc.

Figure 7
Courtesy of U.S. Bureau of Reclamation

Figure 8
Courtesy of U.S. Bureau of Reclamation

CANADA

During the past three years the province of Alberta has begun an extensive rehabilitation and modernization program of several hundred miles of major distribution canals on the Lethbridge Northern Irrigation District (LNID) and St. Maries projects. The lining material selected for these projects is 20 mil PVC. In Alberta, the rehabilitation of canals must be carried on during the off season from October to March. During this period, the temperatures are subzero much of the time. In Alberta, plastic material must be tested with a cold crack test that is carried out at temperatures between -8 degrees F and -15 degrees F. In this test, the material is folded back on itself 180 degrees and then unfolded. This must be done with no cracking of the plastic. This is proving to be an exceptionally hard test for the manufacturers to comply with. In such cold climates, it is found that subgrade fill material can cause microscopic perforations in the plastic during construction. Figures 9, 10 and 11 show construction being carried on outside Lethbridge, Alberta, Canada on the LNID project.

Figure 9

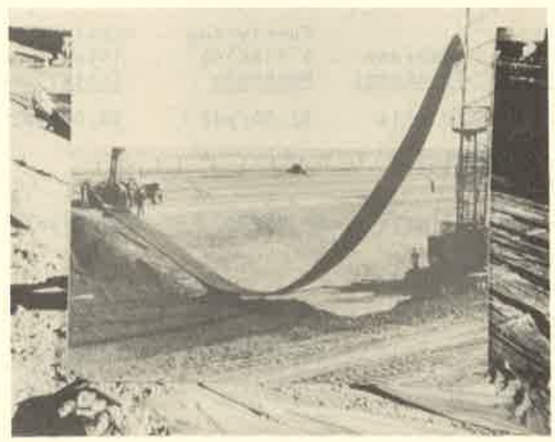

Figure 10

Figure 11

Although Canada has done considerable testing over the years, to prove the value of plastic membranes, this work has not been summarized for easy public review. Canada and the United States collaborate in this area and now both countries have accepted a standard of using 20 mil PVC as a base material for new canal construction and for rehabilitation.

ENGLAND

The Water Research Center of England has published an extensive report, "Technical Report TR-113" dated May 1979 and written by P.C. Kirby. The report entitled "The Selection of Membrane Materials for Water Distribution and Allied Applications" covers a very exhaustive review of all materials that are being used for seepage control in ponds and canals. Mr. Kirby summarizes his report which analyzes some 14 different rubber, plastic and bitumastic materials.

Summary

Membranes are being increasingly used in water undertakings in different parts of the world for three main fields of application:

1. The lining of earth built water retaining structures such as water storage reservoirs, canals and waste water lagoons, and the renovation of existing conventional tanks and reservoirs.

2. The protection of open reservoirs with floating covers.

3. The lining and covering of refuse tips.

Applications 1 and 2 offer the possibility of substantial cost savings over conventional methods while 2 and 3 safeguard water quality from contamination.

Each of these applications requires different key properties in the membrane material. The factors giving rise to stress and promoting deterioration of the membrane in each of the principal situations are analyzed in this report in order to facilitate selection of the most suitable material for a given scheme.

USSR

Today the Soviet Union has a great deal of field experience in the installation of thin films such as LDPE or PVC. The Soviets are using PE film in their irrigation canals in Central Asia, the Trans-Volta Region, the Ukraine, as well as other areas. In their small canals they have used 8 mil (0.2mm) thickness under concrete lining (monolithic and precast concrete) and in large canals they have used two layers of the same film covered with about one meter of earth in the bottom and precast concrete slabs at the waterline for erosion control.
The Soviet Union has experimented with plastics with their own independent programs during the same time period the western countries have. Most of the USSR work has been done with polyethylene instead of PVC. Mr. Timblin of the USBR is presenting a full-length paper at this congress on this subject. LDPE (low density polyethylene) has been used extensively by both Russia and India. In 1975, during a normalization of relations between the U.S. and USSR, their USSR Ministry of Reclamation and Water Management entered into a joint venture project with the U.S. Bureau of Reclamation to combine the state-of-the-art knowledge of both countries and to do experimental work on a program to prove the effectiveness of various membrane linings with respect to reducing seepage losses in canals.

Figure 12-US/USSR Joint Venture

Materials supplied by the United States were installed in test ponds constructed in the Ukraine and then seepage losses were monitored during succeeding years. Nine test sections were constructed and tested for seepage losses as follows:

Figure 13

Lining Filtration Flow-Preliminary Seepage
Seepage Losses - US/USSR Test Program

Pond No.	Lining System	Liters Per Day Per Square Meter 24 hrs.	72 hrs.	1 week	2 weeks	1 month
1	PVC film w/soil cover	2.5	11.7	0.9	2.8	1.4
2	PVC film w/concrete lining	0.0	9.3	0.0	0.4	0.7
3	PVC film w/shotcrete lining	18.4	5.6	6.5	3.9	2.4
4	PE film w/soil cover	10.1	12.4	4.5	5.8	4.6
5	PE film w/concrete lining	19.4	2.9	2.5	2.4	0.3
6	Polyolefin film w/ soil cover	4.5	14.7	2.8	4.0	1.0
7	Polyolefin film w/ concrete lining	2.2	9.4	0.0	0.4	0.0
8	Compacted synthetic material M-179	30.0	6.0	1.0	4.1	3.1
9	Urethane sealant	31.5	6.4	2.2	7.8	9.4
	Control	20.1	32.6	29.3	37.3	30.6

The USSR has done considerable work installing plastic membranes beneath precast concrete slabs, a process not used in the United States.
This brief interchange of information between the two countries demonstrated the benefits the US/USSR can derive in working together to achieve a common goal. It was a good experience from which all benefitted.

INDIA

In India, extensive research has been carried on with plastic membranes. The Irrigation and Power Research Institute at Amritsar has developed a suitable lining technique for existing earthen canals and channels. Their system called Combination Lining makes use of low cost polyethylene film laid on the bed of the canal whereas the sides are lined with precast brick-tile blocks. This type of lining has been used for the past 15 years in

Punjab and has proven durable, efficient, and quite suitable for lining existing distributaries. The Gujarat State Irrigation Department is making extensive use of low density polyethylene (LDPE) on the bed of their canals and is then covering the bed with a single layer of brick masonry. They are also experimenting with solid cover in place of bricks. In other areas of India, the Public Works Department of Tamil Nadu as well as the UP State Irrigation Department of Uttar Pradesh are carrying out similar experiments. The Central Water and Power Commission of India estimates that seepage losses from unlined canals can amount to as much as 45% to 60% of the carrying capacity of the canals. LDPE has been reported to give better results than PVC. As early as 1958, black polyethylene (100 microns thick) was installed at the Irrigation and Power Authority at Amritsar showing favorable test results. In this case, it was reported after 8 years of field installation that tensile strength and elongation value of films both natural and black remained practically unaffected by their use. Other experiments showed that after 12 years, seepage losses using LDPE of 100 micron thickness in combination with single tile lining, water seepage was nil. At places with animal traffic, the membrane had not been affected by the hoofs of animals where 12 inches of soil had been maintained over the plastic. LDPE film laid 19 years ago on the Doburju distributary near Amritsar has been found in good condition and not affected by submerged aquatic weeds where 12 inches of soil cover was used.
From 1959 to 1974, some 26 major canals in India had black polyethylene installed with damage reported only on exposed sections of canals where the cover material was removed or washed away.

MIDDLE EAST

The Middle Eastern countries of Iran, Iraq and Syria all have soil problems that have necessitated the construction of watertight canals where plastic membranes have been installed beneath concrete. This unique design has been required to combat some extremely high gypsum problems that developed on the flood plains of the Euphrates and Tigris rivers. These troublesome soils have a unique characteristic of subsiding when water is applied to them through leakage in the joints of regular concrete lined canals. Small amounts of water penetrating the concrete lining is sufficent to cause a piping action creating large cavities underground with a resulting collapse of the canals. Canals completed in the early 70's on these soils without watertight seals were completely destroyed within a two-year period of startup. In Iraq, the Kirkuk project was partially constructed 10 years ago. Construction had progressed to the point that major excavation had been done, a plastic membrane had been placed and cover material, to a depth of 12 to 18 inches, had been placed over the plastic. An Egyptian contractor did the work using American materials. Before the contract was completed, the Egyptian was removed from the job and the canal was never completed. Thus the canal stood without water for 10 years.

The author had the opportunity to visit this jobsite and review the condition of the plastic and found the material badly deteriorated. The membrane had lost most of its plasticizer and was full of holes and greatly aged. An evaluation of what had happened showed that the earth had acted much like an oven during the summer months with temperatures rising to 125 degrees F. This alternate heating and cooling, even though the plastic was covered, had a very deteriorating affect. Had the canal been filled with water after the construction was completed it is possible that the plastic would still be in good condition. This confirms a point brought out by the Bureau of Reclamation that material installed above the water line does not hold up as well as that which is below water. A constant heating-cooling cycle applied to thin plastic membranes is not conducive to long life.

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Figure 14

Conversion of Material Thicknesses

1 inch	1,000 mils	25mm
.01 inch	10 mils	0.25mm
.02	20	0.50
.04	40	1.00
.06	60	1.50
.08	80	2.00
.12	120	3.00
.20	200	5.00

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The Role of Flexible Membrane Liners in Support of RCRA Regulations

This paper describes the current research program on flexible membrane liners (FML's) being conducted by the U.S. Environmental Protection Agency's (EPA's) Office of Research and Development (ORD). Three main program areas are discussed--prediction of service life based on liner-waste compatibility and liner resistance to various waste chemicals, installation requirements for FML's, and field verification. Also examined are the cooperative efforts of EPA, the liner industry, and the user community to develop the full potential for FML use in waste management facilities.

INTRODUCTION

Hazardous waste regulations for the Resource Conservation and Recovery Act (RCRA) of 1976 have been promulgated by the U.S. Environmental Protection Agency. These regulations suggest that flexible membrane liners (FML's) are to be used for certain kinds of waste management facilities. The Office of Research and Development (ORD) continues to support the development of these regulations through a dynamic FML research program. Major emphases are service life prediction based on liner-waste compatibility and liner resistance to wastes, improved field installation procedures, and long-term maintenance in the field.

RCRA applies to all landfills, surface impoundments, waste piles, and land treatment units used to treat, store, or dispose of hazardous waste. The design and operating standards of the Act require all these facilities (except land treatment units) to have liners that prevent the migration of waste or waste components (leachate) into the subsurface (soil, groundwater, or surface water) during the active life of the facility.

The use of FML's in waste management facilities provides the necessary barrier for preventing waste migration. But other components such as the leachate collection and removal system must also be incorporated in the overall design to ensure contaminant control in case one of the components fails.

Great care must be taken in selecting FML's as design components of hazardous waste facilities. Both their benefits and limitations must be fully appreciated for designers to achieve the greatest potential of these materials. To help users realize this objective, ORD has undertaken a substantial research program to improve our knowledge of how FML's function and react under a variety of physical, chemical, and biological conditions.

DESCRIPTION OF THE RESEARCH PROGRAM

The ORD FML research program was initiated in 1973 after users raised serious questions about the useful lifetimes of liners that had undergone only brief periods of laboratory testing. Such liners needed to last for tens of years in the field, but their test exposures had been limited to less than 2 weeks in small beakers. The early research program was directed primarily at developing a protocol for evaluating FML's exposed to wastes. Two major projects (1, 2) developed a data base of information on the reaction of FML's exposed to municipal and industrial wastes. The major conclusions of these projects were that (a) municipal and industrial wastes can be successfully contained by FML's, and (b) successful waste containment with FML's requires that these liners be pretested by exposing them to the actual waste to be contained.

These results prompted EPA to develop a multifaceted approach to determine whether FML's could be used with confidence in waste management facilities to protect both the public health and safety and the environment. Major program areas defined were the prediction of service life based on liner-waste chemical compatibility and liner resistance to various waste chemicals, installation requirements and procedures (including seams), and field verification. Each program area is defined here briefly.

Chemical Compatibility and Resistance

In this program, FML samples are exposed to wastes or simulated wastes (e.g., in a project with the National Sanitation Foundation, CR810727)*, followed by measurements of physical properties as a function of time and temperature. This phase of the program emphasizes understanding the results and interpreting the data. The rate of change suggests, but may not directly determine, whether the FML material is compatible with the waste to which it is exposed. Another approach (Drexel University, CR810977) being evaluated is the relationship between

*Contract awarded to institution and contract number.

resistance to chemical modification and change in permeability of FML's. These characteristics will then be related to the service life of a liner system when exposed to chemical attack.

Chemical resistance test data from other uses of polymeric membranes (protective clothing, food packaging, etc.) are being evaluated (A.D. Little, Inc., 68-01-6160) to determine their applicability to FML's of similar composition used in waste management. Along with this project, the basic chemistries (comparable solubility parameters) of the geomembrane and waste streams are being evaluated to determine whether chemistry alone can be used to predict compatibility.

Placing soil and soil-like materials over a liner exposed to chemicals is being evaluated as a means to increase service life. Early exposure tests suggested that this approach would decrease the chemical attack on the FML's. This subject was also suggested for research in a peer review of the liner research program. A study (Bureau of Reclamation, AD-14-F-3-A411) has been initiated to evaluate depth and type of coverings needed to obtain the benefit.

Future research activities on chemical compatibility and resistance appear promising. For example, the use of solubility parameters, hydrogen bonding indexes, and relative polarities have proved useful in the rubber and protective clothing industries for predetermining the compatibility of tank linings, hydraulic hoses, and piping within plants and for chemical warfare agents that may degrade clothing. As noted earlier, the technology is being re-examined to determine whether predictions can be made for FML and waste compatibility and for service life.

The research program is currently evaluating the Experts' System for determining chemical compatibility. This computer-based system is based on decisions being made on chemical compatibility by experts within the industry, particularly where no definite criteria exist and decisions are based largely on experience or judgment. This concept has been successfully used by the medical profession in diagnosis and by geologists in locating promising areas for exploration. The system is not hooked into any specific decision path; rather it can choose from alternative paths to develop conclusions. The line of reasoning can be displayed on request in human language form. Locating experts may be a greater problem than programming the computers.

Installation Requirements and Procedures

Soil Foundation and Placement Practices--

Once an FML has been selected for a site, other problems arise. Guidance is needed on soil bedding conditions and current placement practices. Projects have been conducted in these areas (3, 4) to provide users with information on how FML's are best handled in the field and what conditions are necessary for the soil foundation.

Seaming--

Current projects include the development of seaming criteria (Bureau of Reclamation, AD-14-F-3-A411). This project will compare the strength and durability of seams that are fabricated according to the manufacturer's recommendations under ideal laboratory conditions and those that are constructed at disposal facilities by installation contractors. In this project, seams are being exposed to chemicals for various periods of time and evaluated for changes in bonding and strength. Accelerated weather exposure of the seams is also being evaluated.

Leak Detection and Repair--

The potential for leakage through tears, parted seams, pin holes, etc. is being evaluated (Texas A&M, CR810940) for several FML types and thicknesses and various hydraulic head and sub-base conditions. Development of actual data on leakage rates should help determine what degree of quality control (quality assurance/quality control) is needed during liner construction and when imperfections must be corrected.

For those liner systems already in place and for future identification the ORD research program is developing leak detection techniques. Time domain reflectrometry (TDR) and acoustic emission monitoring (AEM) (EarthTech Research Corporation, 68-03-3030) have both been used successfully to locate known leaks in an experimental pond. The TDR method is technically feasible for new sites, but it may be too expensive. The AEM method does detect the leak, but the acoustic level of the source decreases rapidly to a point that is difficult to distinguish from background noise. Both techniques are being re-evaluated for cost effectiveness and technical improvements that can be made. The electrical resistivity (ER) technique (Southwest Research Institute, 68-03-3033) has demonstrated its ability to locate 1-, 5-, and 12-in.-diameter leaks in a 1-acre pond filled with water. Both single and multiple leaks were detected. Tests are continuing to determine the effects of liquid depth and to correlate ER response to leak size.

The repair of leaks in situ is the objective of another study (EarthTech Research Corporation, 68-03-1714). The project has identified several grouts and procedures that could be used for repairing known leaks.

Future Research--

Future activities in the area of liner installation include determining the effects of upward hydrostatic forces on liner integrity and of long-term point loading. Placing soils on membrane surfaces at steep slopes will also be evaluated. The intent is to determine the conditions under which the soil starts to slide off the membrane and the measures necessary to correct the condition. Future studies may also evaluate the use of geotextiles and plastic nets to increase soil cover stability for steeper side slopes.

Quality control and quality assurance (QA/QC) will be addressed. Though it is not difficult to develop a design for a waste management facility, it is very difficult to achieve that design in the field. Experience has shown that a good QA/QC plan must be developed and implemented for all phases of construction.

FML's are being used more frequently for upgrading existing facilities that may be leaking. Existing and new facilities may require mechanical connections (e.g., to drain pipes). Future studies will systematically review such connections for consistency of performance and application.

Though FML's are not considered as structural members of the facility, their physical properties can be used to prevent problems. The use of FML physical properties will be reviewed to identify their limitations in design of land based waste management facilities.

Field Verification

Most ORD research on FML's has been conducted in the laboratory at the bench scale, but samples exposed in the field have been positively correlated with laboratory-exposed samples (5). Other field data are being collected to determine why systems failed or succeeded (A.D. Little, Inc., 68-03-1771; Woodward-Clyde, 68-03-1772). Improved procedures for compatibility testing, placement, and field maintenance of FML's will be identified by these results.

Since field sites represent real-world conditions, it is necessary to obtain samples that represent these. Each site is unique and presents a different challenge to the membrane. The ORD research program continues to seek sites that have failed, are being retrofitted, are decommissioned, or otherwise offer samples that can yield long-term performance data.

TECHNICAL RESOURCE DOCUMENT

Research results from ORD, manufacturers, and users and information from other related industries have been combined into a single document (6). This document is perhaps the single most comprehensive report on the use of FML's in waste disposal. The data will updated and revised periodically as research and field results become available.

INDUSTRY PARTICIPATION

From the beginning, the ORD liner research program has been conducted with the full participation of industry. They have offered materials and information, and they have suggested ideas that could be mutually explored to improve the credibility of FML's.

The Agency has a strong policy regarding the quality of its research and reports. Industry has contributed considerable time and effort to helping conduct program reviews and to assessing the quality of Agency reports. This excellent response has helped provide a more complete understanding of research results.

Perhaps the single most important contribution by industry is the participation and development of Standard 54, the FML standard recently published by the National Sanitation Foundation. Though it has been under development for 4 to 5 years, the standard will lead the way in instilling confidence in the user community that FML's can be used in waste management facilities as well as other applications. The standard is expected to be improved as additional information becomes available. Industry has already indicated its commitment by expressing a willingness to continue participating.

CONCLUSION

ORD's continuing research program on FML's is essential to expand their use for hazardous waste disposal and to assure that they will perform as designed. Continued support from industry is required for acceptance and growth of this technology.

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PVC/EVA Grafts—A New Polymer for Geomembranes

The objective of the study was to develop a polymeric film that would satisfy the major criteria required of an exposable geomembrane. A variety of possible polymer combinations based upon PVC technology were explored leading to the synthesis of a PVC/EVA graft copolymer. Compounding methods for the PVC/EVA graft are explored briefly. Finally, test data including weatherability studies, chemical resistance and general physical properties are presented suggesting that this base polymer, properly compounded, is well suited for exposed geomembrane use.

INTRODUCTION

Plasticized Polyvinyl Chloride has been used quite extensively and quite successfully as a geomembrane. In the past, it was necessary to bury the PVC to get extended service life due to the destructive effects of sunlight (polymer & plasticizer degradation) and temperature (polymer & plasticizer degradation, plasticizer volatility and low temperature flexibility problems).

These problems are not adequately solved by the use of traditional PVC technology. For example, a logical approach is the use of polymeric plasticizer - typically, a polyester based on Adipic acid. While such plasticizers resolve the volatility problem, they cause additional problems such as high sensitivity to biological degradation and extreme sensitivity to alkaline environments. Polyblending or alloying of PVC with other polymers which provide a similar level of flexibility to that of the typical plasticized PVC have been used with limited success. Choosing PVC compatible polymers, we note the following deficiencies:

Nitrile Rubber/PVC - Poor weathering.
Polycaprolactone/PVC - High price, moisture sensitivity.
Chlorinated Polyethylene/PVC - Poor physical properties, unsupported and difficult to process.
Polyurethane/PVC - High cost, hydrolytic instability, high permeability.
Ethylene/Vinyl Acetate/PVC - Difficult processing & marginal physical properties.

In seeking out a new polymer for use as a geomembrane, we looked for the following properties:

1) Resistance to weathering, so it can be used in exposed applications.
2) Good low temperature flexibility, so that it can be installed in cold weather and will not shatter on impact.
3) Resistance to biological degradation such that it can be buried or put in direct contact with the soil.
4) Physical properties comparable to plasticized PVC such that it can be used without fabric support.
5) Resistance to the common chemicals found in sanitary landfills and municipal liquid wastes.
6) Better resistance to oils and alkali than is available in common liner material on the market today.
7) We sought a polymer whose stability and physical properties were not dependent upon high loadings of carbon black. This would allow for lower heat buildup in the membrane and more compounding flexibility.
8) Moderate cost.
9) Heat, solvent and dielectrically weldable.
10) Finally, easily processible on common plastics processing equipment.

POLYMER DEVELOPMENT

From the outset, PVC/EVA polyblends appeared to offer the most promise in that they satisfied most of the requirements, except they suffered from marginally poor physicals and were a horror to blend and to process on extruders or calenders.

The literature was replete with references to EVA/PVC graft polymers but, unfortunately, most of the work related to the use of the polymers as impact modifiers for rigid PVC and little was reported on this material processed in neat form.

These graft polymers are synthesized by dissolving an ethylene/vinyl acetate copolymer (minimum 40% vinyl acetate content) in vinyl chloride monomer. The subsequent polymerization is then carried out in aqueous suspension using standard free radical initiators and suspending agents.

After completion of the polymerization, the slurry is steam-stripped to remove residual monomer, centrifuged and dried.

Graft polymers containing from 5% to 60% EVA have been synthesized and levels of 40% to 70% VAc in the EVA moiety have been used.

The molecular weight of the PVC portion of the polymer is controlled by temperature and/or chain transfer agents.

The grafting efficiency is controlled by:

a) total EVA level,
b) % vinyl acetate in the EVA moiety,
c) polymerization temperature,
d) initiator concentration, and
e) initiator type.

From the above description, it is obvious that a broad family of graft copolymers are possible with wide-ranging physical and chemical properties.

THE 50% GRAFT POLYMER

It was quickly recognized that polymers containing less than 30% EVA did not possess suitable elasticity for use as geomembranes. Therefore, for our initial effort, we synthesized a graft polymer containing 50/50 PVC/EVA using an EVA with 45% vinyl acetate and a number average molecular weight of 100,000-120,000.

The literature(1) suggested we might expect the following comparison to plasticized DOP:

TABLE I

	PVC/DOP 50/50	PVC/DOP 65/35	Graft 50/50
Tensile Strength, psi	1491	2980	2550
Elongation @ Break, %	420	316	260
Shore A Hardness	55	80	85
Brittleness Temp., °C	-45	-30	-55

However, we needed to determine the efficacy of the grafting process vs. physical mixing of PVC & EVA.

Pearson(2), working in our laboratories in 1981, did this work in a wire & cable-type formulation and his work has since been confirmed in more typical film and sheeting recipes.

TABLE II

	Graft Copolymer No.2	Mechanical Mix
Hardness, Shore A, 10 sec.	92	72
Tensile Strength, MPa (psi)	13.6(1965)	3.4(490)
Elongation, %	250	200
Modulus @ 100% Elong., MPa (psi)	13.4(1950)	3.4(490)
Oven Aged 7 Days @ 158°C		
% Retention of T.S.	140	Melted
% Retention of Elong.	54	—
Aged 18 Hrs. @ 121°C in ASTM #2 Oil		
% Retention of T.S.	56	Melted
% Retention of Elong.	56	—
Brittle Point, °C	-30	-12
Oxygen Index, Vol.%	35	32
Deformation @ 136°C (2000 gram wt.,%)	33	98

As the work proceeded, it became more and more obvious that grafting the EVA onto the PVC nicely resolved the physical property problem associated with the prior known mechanical mix data.

PROCESSING

By virtue of chemical combination (covalent bonding) of two high molecular weight species, we had managed to make elastic the typically brittle homopolymer PVC and considerably improve the strength of the EVA. In joining together such large molecules, we considerably restricted molecular movement, and processing this material became an unusual challenge. Thus, to achieve plastic flow through conventional thermoplastic processing equipment (extruders and calenders), the temperature had to be raised to a point that, in a short time, dehydrohalogenation occurred on the PVC which in turn created the atmosphere for polyvinyl acetate decomposition to acetic acid, etc.

In order to overcome these difficulties, special multifunctional stabilization systems had to be developed to prevent polymer degradation and additives had to be found that would allow the polymer to be processed at lower temperatures.

While this research still continues, these processing problems are largely behind us.

While the stabilization research followed rather traditional lines, the temperature/flow modification studies led to yet another interesting find.

We knew that EVA has a very non-polar end (ethylene) and another very-polar end (vinyl acetate) and acts like a "polymer surfactant" and is therefore compatible with many dissimilar polymers and may act as a bridge to help compatibilize polar and non-polar polymers.

We found that this type of behaviour now carried through to the graft polymer and that our graft was now compatible with a broad variety of other polymers and other compounding ingredients. This characteristic of the graft polymer helped us find suitable means of compounding the material such that it could be readily processed on conventional equipment.

There would appear to be more to be done in the use of this material in alloying with other polymers.

PREPARATION OF THE GEOMEMBRANE

The graft polymer being in free-flowing powder form was compounded with stabilizers, reinforcing agents, pigments and lubricants, etc. in a ribbon blender, masticated in a Banbury mixer, further processed on a two-roll mill and calendered on a 4-roll, inverted L typical flexible PVC calender. 15, 20, and 25-mil-thick sheeting was prepared with few processing problems. The sheeting as such was conditioned for 24 hours and tested -see Table III- and some was laminated into a typical 3-ply, 36-mil geomembrane supported by a 10 x 10-1000d polyester scrim.

TABLE III

PROPERTIES OF 0.020" SHEET

Property	ASTM Test Method	EVA/PVC Graft	Monomeric Plasticized PVC	Polymeric Plasticized PVC
Shore A Hardness	D2240-81	88	86	87
Tensile Strength, psi	D882-75	2500	3200	3100
100% Modulus	D882-75	1400	1800	1900
Elongation @ Break, %	D882-75	360	320	305
Cold Crack °F	D1790-76	-40	+15	+40
Brittleness Temp. °C	D746-63	-55	-38	-21
Graves Tear lbs./inch	D1004-66	325	380	410

TESTING

a) Chemical Resistance

In a series of standard chemical immersion tests, the unsupported film was compared to plasticized PVC, oil-resistant PVC, chlorinated polyethylene, and chlorosulfonated polyethylene. Table IV details these tests.

TABLE IV

PERCENT WEIGHT CHANGES @ 158°F FOR 4 MONTHS, ALL FILMS 0.020" THICK

Effluent	PVC	Oil Resistant PVC	CPE	CSPE	EVA/PVC Graft
3% H_2SO_4	+0.2	-6.6	+18.3	+9.9	+5.5
10% HNO_3	+44.1	+18.36	blisters @ 1 mo.	blisters @ 1 mo.	cracked @ 2 mo.
10% HCL	+18.5	-5.3	blisters @ 2 mo.	+83.9	+14.4
10% NaOH	-29.4	-24.1	+4.4	+18.1	+0.4
10% NH_4OH	+461.0	+63.2	blisters @ 2 wks.	+295.0	+21.8
10% NaCL	-0.39	-9.15	+0.60	+0.76	-0.25
ASTM #1 Oil	-26.2	-13.6	-4.65	destroyed @ 24 hrs.	+6.7
Distilled Water	+1.9	+18.9	+139.6	+35.4	+14.2
Crude Oil	-22.9	-7.5	destroyed @ 1 wk.	destroyed @ 24 hrs.	+40.0

CPE = Chlorinated polyethylene
CSPE = Chlorosulfonated polyethylene

b) Weatherability

Exterior weatherability studies were carried out in the

1) Atlas Weatherometer
2) QUV
3) Atlas UVCON
4) Florida -- 45° south
5) EMMAQUA
6) RSM Sunlamp

Table V lists comparable information accumulated to date. These tests are ongoing.

TABLE V

PERCENT RETENTION OF ORIGINAL VALUES UPON WEATHERING OF 0.020" THICK FILM

Test Method	100% Modulus	Elongation at Break	Tensile Strength at Break
	PVC/EVA Graft		
3000 hours Atlas UVCON	135	70	100
3000 hours C-arc Weatherometer	117	106	105
3000 hours RSM Sunlamp	107	71	95
3000 hours Q-Panel	117	105	106
24 months EMMAQUA	115	60	80
12 months Florida 45°S	107	105	94

CONCLUSION

We have presented several studies that suggest that a geomembrane with an excellent balance of properties may be produced using a PVC/EVA graft polymer as base polymer in a specially compounded system.

This new noncuring material, in a variety of colors, is moderate in cost, weatherable, easily weldable by all standard methods, has good physical properties, excellent chemical resistance, remains repairable, is strongly resistant to biological attack, and has good low temperature properties. Properly compounded, the graft polymer is easily processed on conventional thermoplastic processing equipment.

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PVC/Ethylene Interpolymer Alloys for Plasticizer-Free Membranes

The market for geo-membranes is not only growing, but it also requires improved membranes. European industry in particular tends to use traditional materials, mainly soft vinyl and butyl rubber, but design engineers need more durable and more chemically resistant membranes which are easy to install, fabricate and repair. PVC resin alloyed with a special ethylene interpolymer (EI) results in a flexible but plasticizer-free material. This EI is a solid, tough plastic material in its own right developed to form homogeneous blends with other polymers. Membranes produced of such blends maintain all the known advantages of PVC plus a high degree of durability. Exposed to a number of agressive fluids, sheets of PVC/EI exhibit no stiffening and shrinkage whereas ordinarily plasticized vinyls suffer weight losses up to 35% under the same conditions. Ample availability of the basic ingredients and use of standard PVC converting equipment enhance widespread use . Several years' satisfactory performance pf PVC/EI membranes used for demanding applications is reported.

Impervious membranes are used to form a barrier to contain water or a pollutant. Methods for waste disposal such as recovery, encapsulation prior to disposal, ocean dumping, deep welling or incineration are creating new problems or are simply too high in cost. A growing demand for protection of nature, prevention of ground water contamination by industrial waste and safe storage of scarce drinking water is contributing to a promising future for the membrane industry. Projecting growth is difficult as the demand for membranes depends heavily on legislation. Many countries have not started to survey contamination but are expected to do so soon, in response to environmental pressure groups.

Looking at European and neighbouring markets, we have however not observed the high degree of interest by existing suppliers and the formation of new companies that normally accompany a rapidly expanding market. Reasons for this may be due to the traditional tendering system common throughout the construction industry, the high degree of liability or lack of possible differentiation between services and types of membranes. The strongest pressure is applied on the membrane manufacturer to produce a commodity product at the lowest price. Not only does this make it very difficult for a newcomer to enter the market, but it also makes development and use of newer and better membranes unlikely. There is, however, a strong demand for improved membranes according to a survey carried out recently among installers and specifiers. Membranes are needed which :

- can be welded faster to save labour
- give stronger seams to prevent frequent leaks
- are lighter in weight to permit larger fabricated sheets
- have better resistance to hydrocarbons and extreme of temperature.

Most specifying engineers lacking detailed liner material knowledge opt for traditional membranes. Sheet manufacturers on the other hand perceive no demand for improved materials and are therefore not encouraged to develop, test and promote any new material.

The most widely used traditional flexible membranes in Europe are certainly soft vinyl and factory-cured butyl rubber sheetings. Both products have inherent shortcomings. Soft vinyl particularly lacks durability, whereas cured rubber sheetings basically lack the ease of seaming of thermoplastic materials. No other flexible material has yet had a real impact on the market in Europe.

Requirements

A membrane ideally suited to all possible applications will probably never exist. However the task of specifying engineers would be greatly facilitated by wide availability of a membrane offering all essential properties needed for the majority of applications.

Requirements for flexible membranes in particular are :

mechanical	- tensile strength, elongation, creep and puncture resistance according to the application. Reinforcement should be done whenever necessitated by a particular application but should not be the rule - new difficulties are introduced with reinforcements.
environmental	- resistance to most widely encountered chemicals - weatherability in all climates - resistance to UV exposure - immunity to bacterial and fungal attack.
installation	- light weight, ease of roll and sheet handling - rapid, strong and reliable field seaming - ability to reliably repair.
economical	- production on standard, widely available equipment to ensure sufficient supply sources - widely available, low cost ingredients.

Soft vinyl sheeting seems to meet all the requirements listed. It can be made using state of the art equipment and ingredients to have outstanding physical properties. It has the ability to be welded to itself to form a tight seal. PVC is easy to solvent-weld or heat-weld with simple equipment, while minimizing worker exposure to solvents and other pollutants. Seam peel strengths obtained are severalfold greater than those obtained with contact adhesive - an important advantage over the normal overlapped seam.

In spite of those many advantages, PVC does not enjoy a good reputation. Numerous past failures are cited to prove the point. But the only real problem of soft PVC is the limited environmental resistance; the sheeting lacks durability, due to loss of the liquid plasticizer which is used to soften the rigid PVC polymer. Loss of plasticizer readily occurs due to extraction (e.g. in the presence of hydrocarbons), volatility at elevated temperature and migration into materials in contact with the membrane. The main ingredient of soft vinyl, namely the PVC resin, is never in question. This material has high mechanical strength, excellent chemical and environmental resistance - rigid sheeting is used for lining storage tanks for very agressive chemicals and weatherability, proven by over 30 years successful use for house siding and window frames in all climatic zones. PVC is available worldwide with production in all industrial countries, and it is low cost.

Ethylene Interpolymer

With the development of a new polymer by Du Pont in 1973, the PVC sheeting industry was offered a new way of making soft vinyl membranes without the use of conventional plasticizers. Although blended with PVC resin in the conventional way and functioning as a plasticizer to flexibilise the rigid PVC resin, this new polymer differs dramatically from any plasticizer (Table 1)

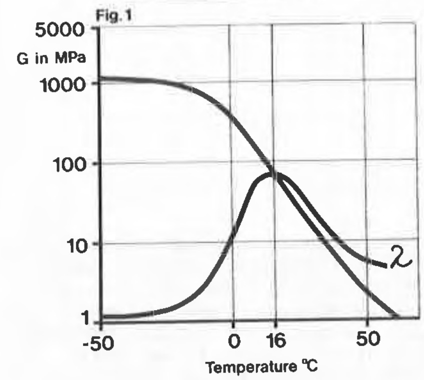
Table 1

PROPERTY	PLASTICIZERS	ETHYLENE INTERPOLYMER
Physical form	liquid	solid, granular
Molecular weight	300-9000	< 250 000
Tensile strength, MPa	-	4,2
Modulus of elasticity at 100%, MPa	-	2,8
Elongation %	-	1200
Glass transition temperature (Tg) ^{o}C	-	-36
Specific gravity g/cm3	0,98-1,05	1,02

From the table, it is evident that this ethylene interpolymer is a solid, high molecular weight plastic material in its own right. Since its polarity is similar to PVC, both polymers are fully compatible and form a true plastic alloy.

Figure 1 shows the torsion modulus against temperature of a typical PVC/ethylene interpolymer blend used for liners. The fact that there is only one glass transition temperature (Tg) at $16^{o}C$ expressed as one clear maximum of the logarithmic decrement λ proves the formation of a completely homogeneous material. PVC alone would have a Tg of $+80^{o}C$ and the interpolymer one of $-36^{o}C$.

TORSION MODULUS PVC/EI



Production of PVC/Ethylene Interpolymer Membranes

In developing flexible liner formulations incorporating interpolymer in lieu of liquid plasticizer, it is necessary to use higher amounts of interpolymer in the formulation than liquid plasticizer to give the same membrane hardness. The amount might be an additional 30% compared with standard DOP, whereas only slightly higher additional amounts are applied when compared with more viscous polyester plasticizers. PVC always remains the major portion in a typical formulation. Stabilizers and processing aids are used as normal for PVC. Pigments need to be added for prolonged outdoor stability. Sheet production is essentially the same as for conventional soft vinyl. Both widely known processes - sheet extrusion and calendering - are used. Care must be used in blending since the solid interpolymer is not absorbed by the PVC powder as are liquid plasticizers. To form a homogeneous blend, processing equipment must be set to obtain sufficient shear stresses during compounding. Reinforcement, if desired, and laminating to form thick sheets, are easily done either on or off calenders due to the excellent adhesion properties of the alloy.

Properties of PVC/Ethylene Interpolymer Membranes

Let us now investigate the properties of a plasticizer-free, unreinforced PVC/ethylene interpolymer membrane of a typical 1,2 mm thickness.

Typical mechanical properties are :
- tensile strength 28 MPa
- elongation at break 360%.

These values which are high for flexible liner material are however also obtainable in soft vinyl. Dramatic changes of properties might occur, however, once a soft vinyl sheeting is subjected to the conditions that membranes will encounter in use. The weight change before and after exposure of a sample to a specific environment is commonly used to indicate a change in properties. For example a soft vinyl will change properties with loss (reduction) of plasticizer content as shown in Table 2.

Table 2

PVC/DOP SHEETING

	Property Change % of initial value	
Plasticizer loss, % weight	-10%	-20%
Tensile strength	140	290
Elongation	66	25

Reduced plasticizer content results in severe stiffening and also in considerable sheet shrinkage leading quickly to sheet failure through cracking.

Exposing membrane samples to some typical conditions and test fluids were obtained weight losses as shown in Tab. 3. Liquid plasticizers performed differently depending on their molecular weight. Apart from the most commonly used DOP, a polymeric, medium-high molecular weight polyester plasticizer was used for comparison.

Table 3

PERCENT WEIGHT LOSS OF SHEETING SAMPLES

		% Weight Loss		
		Soft Vinyl		
Medium / Exposure condition		DOP	Polymeric plasticizer	PVC/ethylene interpolymer (incl. 5 PHR epoxydised soyabean oil)
Alkalis	1% soap water / 1 day, RT	-6	-5	no loss
	Detergent / 7 days, 30°C	3,5	-8,7	no loss
	NaOH (25%) / 8 days, 60°C	-20,8	-8,8	-2,2
Hydrocarbons	Hexane / 1 day, RT	-28	-6	no loss
	Isooctane / 8 days, 60°C	-26	-6,5	+ no loss
	Isooctane / 50% toluene / 8 days 60°C	-28	-25	-7,6
	Lead-free gasoline / 1 day, RT	-22	-13,5	no loss
Oils	ASTM No 3 oil / 4 days 70°C	-11	+ no loss	no loss
	ASTM No 2 oil / 4 days 70°C	-13	-3	+ no loss
Other	Methanol / 8 days 60°C	-28	-34,9	-6,4
	Hot air / 7 days, 70°C	-3,6	-1,4	-1,4
	Hot air / 1 day, 90°C	-9,4	-1,5	-0,5

Significant loss of weight,particularly after such short exposure,indicates a material's unsuitability to contain these fluids. Small weight gains, although not desirable as they will generally result in a lowering of tensile properties and increasing softness, are less critical since any swelling will quickly reach a maximum and the resulting "new" physical properties will be maintained throughout the service life. In cases where these new properties are inadequate, a scrim of fabric reinforcement will assure sufficient tensile and creep properties. Swelling is also reversible with the possibility that the absorbed fluid can be dried out and the original properties restored. Loss of plasticizer is permanent, however. Having eliminated the most likely source of failure of a PVC membrane by eliminating all liquid plasticizer, let us consider other important membrane properties.

UV Resistance and Weatherability

Apart from satisfactory results from numerous accelerated tests carried out over 10'000 hours, actual installations over more than seven years in many parts of the world attest to the suitability of a PVC/ethylene interpolymer liner when exposed to various climatic conditions.

Installation

Substitution of plasticizer by ethylene interpolymer does not impair the excellent welding characteristics of a vinyl membrane. Peel strength values over 50 N/cm are obtainable using standard hot air, hot plate, dielectric or solvent welding. The true thermoplastic nature of the material also allows repair using the original welding methods, even after several years. With a density of 1,15 g/cm3, light-weight easy to handle rolls and prefabricated sheets can be made. Due to the absence of migrating plasticizer and the high proportion of PVC resin in the alloy, these membranes have no tendency to stick together and are essentially block-free, facilitating fabrication and installation. Also during calendering, the material does not adhere to the calender rolls, thus giving very stable and uniform dimensions. Shrinkage values are typically less than 0,5% in machine and cross direction measured on unreinforced sheeting after 22 hours conditioning at 70°C. The high molecular weight of the ethylene interpolymer also contributes excellent resistance to tear, low temperature impact and creep, eliminating the need for reinforcement in most applications. Typical values are shown in Table 4.

Table 4

	Soft Vinyl (DOP)	PVC/EI
Tear resistance, ASTM D 1044 kN/m	46	52
Impact brittleness temperature, Masland °C	-18°	-40°
Creep %, 60°C, 7 days	2,5	0,5

Proof of Performance

Membranes made from PVC/ethylene interpolymer have been in commercial use since 1976. First uses sought were particularly demanding applications for which no other liner was considered suitable. Membranes have been laid in various climatic zones of the USA including Alaska as well as in Africa and in Europe. All these installations are todate fully functional and performing well. A few examples of such uses are described in more detail.

Solar Pond Liners

In salt gradient solar ponds, water temperatures can approach boiling point. The highest temperatures, varying between 50°C in winter to 90°C in summer, occur in the bottom layer of the pond where salt concentration also reaches a maximum of 27%. This combination of heat and salt is particularly difficult to handle. In addition, material surrounding the top of the pond must withstand exposure to direct sunlight. The liner chosen for almost all solar ponds installed in the USA since 1978 is a membrane produced from high tenacity industrial fibers coated with a special compound containing the ethylene interpolymer.

The picture shows the salt gradient solar pond built by the Tenessee Valley Authority in 1982. It is filled with 11360 m3 of water and 2000 tons of salt. The impermeable liner prevents leakage of the brine solution into the surrounding ground.

Biogas Collection Cover

Collection of gases generated by fermentation of organic waste and their use for heating purposes is a modern technology to turn a pollution problem into a new source of energy.

Various projects taking advantage of the development of biogas containing 70% valuable methane are under study. Whereas the process may not yet be economical for the majority of farmers, it is definitely practical for larger industrial food processors.

Several European producers of frozen prepared potatoes for instance, employ bulk volume fermentation of their waste water in lagoons. Key to the system's success is a floating, impermeable, gastight membrane which needs to be resistant to vegetable oils and greases as well as to bleaches and other chemicals used in the process.

Resistance to microbial attack at temperatures of 40°C is also essential. The photograph shows an installation at Harnes in France where a PVC/ethylene interpolymer membrane covers a 6000 m2 lagoon. A vacuum under the cover allows gas to be pulled off and transported through a series of pipes supported on floats. The pipes merge with a central duct that takes the gas directly to the plant's boilers.

Crude Oil / Water Deballasting Pond

To stop modern oil tankers emptying their ballast water contaminated with residual crude oil into the sea, deballasting ponds are used at crude oil terminals. There the oil floating on the water can be easily separated from the water. The membrane used to line these containment ponds must resist a number of adverse conditions. Apart from the strong attack from the oil concentrated at the top of the water the extreme climatic conditions at many terminals are a real challenge for the membrane. The terminal of Trabsor in the Gulf of Gabes in Southern Tunisia e.g. is exposed to cold winters with temperatures below freezing point and extremely dry and hot dessert summers. This combination is so severe that it had previously led to the failure of a soft vinyl membrane plasticized with a polymeric plasticizer only 8 months after installation.

To retrofit this 25 000 m2 pond a PVC/EI membrane has been used. In use now for over two years no sign of deterioration can be detected and a satisfactory service life of 10 years and more is expected.

Easily installed chemical resistant, durable membranes of plasticizer-free PVC/ethylene interpolymer are available from a growing number of sheet producers in the USA and in Europe. The use of these materials will contribute to a safer cleaner environment all over the world.

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Membranes and Elastomer Bitumens

Waterproofing a construction and protecting it from water is an ever-present question. Among the solutions which arose over the last few years, bituminous and specially modified bituminous geomembranes reinforced by a fabric, are a pertinent and economical one.

In order to achieve a geomembrane with the performances required on site such as the resistance to various forces and aggressions, cold and heat, ageing ; it is necessary to pay attention to all the components. Bitumens have to be analysed, the proportionning and the molecular weight of the polymer have to be checked, the fabrics have to be chosen in respect of conditions of use.

The paper deals with the means used in laboratory and in plant for this purpose.

A. GENERAL

Waterproofing a construction and protecting if from water is an ever-present question. Over the last few years, different sources of pressure, such as the appearance of new materials, fibres and polymers, the cost of raw materials and manpower, have caused application techniques and waterproofing materials to make considerable progress. Among these materials, bituminous prefabricated geomembranes are a particularly pertinent solution to the problem posed. They are relatively easy to lay with reproducible characteristics and have performance levels which can be modulated according to the way they are used.

What are the characteristics that such membranes should have to cope with the job to be done? Without attempting to be exhaustive, we think that the following properties are essential :

- Resistance to instantaneous and permanent forces,
- Resistance to cold and heat,
- Resistance to different forms of ageing,
- Aptitude for application which allows for shortcomings in the site.

Now let us review the various component parts of a membrane in order to obtain such properties. It will be seen that a membrane consisting of a fibre reinforcing agent (glass, polyester, polyamid or others), a bitumen or a polymer bitumen mix, meets all the above criteria economically.

In the following, we propose to demonstrate how the components are chosen and how a bitumen geomembrane is manufactured and checked.

The first of the components of a bitumous membrane is the mix because it will ensure the following functions :

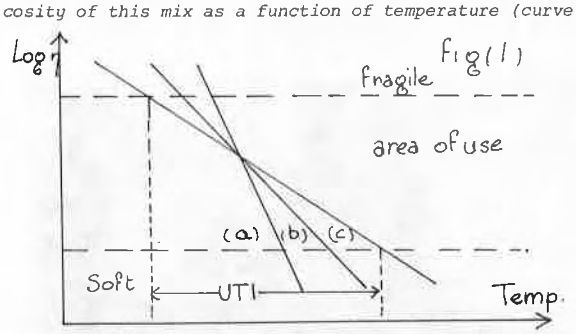
- Waterproofing,
- Resistance to cold and heat,
- Resistance to ageing,
- Aptitude for application.

These mixes are generally modified bitumens.

B. BITUMEN AND ITS MODIFICATION

1. AREA OF USE

The simplest bitumen is straight-run bitumen as represented in the graph of figure 1 where the ordinate is the viscosity logarithm and the abscissa the temperature ; we have plotted a typical straight line representing the viscosity of this mix as a function of temperature (curve a).



From this curve it can be seen that mix passes through three stages :

- The upper part in which the mix is extremely fragile. The temperature corresponding to the moment when the mix passes through the fragile state is known as the fragility temperature.

- The lower part in which the mix has no consistency. The temperature at which the mix becomes liquid is the softening temperature.

- Between the fragility temperature and the softening temperature is the mix utilization zone which we have termed the UTI or Useful Temperature Interval.

Note that straight-run bitumen has a low UTI. This is due to its chemical composition. In fact, it contains little asphaltene, a great amount of oils and aromatic resins and very little saturated oil.

2. MODIFICATION OF BITUMENS

a. Blowing

Very soon, an attempt was made to widen the utilization range of straight-run bitumen. To do so, straight-run bitumen is oxydized in a controlled manner. This is equivalent to greatly increasing the content of asphaltenes and substantially decreasing the resin content. Under these conditions, by aggregation of asphaltenes, the bitumen is transformed into a gel state. This operation results in a bitumen whose softening temperature is far higher than that of the original bitumen and whose fragility temperature is slightly higher than that of the original bitumen.

The curve (b) of figure 1 describes the typical performance of this type of bitumen whose UTI is far greater than that of straight-run bitumen.

b. Addition of polymers

With this new technique, a polymer network is created within the bitumen. This microscopic network considerably modifies the chemical balance and characteristics of the bitumen. Such modifications relate both to the rhéological characteristics of the bitumen, thus to the increase of the useful temperature interval (UTI), but also to the mechanical characteristics of the mix, and more particularly to its elasticity. The curve (c) of figure 1 describes typical performance of such a bitumen with an even higher UTI. To succeed in this modification, the polymer and bitumen must be compatible and accordingly, bitumen analysis is necessary. To do this we have developed a thin film chromatography technique (photo 2).

Photo 2

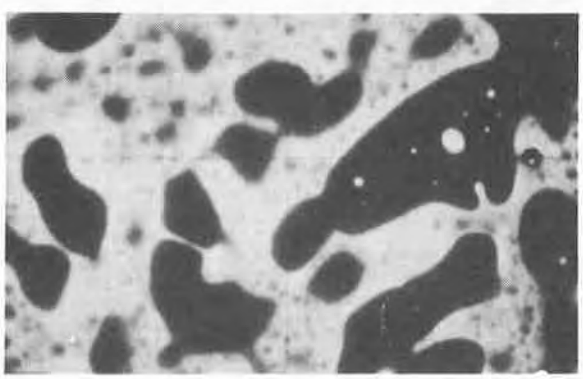
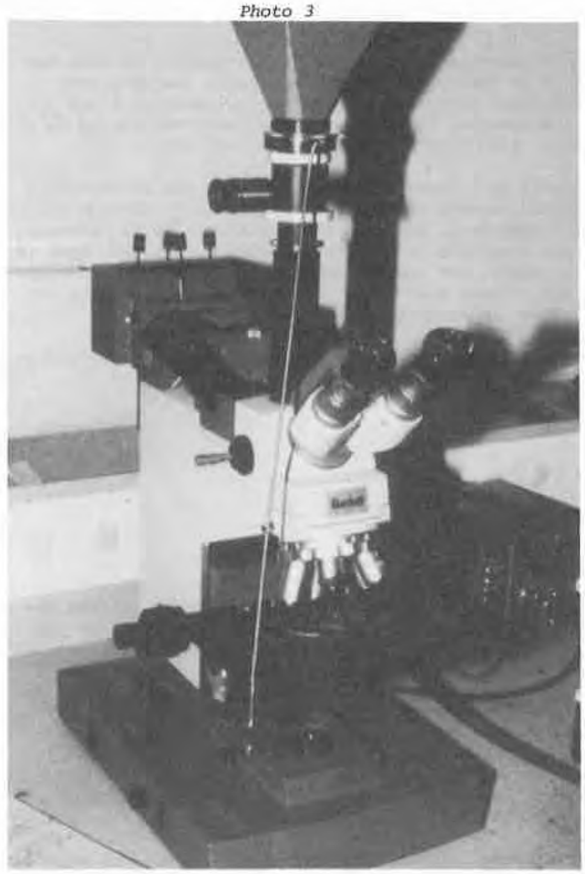


Photo 4

Further, the polymer must be properly dispersed in the mix and form the continuous phase of the mix. To do this, we have developed a technique of fluo-microscope examination (photo 3) and, as an example, we show a poorly mixed elastomer bitumen (photo 4) and a well mixed product (photo 5). The type of polymer used is a styrene butadiene styrene copolymer. This type of homogeneous continuous phase can only be obtained under very accurate polymer proportioning conditions when the chemical formula of the bitumen is correct and using appropriate means of dispersion.

Photo 3



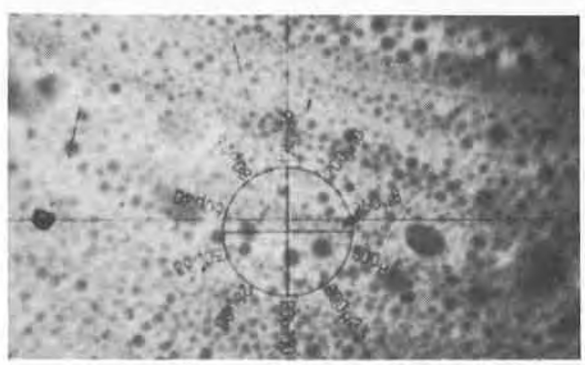


Photo 5

3. PROPERTIES OF BITUMEN MIXES FOR GEOMEMBRANES

In the following table 6 (below), the main properties of the bitumen binders found in widely used materials can be seen.

	BLOW BITUMEN		ELASTOMER BITUMEN			
			General purpose		for extreme cold	
	Before	After 3m. at 70°C	Before	After 3m. at 70°C	Before	After 3m. at 70°C
TH	70°C	120°C	110°C	120°C	90°C	95°C
TC	0°C	30°C	-30°C	-20°C	-40°C	-40°C
E	50%	2%	1500%	1200%	1500%	1500%
R	0,1	0,5	0,6	0,5	0,6	0,6
W	Excel-lent		very good		Excel-lent	

Table 6

- The utilization temperature in hot environments is the temperature that the membrane will tolerate permanently without any appreciable deformation (TH).
- The low temperature fragility temperature is the temperature at which material will tolerate bending or extension (TC).
- The resistance and breakage extension measure the aptitude of the material to tolerate major movements of the support (R, E).
- The site work ability is evalued by the facility of welding the materials together to form a watertight membrane in the presence of dust, mud or water (W).
- There may be differences in performance between the first product, which has proved satisfactory over approximately ten years of use, and elastomer bitumen based products, which are derived from the first and have the same workability advantages but which apply to a wider scope of use.

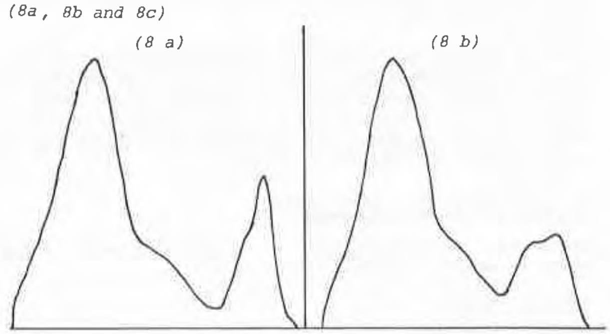
4. AGEING

However, even more than these properties of new mix, it is in particular the ageingproperties of polymer-bitumen mixes which must be maintained. These new mixtures are used under extremely severe conditions of heat, cold, movement of support and at the same time, we demand excellent long life of them. Therefore, it is important to know accurately what the ageing of these mixes is and to do so, it is necessary to follow the evolution of the continuous phase of the mix during ageing. That is why we have developed specific methods for measuring ageing by the proportioning of polymer and its degradation by gel permeation chromatography (GPC) (photo 7).

Photo 7

To illustrate these theories, see the photos below (8a, 8b and 8c)

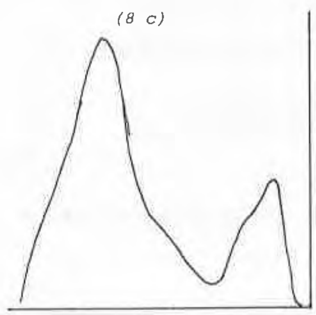
(8 a) (8 b)



The chromatogram of a new SBS-bitumen mix and After three months of artificial ageing

The elastomer peak is to the right of the bitumen peak.

(8 c)



The chromatogram of an SBS mix from a membrane after 10 years on site.

If we use the different chromatograms and the general properties given in the table (6) it is derived that one may expect a life on site of about 30 years at least.

C. FABRICS

The second major component of a membrane is its fabric.

The fabric can consist of several elements. It is the fabric which supplies :

- Resistance to aggression and instantaneous or permanent forces generated both by the support and by the surface,
- The geometrical shape of the product.

The choice of fabrics is of great importance and must be adapted to the service and utilization conditions.

Several types of armatures can be used. Among those most often used we can mention :

- Woven and non-woven glass fibers,
- Woven and non-woven polyester fibers.

Glass fiber fabrics are preferable when thermal dimensional stability is important.

Polyester fiber fabrics are preferred when great extensibility is necessary.

Woven fabrics should be preferred to non-woven fabric when mechanical properties are of the utmost importance.

Depending on the case in hand, it is also possible to associate two different types of fabrics to obtain complementary properties.

Generally speaking, because of the performance levels required of geomembranes, non-woven polyester fabrics are those offering the best price to quality ratio. Often, they are used in association in a geomembrane with non-woven glass fiber providing the membrane with the necessary dimensional thermal stability.

D. FABRICATION OF A GEOMEMBRANE

Producing a geomembrane involves associating fabrics and mixes to obtain a product which is :

- *in roll form,*
- *easy to handle,*
- *mechanically and rheologically to the performance levels that the designer requires,*
- *free of any defects such as a lack of material, irregular thickness, irregular length and width, poor impregnation or incorrect position of fabrics, pleats, incorrect alignment for the most importants.*

Production can be subdivided into several steps :

- *production of mix,*
- *preparation and unrolling of fabrics,*
- *impregnation and laying of fabrics by mix,*
- *cooling and finishing,*
- *rolling and cutting.*

At each stage of the process, the project is the object of :

- *quality control.*

1. MIX

1.1 Oxydized bitumen

Generally, 100/40 bitumen is used with filler (limestone) of which 92% will pass through a 30 microns sieve.

1.2 Modified bitumen

Bitumen-elastomer mixes are obtained from direct straight-run bitymen or, in some cases, from a mixture of oxydized bitumen and straight-run bitumen with polymer-styrene-butadiene-styrene. The chemical characteristics of the bitumen must be adapted to those of the elastomer to ensure mix stability.

The filler used is the same as for manufacturing an oxydized bitumen base mix.

1.3 Manufacture of mixes

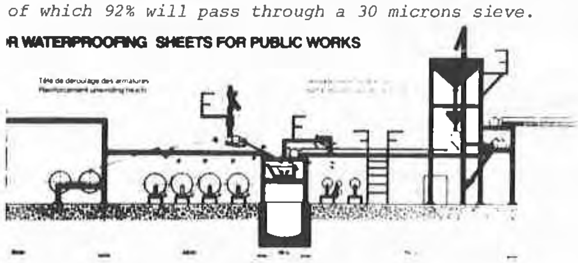
The mixes are manufactured in an installation controlled by a programmable logic controller, an industrial computer permanently controlling the flows of the different components.

The system guarantees at all times conformity of the mix with the theoretical formula, even if there is a malfunction during fabrication.

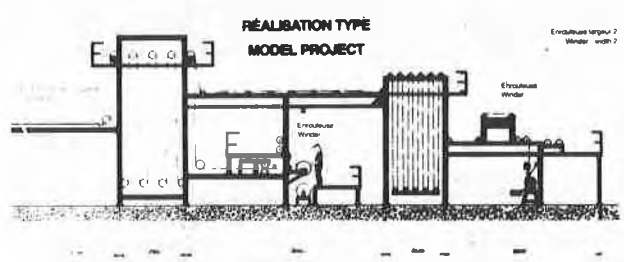
The mix is stored in one or several tanks, with capacities from one to four tons, in which stirring and thermal regulation devices maintain it in operating condition.

Drawn off the storage tank by pumping, the mix is directed to the coating tank, passing through a grinder which eliminates any solid particles in suspension.

The coating tank is equipped with an overflow so that the level is maintained constant in the tank regardless of any irregularities in production. The mix circuits in the coating tank are studied to avoid any dead zones in which an accumulation of mix could form and cause it to degrade.

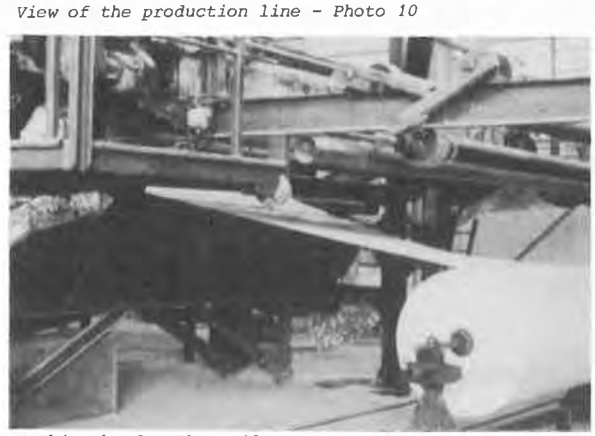


View of the production line - Photo 9



View of the production line - Photo 10

General view - Photo 11



Machine head - Photo 12

2. <u>MEMBRANE</u>

The association of the fabric and the mix is carried out by coating, preceded or not preceded by impregnation of the fabrics. The membrane is generally calibrated in association with the coating process.

Then, cooling, sintering, rolling and preservation of the membrane are carried out.

Preparation of fabrics

All the fabrics are supplied in rolls of 2 to 5 meters width and lengths of 100 to 800 meters.

Two rolls of each fabric must be set up simultaneously to allow rapid change over from the one to the other. The link between two successive lies of the same fabrics is obtained by overlaying.

The coating operation at the same time results in the forming of the membrane and the core-impregnation of the different fabrics. (Photo 11).

This requires a period of dwell in the coating tank, varying greatly as a function of the fabric. The circuit of the fabrics in the tanks therefore requires studying.

In some cases, the fabric can be reimpregnated (the same can apply to 2 armatures) in an impregnation tank before entering the coating tank. (Photo 12)

The mix used for preimpregnation is usually the same as that forming the body of the membrane, in order to avoid all the problems of incompatibility that appear automatically.

The choice of temperatures of the mix in the tank used for coating must enable the desired viscosity to be obtained to reach the right membrane thickness. Obviously, this temperature must be compatible with the resistance of the components to the effects of heat.

During coating, a substantial thermal shock is applied to the fabrics, liable to cause their expansion or shrinkage. These will be aggravated by any abrupt change of speed. Electronic protection devices provide the means of avoiding these variations to the greatest possible extent.

The thickness is controlled by fixed or rotating scrapers which must be set very accurately and maintain this accuracy regardless of the temperature variations (machine stoppage).

After coating, accelerated cooling of the membrane is performed using conventional means such as cooled cylinders, cold air blowers, water spraying. (Photo 13)

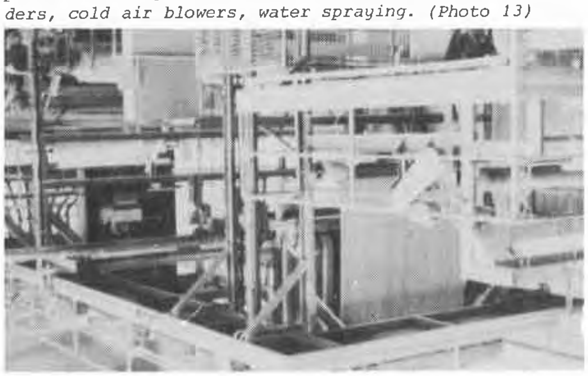


Photo 13 - Cooling festoons

During cooling, the membrane is given a finish consisting of sand or colored granules or plastic film (Photo 14).

Photo 14 - Surfacing unit

The membrane is then rolled on a mandrel for transport to the place of use. (Photos 15 and 16)

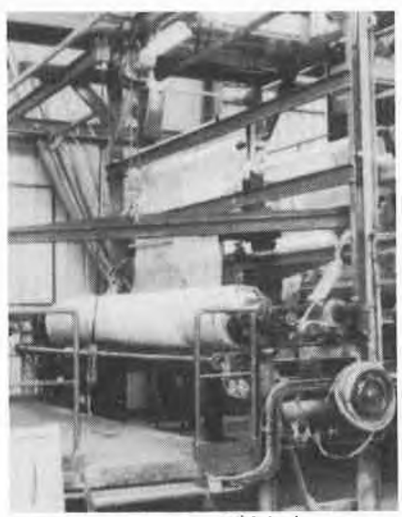


Photo 15 - Winder - width 4m.

Photo 16 - Winder - width 2 m.

The length of the rolls generally varies from 10m to 80m for widths of 2 to 4m. The weight can range between 200 and 1700 Kg. The rate of production is in the region of 600 to 800m per hour.

Tests are performed during production and relate to the characteristics of the menbrane i.e. :

- *raw materials, fabric characteristics, those of the bitumen and the polymer ;*

  and,

- *on the finished product, they relate to the appearance and characteristics of the product.*

E. CONCLUSION

From the above, in particular when we consider the rhéological, mechanical characteristics and the durability it is observed that bitumen geomembranes, specially those based on SBS-bitumen polymers and non-woven polyester and glass provide excellent work characteristics, both with respect to permanent tension and sollicitation even under the extreme conditions that are encountered on site.

The membrane designed in this way benefits from the properties of bitumens as far as workability and ruggedness are concerned, from those of fabrics as far as the résistance to permanent traction is concerned, even in severe ultraviolet, ozone and heat environments. it benefits from the properties of polymers as far as elasticity, fragility at low temperatures and resistance to heat are concerned.

These exceptional properties are obtained only at the cost of a careful choice of raw materials based on chemical, rheological and mechanical criteria, and an equally severe selection of the method of dispersion and the process of manufacture.

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Flexible Vinyl for Exposed Geomembrane Use—Is It Possible?

The usefulness of plasticized PVC as a buried geomembrane and its failings as an exposed geomembrane are reviewed. The effects of film thickness, UV light, ambient temperature, and microbiological species on flexible PVC is detailed. The existing and potential solutions to these problems are explored primarily in terms of plasticizer choice and plasticizer stabilization. A rationale is developed for the future use of plasticized PVC as an exposed geomembrane.

INTRODUCTION

Plasticized flexible PVC was one of the earliest and still remains one of the widest used geomembranes. It is particularly useful because of its excellent physical properties in an unsupported state, its resistance to acids, mild alkali and other chemicals, and its relative insensitivity to most aqueous environments. It is factory-fabricated easily by heat, dielectric sealing or solvent welding. Field seaming is readily accomplished with bodied solvents or the like.

It is available, unlaminated, in thicknesses up to 60 mils and is probably the most economical, field-proven geomembrane on the market today.

To date, plasticized PVC is only recommended for use when buried under a minimum 12" of earthen cover. For exposed berms and the like, CSPE and CPE are usually used; even though in the same installation, flexible vinyl may be used in the unexposed portion. Historically, flexible vinyl, when exposed directly to weather, has shown a progressive tendency to become brittle and ultimately crack, resulting in a failure of the lining system. The brittle failure is attributed to the loss of the flexibilizer (plasticizer) from the vinyl due to volatilization. Analysis of exposed vinyls vs. their unexposed controls does tend to support this hypothesis since significantly reduced levels of plasticizer are found in the exposed samples. Further support for the plasticizer volatility theory is found when one considers that rigid PVC has an outstanding history as a polymer system that withstands, exceptionally well, all types of weathering conditions. For example, rigid PVC is used extensively worldwide for house siding and window frames and is sold with 15-20-year warranties against fading and brittle failure. This suggests that the basic polymer, Polyvinyl Chloride, is reasonably resistant to the effects of the exterior environment.

However, one curious fact that sheds considerable doubt on the volatility theory is that PVC plasticized with certain plasticizers of lower volatility than others show brittle failures and higher percent plasticizer loss when exposed to real-time or accelerated weathering conditions. This suggests that phenomena other than plasticizer volatility may be at work causing plasticizer loss and ultimate brittle failure.

Because of the belief in the volatility theory that says that all plasticized PVCs will ultimately fail in the brittle mode because plasticizer is inevitably lost, researchers have not attempted the apparently fruitless task of developing plasticized vinyls for prolonged exterior use.

To get around this problem, special polyblends of PVC with EVA and CPE (replacing plasticizer), etc. have been developed for exterior use but do not concern us in this paper since they represent high-cost technology and significant polymer processing problems.

EFFECTS OF THE EXTERIOR ENVIRONMENT ON GEOMEMBRANES

The geomembrane that is uncovered is exposed to the following influences:

1. UV light of wavelength 300-350nm from the sun.
2. Exposure to water through rain or the fluid that the liner contains.
3. Temperature extremes between $-20^{o}C$ and $+100^{o}C$.
4. Microorganism exposure from the air or more likely from the soil on which the geomembrane is placed.

UV LIGHT

We know that UV light can catalize, in the presence of oxygen, many types of free radical reactions. One must consider that both plasticizer and polymer are potentially subject to degradation in an environment of free radicals.

WATER

Typical PVC plasticizers have very low ($<0.1\%$) solubility in water and therefore one would not consider they will be substantially leached, if they remain chemically unchanged, by the normal rainfall. However, in the case of constantly running water (a riverbed), the solubility of the plasticizer in water is never satisfied because the water is constantly moving.

Therefore, even with low water solubility, leaching can be expected in such a constantly-moving water situation.

TEMPERATURE

While ambient temperatures may only be moderate, the membrane itself, depending upon its color and its physical chemistry, may reach extremely high temperatures. For example, a black exposed membrane of EPDM has been measured with a surface temperature of 180°F on a hot summer day. Such extreme temperatures no doubt result in volatile loss of plasticizer and, in addition, can, in the presence of oxygen, cause free radical reactions to occur.

On the other extreme at extremely low temperatures, vinyl tends to become brittle - varying in brittleness temperature with the chemistry of the plasticizer used. However, the brittleness temperature with any given plasticizer system is directly proportional to the amount of plasticizer present. The higher the plasticizer level, the lower the brittleness temperature. Bearing this in mind, as the membrane loses plasticizer from other environmental factors, temperature, UV, etc., the brittleness temperature is adversely affected.

MICROBIOLOGICAL EFFECTS

Plasticizers can be consumed by a variety of microbiological organisms and converted into non-plasticizing moieties. This action, in effect, removes plasticizer from membrane causing it to stiffen. Certain plasticizers, e.g., epoxidized soya oils, are much more susceptible to microorganism attack than others.

THE INFLUENCING CHEMICAL & PHYSICAL FACTORS OF THE MEMBRANE

1. Thickness - The thickness of the geomembrane is extremely significant in plasticized vinyls as regards the useful lifetime of the product.

The major environmental effects take place on the surface;and the membrane, if it could be analyzed at the instant before it equilibriates, one finds lower levels of plasticizer at the exposed surface than at the unexposed surface. Since there is a constant driving force to achieve equilibrium, the higher plasticized portion acts as a well for leaner surface. Obviously, the deeper the well, i.e., the thicker the membrane, the longer the useful life of the product.

2. Color - As suggested earlier, some colors allow the geomembrane to absorb radiation and become very hot while others reflect radiation and do not allow the membrane to reach high temperatures

For the most part, in this comparison we are dealing with the difference between carbon black and TiO_2. Carbon black, while it does allow the temperature of the membrane to become high, acts as both a UV absorber and an antioxidant and, as such, is unique. TiO_2, while it does absorb UV light, it, in a secondary reaction, may generate free radicals which have a degrading effect.

PLASTICIZER CHOICE

As a general rule -

- Those plasticizers that offer the best low temperature properties are the most volatile.
- Those plasticizers that are most efficient (require the least to attain a given elongation) are the most volatile.
- Those plasticizers that have the best resistance to microbiological attack have the poorest resistance to UV-light degradation.

With all the potential problems described, what chances are there to formulate vinyl sheeting that will be suitable for exposed geomembranes? We believe the technology is here today to accomplish this. The experimental work must be carried out, followed by confirming exterior exposure studies.

Let's examine our rationale.

ROOFING EXPERIENCE

We know that laminated, exposed PVC roofing has been reported in satisfactory use for greater than 10 years. Such laminates use a very thick upper ply (30-40 mils) to take advantage of the thickness principle described earlier, a heavy-duty polyester scrim supporting layer to provide dimensional stability and a lighter weight backing (15-20 mils).

EFFECT OF THICKNESS

Three formulations, all quite similar except for minor changes in the stabilizer system, were weathered for 4500 hours in a Q-UV apparatus. Ths retained physicals measured as a percent of the original values were as follows:

TABLE I

Formula No.	Thickness	Retained 100% Modulus	Retained Elongation
A	7 mils	226%	33%
B	10 mils	206%	79%
C	14 mils	161%	99%

Reversing the thickness order of the same three formulations and weathering now in the Atlas Weather-Ometer:

TABLE II

Formula No.	Thickness	Retained 100% Modulus	Retained Elongation
A	13 mils	132%	98%
B	10 mils	129%	91%
C	8 mils	178%	72%

Clearly, the retention of original physicals of these specimens suggests that key physical property retention is closely controlled by the thickness of the specimen weathered.

UV LIGHT EXPOSURE

We believe that far and away, UV light from the sun plays the most profound roll in the degradation of plasticized PVC. Williams & Gerrard (J. Poly Sci 21;1491-1504 (1983))working with phthalate plasticized PVC showed significant changes in the chemistry of both the plasticizer and the polymer after short-term exposure to high intensity UV light.

It is suggested by us that a primary photochemical event occurs - through photon absorption - within the plasticized PVC matrix that is followed by a series of secondary reactions. These secondary reactions result in the formation of a variety of decomposition products of both the plasticizer and the base polymer PVC. Alcohols, anhydrides, peracids, peresters, carbonyl compounds, HCL, peroxides, and various polyenes have all been found by us in plasticized PVCs subjected to UV light using ATR-IR analysis techniques.

It is interesting to note that often conditions are not achieved during the UV light exposure experiments to get substantial plasticizer loss from temperature alone;

yet, embrittlement and apparent plasticizer loss occurs. Under similar exposure conditions, unplasticized PVC is substantially unaffected. These facts suggest that the initiating reactions start with the plasticizer which produces by-products that have a degrading effect on the polymer. Confirming this are our own studies which show that when efforts are made to stabilize the plasticizer against the effects of UV light, the specimens show less deterioration after exposure.

EXTREME TEMPERATURE EFFECTS

We are able to formulate PVC films that meet the low temperature flexibility requirements of an exposed membrane. Retaining that property after years of exposure to summer heat, high concentrations of UV light and other environmental effects is the technical challenge. Table III shows some interesting effects;

TABLE III
T_f (ASTM D1043)

Changes as a Result of Heat and UV Aging
(70-mil specimens)

	Plasticizer A	Plasticizer B
T_f original	-34°C	-28°C
T_f 28 days oven @ 110°C	+4°C	-6°C
ΔT_f " " "	+38°C	+22°C
T_f 12,000 hrs. Atlas Weatherometer	-23°C	-9°C
ΔT_f " "	+11°C	+19°C

Levels of each plasticizer adjusted to achieve same level of hardness for both specimens.

The table certainly suggests that a more volatile plasticizer (Plasticizer A) shows a substantial change in T_f when heat aged in the dark (38°C loss) when compared to the less volatile Plasticizer B (22°C loss). The situation reverses itself when the primary effect on the film is UV (Atlas Weatherometer) light, suggesting that the chemistry of the plasticizer, rather than its volatility, is significant in determining the weatherability of a plasticized PVC film.

We suggest that plasticizers and plasticizer mixtures are available today and with still better ones coming in the near future that provide the desirable combination of low volatility, satisfactory low temperature flexibility, with good resistance to the degrading effects of UV light.

PREVENTION OF MICROBIOLOGICAL ATTACK

Plasticized PVC is subject to microbiological attack which manifests itself by discoloration, staining and stiffening. The basic polymer - PVC - is quite resistant to bacteria and fungal attack. Therefore, the other components of the formulation are obviously the sensitive species and must be carefully chosen. Selection of components that are strongly resistant to microbiological attack limit ones ability to achieve several of the other desirable characteristics of a geomembrane. For example, epoxidized oils, which may be an essential part of the thermal and UV-light stabilization system of a geomembrane formulation, are unfortunately very sensitive to microbiological effects, as are some of the most UV-resistant plasticizers.

In recent years, new improved biocides and fungicides have been developed that do not have the negative secondary effects of some of the traditional types. These allow compounding flexibility without fear of biological failure. Thus, one must select a combination of microbiological resistant ingredients combined with suitable biocides and fungicides for longevity of a geomembrane.

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Comprehensive Quality Control in the Geomembrane Industry

This paper discusses the key stages of quality control in the manufacture and installation of geomembranes, and explains how they relate to each other in the final application. The use of geomembranes for containment applications is not new, yet many end users and their consultants are unfamiliar with the decision making process for liner selection and specification. A geomembrane should meet quality control standards that are applied:

1. before the base material is accepted from a supplier;
2. during manufacturing; and
3. during and after field installation.

The best safeguard against a lining system failure is for the end user or specifier to insist on and ensure strict quality control by the system supplier. It has been the unfortunate experience of this author that this is often not the case. To understand the significance of quality standards, one must first grasp the sequence to which quality control standards are applied.

MULTIPLE CHOICES

Despite the fact that synthetic membranes have been in use for more than 40 years, there is still a lack of good data and understanding from which proper lining system choices can be made. Some of the reasons for this are:

1. There are at present seven basic synthetic materials used for geomembranes, as well as several others that are used to a lesser degree. This presents the specifier with a great variety of geomembranes from which to choose, all with different properties and characteristics; thus the specifier must have sufficient and necessary data to make the best choice of material for his application.

2. Geomembranes are not pure materials. For each synthetic type, there are literally hundreds of combinations available. It is possible to choose a geomembrane that, theoretically, should meet the needs of an application, but which, in fact, will fail because specifications or quality standards were not exacting enough.

3. All geomembrane base materials have other materials added to them during the manufacturing stage. These additives can have a major impact on the success or failure of a liner, depending on what is (or is not) added and to what degree.

4. The finished product that reaches the field has already, in most cases, been through several hands. The end user must be familiar with all the stages of the supply chain in the geomembrane industry, from raw material supplier to field installer, if he is to ensure that the needs of his application are met.

Figure 1 is a flow chart that shows the various contributions made to the process of producing and installing geomembranes. The boxed areas are those key stages where quality control standards must be established and met.

GEOMEMBRANE MATERIALS

There are many materials and combinations of materials which make up the geomembrane types. The most common are:

- Butyl Rubber - BR
- Chlorinated Polyethylene - CPE
- Chlorosulfonated Polyethylene - CSPE
- Ethylene Propylene Diene Monomer - EPDM
- Polychloroprene - PC
- Polyethylene (Low Density and High Density) - LDPE and HDPE
- Polyvinylchloride - PVC

In addition to the above, there are a few other base materials that have been used to a lesser degree, as well as several blends and alloys. A few examples are:

- Epichlorohydrin Rubber
- HDPE with EPDM
- Ethylvinyl Acetate with PVC

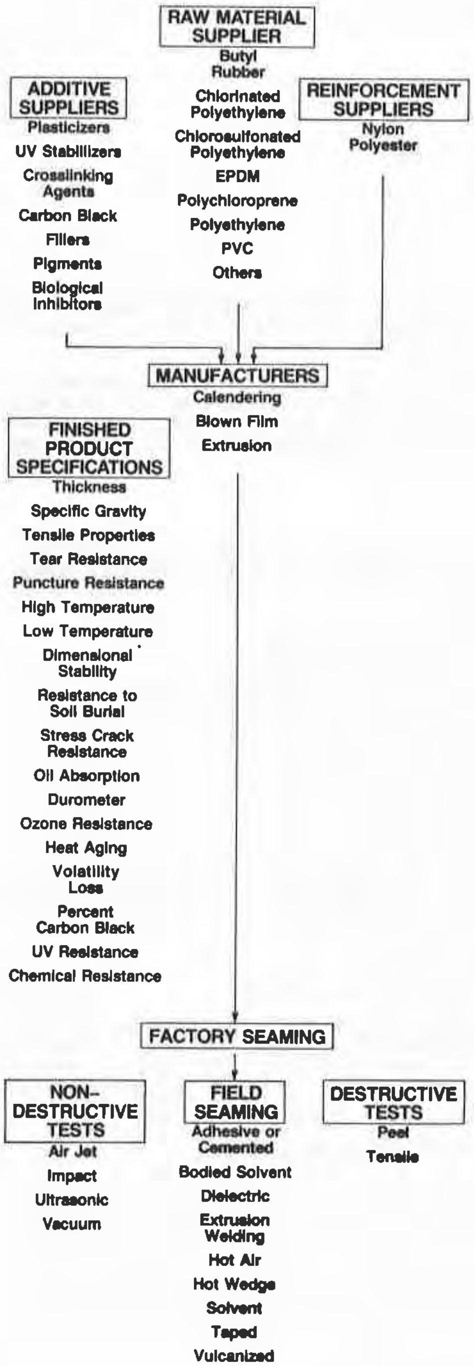
The base material as supplied to the liner manufacturer should already have met certain standards of quality control; the raw material manufacturer, however, has no control over the further addition of additives or processing of the material into a geomembrane.

GEOMEMBRANE MANUFACTURING AND ADDITIVES

When a specifier has made his material selection, and is satisfied with the appropriateness of its quality control program, he must then turn his attention to geomembrane manufacturing in general and the inclusion of additives in particular. Scrutiny of additives is a step all too often overlooked during geomembrane selection and specification.

The three most common manufacturing processes for geomembranes are:

- Blown Film Extrusion,
- Calendering,
- Extrusion.



Stages in the Process of Producing and Installing Geomembranes
FIGURE 1

During manufacturing a complete quality control monitoring system should be in effect for geomembrane specifications, which are listed in Figure 1 to the left of manufacturing. It is thoroughly appropriate that an end user or his agent request documentation that these standards have been met.

As stated earlier, none of the base materials are used to produce pure liners. All geomembranes contain one or more additives that are introduced before or during the manufacturing stage. Thus it becomes important that the end user know what these additives are, what their properties are, in what percent they are used, and why they are employed.

Additives are mixed with the base raw material before production, and they are thus an important component of the final product. Some examples of additive types and their purposes are:

1. Plasticizers - Generally a liquid material that is added to make a base material more workable and the final product more flexible (e.g., PVC).

2. UV Stabilizers - Introduced in some products to improve their ability to withstand UV radiation. Carbon black serves as a UV stabilizer.

3. Crosslinking Agents - Promotes the formation of chemical bonds between polymeric chains. Generally used with rubber (thermoset) materials. Crosslinking changes the polymeric structure and thus the properties of a base material.

4. Fillers - Added to give body and reduce costs. Fillers are also added to improve the surface smoothness of the geomembrane and its dimensional stability. Calcium carbonate is one of the most commonly used fillers.

5. Pigments - Added for coloring purposes, but if properly selected can also act as UV stabilizers.

6. Biological Inhibitors - Added to materials which are susceptible to attack by micro-organisms. This is particularly true for geomembranes containing plasticizers which serve as food to some species of micro-organisms; correct selection of plasticizers by the manufacturer can reduce this problem.

7. Scrim Reinforcement - Added to a geomembrane after the mixing process, but before the final take-off. Reinforcement is often referred to as scrim, and the two fabrics commonly used are nylon and polyester. The choice of either is based on the chemical environment in which they will see service; an acidic solution would require polyester, and an alkaline one nylon. The purpose of scrim reinforcement is simply to increase strength, specifically in the areas of tear and puncture resistance. Reinforcement is used with those geomembranes that have low strength properties or where an application demands strength greater than the unreinforced geomembrane could provide. Hypalon, for example, is often reinforced. From a quality control point of view, the scrim should be checked against its manufacturer's specifications, so that it does in fact provide the desired strength properties in the geomembrane.

FACTORY SEAMING

With many geomembranes, an additional step of factory seaming of small panels of material takes place before the liner is shipped to the field. Although this process is loosely termed "factory" seaming and is sometimes performed by the manufacturer, this operation is more frequently performed by a separate firm.

The purpose of factory seaming is to increase the size of a liner section for geomembranes whose processing results in narrow width panels, and reduce field labor costs and potential quality problems associated with unknown and uncontrollable weather and site conditions. Additionally, seaming methods not easily accomplished in the field can be performed in a factory environment. Dielectric seams, for example, are typically made in a "factory." The process uses a high frequency dielectric current to melt the surfaces of two pieces of material so that they can then be bonded together under pressure. The use of high frequency current in the field is a difficult task as compared to making adhesive or solvent seams. Quality control of factory seams should be performed according to the same test procedures that are used for field seams. These are described below.

FIELD INSTALLATION

The final stage where quality control is important, and most critical, and yet the one that is most difficult to manage is the field installation itself. Even if all decisions made up to this point are correct, even when a perfect geomembrane is manufactured and delivered, a system can fail because of poor field installation. There are a number of reasons for this, as follows.

1. Any method of seaming in the field is susceptible to error. Factors such as uneven terrain, wind, weather and temperature affect results.

2. The capability needed to check seams and workmanship may not always be present. Laboratory facilities and the personnel to man them are not always readily available.

3. The quality of workmanship in field operations, all claims to the contrary, is always subject to human error.

4. Sources for geomembrane puncture, from the installation team to other contractors to the conditions of the site, are always present.

5. The only test method for a quantitative measurement of field seam strength (as opposed to static mode watertightness) is a destructive one, which examines only a small portion of the total seam length.

The method of field seaming is critical and varies with different materials. Each joining method has its own set of quality control measures. The more common techniques for field seaming are:

1. Adhesive seams - These are joined with a chemical adhesive that glues together two membrane surfaces. The chemical adhesive is an additional element to the seam system and, thus, must meet all the criteria of the application such as chemical resistance, temperature extremes and so on. Generally when using adhesives, a two-component system is employed, although this requires care in mixing. When the adhesive is applied, the residence time before bonding is critical, as is pressure and its timing. These are variables that may be difficult to control in a field situation.

2. Extrusion welding - For these seams, a bond is obtained between two sheets by extruding a molten ribbon of the parent material between overlapped liner pieces, followed by applied pressure. The pieces are heated before the extrudate is placed, a process that requires care and skill.

3. Hot air - High temperature air or gas is applied to two surfaces until they melt, at which time pressure is applied to create a homogeneous bond between the pieces of geomembrane. Timing and pressure must be carefully controlled to avoid thermally induced damage.

4. Bodied solvent seams - For this seam, lining material is dissolved in a solvent, which then softens and bonds the liner surfaces together. This method is essentially an adhesive system made up from the parent material. Problems with application timing and pressure can occur.

5. Solvent seams - Solvents are used to soften the surfaces to be bonded; generally, pressure is then applied to the seam area. Under field conditions, the application of the solvent, and control over timing and pressure, can be a difficult task.

Field seams are evaluated for their integrity by both non-destructive and destructive test methods. Non-destructive tests, although important and valuable, indicate only that a seam is watertight; they do not provide data on the strength of the bond. Thus, while a non-destructive test may serve to identify seams in need of immediate repair, one cannot rely solely on them for complete quality control. The most common non-destructive tests are:

1. Air jet testing
2. Impact testing
3. Ultrasonic testing
4. Vacuum box testing

Destructive test methods, and the peel test in particular, do provide substantial data for actual bond strength. The two commonly used destructive tests are:

1. Tensile and
2. Peel test.

Both tests are performed on all geomembrane seams in the same way. Samples should be one inch in width and with two inches of grip separation.

The peel test is the best evaluative tool for assessing seam strength, because it applies force against the bond in a severe manner. A good bond will exhibit elongation and ductility to the naked eye; conversely a poor bond will be readily evident. If the bond is as strong as the liner, the liner itself will tear away from the bond, before the bond breaks. This is called Film Tear Bond (FTB). This is also referred to as 100 percent visual.

The tensile test for seam strength, on the other hand, is not a conclusive one for bond strength, because the change in cross-sectional area through the weld zone concentrates stresses at the ends of the weld. Thus, tensile testing may yield "good numbers," when in reality the bond may be easily peeled apart.

GEOMEMBRANE SELECTION AND QUALITY

The destructive peel test described above, the most meaningful in determining seam quality of the installed geomembrane, is only the final test in a thorough quality control program. Comprehensive investigations should have been performed before this last step, in order to assure adequate performance of the geomembrane in the particular application. A liner may fail for many reasons other than faulty seams. Other guidelines for assuring quality control and thus good performance of the geomembrane are:

1. Thickness - The most important function of thickness is the connection of thickness with a material's strength. Thickness exhibits a linear relationship to tear and puncture resistance, for a given material. Tear and puncture of the geomembrane are potential causes of liner failure. There are a number of standardized tests that will determine a material's strength in relation to its thickness. Reinforced geomembranes, however, derive their strength in tear from the reinforcement, and thus their relationship to thickness is not a proportional one. For this type of geomembrane, the choice of scrim reinforcement will determine minimum values for puncture and tear resistance.

2. Temperature extremes - Since almost all geomembranes are used out of doors, the temperatures encountered in a climate may be a cause of liner failure. The process use which an impoundment serves will also have its own operating temperatures. For any application, the geomembrane being considered should be tested for worst-case high and low temperatures.

3. Biological attack - Some materials, if not properly formulated, are susceptible to microbial attack; this is particularly true for materials containing organic plasticizers which are actually consumed by the microbes. When the plasticizer, used to soften the material, is gone the geomembrane will harden, crack and fail. Proper selection of plasticizers and/or soil sterilization before installation of the geomembrane can prevent this problem.

4. Weatherability - The ability of a given geomembrane to withstand anticipated weather conditions should be investigated. The material should be assessed for its UV stability, ozone resistance, thermal shrinkage and brittleness temperature.

5. Chemical resistance - Because the geomembrane is a barrier between chemicals and soil, it must be compatible with the chemicals it will contain. Chemical compatibility is probably one of the most controversial issues of the industry and a difficult property to determine. There are many unknowns when attempting to determine the chemical compatibility of a material, including:

a. The percentage of a given chemical. Even though a particular material may not be considered resistant to a certain chemical, it may function well if the chemical solution is well diluted.

b. In many applications, the ability to predict future chemical make-up is impossible.

c. There are instances when chemical reactions create new chemicals that were not anticipated during an original analysis of a material. Such reactions might also be exothermic ones, creating additional problems.

6. Seams - Before a material is specified, the seaming method must be known. If the method introduces another material into the bond, as is the case, for example, with adhesive seaming, the new material must also be tested for its ability to withstand the same conditions as the geomembrane itself.

STANDARDS

Until recently the geomembrane industry has been handicapped by a lack of clear standards. New standards are, however, being developed, and they at least begin to give a specifier objective, comparative information that was not previously or easily available. The most recent standard is the newly published Standard Number 54 of the National Sanitation Foundation. Other standards are being developed at this time by ASTM, for seam quality (D-18), and for installation methods (D-34). USBR and NSF are also conducting long-term tests on the chemical compatibility of seams and the chemical compatibility of geomembrane materials.

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Quantifying the Geomembranes Market in North America—Past, Present and Future

While the market for geomembranes has not mushroomed as some had predicted it would, it has remained steady during the past four years. Some indicators point to stronger market sales in 1984, but the industry is taking a wait-and-see attitude.

During the last four years the market for geomembranes in the United States has remained relatively flat. Modest growth occurred in 1982 and 1983 and steady changes have occurred within the market. However, the surging growth forecasted during the late 70's has failed to materialize.

The market has hovered between 122 to 155 million square feet of installed liners annually because of the regulatory and economic climate during the last four years. The paradox of tremendous mounting public interest in the waste and water management crisis versus little growth in one of the major proven products to deal with that crisis needs to be as fully understood as possible in order to chart the industry's future.

The geomembrane industry has been developing in this country for more than a quarter of a century, well before the term geomembrane itself was coined. Firms such as Gundle, Palco, and Staff pioneered the development of pond liners from the late 40's on, and other firms like Watersavers, Carlisle, and Burke Industries have been instrumental in building a sound base of engineering and product design for the industry. Often still referred to as pond lining, the geomembrane industry has been providing impermeable membrane liners and covers for agricultural applications, municipal uses, industrial effluents, and hazardous waste containment. As the industry developed, innovations in usages and materials were steadily introduced. Currently, three major classes of materials are seeing use in the market.

The material classes are calendered hypalon sheet, calendared PVC, and a newer emerging group of high performance materials which include HDPE (high density polythelyene), CPE (chlorinated polyethelyene), and vinyl coated fabrics such as Shelter-Rite's XR5. The market shares of these three classes of geomembranes are given in Table 1. It should be noted that 1980 was not a "typical" year for the market, because of the large amount of CPE material used in the 12 million square foot Mt. Elbert Forebay impoundment. The underlying usage rate for specialized membranes was then running at under 10 percent, but this one impoundment brought the specialized membranes' share up to nearly 20 percent. Hypalon's share has declined over the last four years, commanding a steady 50 million square feet each year. PVC has been maintaining just under 50 percent in the market share. Consumption has increased from about 50 million square feet in 1980 to around 75 million square feet in 1983. The specialty, or high performance materials have been picking up market share at the expense of Hypalon. Market share and consumption have nearly doubled within the period. Starting with an underlying 10 percent share and 10 million square feet in 1980 and rising to about 20 percent share and nearly 30 million square feet in 1983, this group of materials has established itself as a strong new force in the market.

Describing the specific properties of the materials in these three classes cannot be justly covered with this article. Instead, the perceptions of the qualities of each membrane group, as generally understood in the market, should be dealt with. PVC membranes are the least expensive of the three materials, currently running in the neighborhood of $.25 per square foot for 30 mil material with lower prices on 20 mil goods. PVC membranes have a long history and have been shown to work well when buried in so-called less demanding applications. Hypalon sheet has an equally well established track record as PVC. Hypalon is looked at as a more versatile material then PVC, suitable for most demanding applications. However, there are a few chemicals that attack hypalon, so it, like the other two materials, is not a universal material for containing any and every effluent. Hypalon in a 30 mil thickness can sell for between $.50 and $.70 per square foot. The specialized materials such as HDPE, CPE, and vinyl coated polyster are the most expensive of the three material types. Although 30 mil sheets of HDPE, for example command $.35-$.45 per square foot, prices for 100 mil HDPE sheet can range from $.90 to $1.25 per square foot. Currently, 80 mil is the most commonly used thickness for HDPE membranes. These specialized membranes are developing a growing reputation for handling high performance industrial and hazardous applications.

Table I U.S. Geomembrane Market 1980–1983
(In millions of sq. ft.)

	1980		1981		1982		1983		(forecast) 1984	
	sq. ft.	%	sq. ft.	%	sq. ft.	%	sq. ft.	%	sq. ft.	%
Hypalon	50	41	50	41	50	36	50	32	55	30
PVC	50	41	57	47	65	47	75	48	90	49
Specialized	22	18	15	12	22	16	30	20	40	21
Total	122		122		137		155		185	

In looking at geomembranes from a market-oriented basis of general reputation and price, the choices of membranes provide end-users with a complete range of cost/benefit choices to consider. Lower-, medium-, and higer-priced alternatives are available and can be matched with the needs and budget of a specific impoundment design. The key selection criteria should be design, but budget limitations can also effect membrane selection, particularly where a variety of materials appear to be suitable for a given job. The most significant question is, given the diversity of products, prices, and cost/benefits available, why hasn't the use of geomembranes grown as concern over waste and water management mushroomed?

The fundamental point to consider in answering this question is that the uses of geomembranes that will propel market growth do not generate any production efficiencies or income to the user. The two applications that should be generating significant market growth are the industrial and hazardous waste areas. Landfill liners and caps, using PVC materials primarily, were responsible for the 12 to 13 percent growth during 1982 and 1983. From a strictly economic standpoint, money spent on waste management is literally money down the drain. Two types of incentives are needed to encourage improved effluent management practices. The first is good corporate citizenship, and the second is government regulations.

Contrary to the popular media image, the vast majority of American Industry, particularly Fortune 1000 firms, is concerned about the impact of both general industrial and hazardous waste on the environment. Both management and production employees usually live in the same area and drink the same water. They are concerned about their community, their relations with it, and their impact on it. This can be seen in the increase in industrial and hazardous impoundments during the last few years, taking up the slack resulting from depressed economic conditions in the mining sector.

Looking at the second major incentive for market growth, government regulations, the picture has not been as good over the last few years. Again, the development and implementation of the Resource Conservation and Recovery Act (RCRA) can not be covered within the scope of this article. Large amounts of both praise and blame can be credited to any number of involved parties, in government and industry. All will admit that we have experienced a period of delays and confusion during the early 80's. Considering only the impact of regulations so far on the geomembrane market, the period of indecision recently experienced has had a negative effect on market growth.

The geomembrane industry has needed three things for government regulation to have a positive impact on the market: recognition, standards, and enforcement. To date, recognition of geomembranes as a major tool in waste and water management has been fairly well achieved. In the area of standards, the current EPA recommendations have provided significant progress, and the completion of the National Sanitation Foundation (NSF) standard for geomembranes has even more promise. However, these documents were not available during most of the early 80's, and more work needs to be done to apply them directly to the needs of the marketplace. Enforcement has not yet been terribly effective, but recent changes at EPA may bring strong efforts in this area.

Of these three major needs of the marketplace, only one and a half have been met. This has led to confusion and hesitation among probable geomembrane users. Plans for new impoundments have been put on the shelf, awaiting the newest regulations on design and licensing. Waste managers have had to face situations where they haven't known whether the design they specified will be legal by the time it's finally installed.

The market for geomembranes has shown only slight growth compared to true market potential during the last four years. This has been because of the low overall availability of corporate funds for new impoundments and the regulatory "never never land" climate of the early 80's. However, the flat market picture from 1980 to 1981 results from the completion of one major project in 1980 that was not made up in 1981. The only major large-scale project completed in the U.S. between 1980 and 1983 was the Bureau of Reclamation's Mt. Elbert Forebay, installed during 1980. This one project used 290 acres of 45 mil CPE sheet, accounting for eleven percent of the market that year at 12,325,000 square feet. This single project also caused an abnormal shift in material market shares that year. Since the completion of the Mt. Elbert Forebay, no jobs of equivalent size have been installed, leaving the total market at 122 million square feet level in 1981. Low corporate profits during the recession in 1981 and 1982 limited geomembrane market growth, when nearly every departmental budget in major corporations was cut to the bone. To add insult to injury, the mining industry, particularly uranium, was one of the hardest hit by the recession. Long a major source of geomembrane contract during the 70's, work in this area slowed to a near standstill.

Given the overall disappointments of the last four years, the general attitude in the geomembrane industry combines an entrepreneur's innate optimism with the "I won't really believe it till I see it" approach of someone who has waited hoepfully for a ship that never comes in. The combination of improved economic conditions plus EPA's January 1983 issuance of it current guidelines on liner specification have brought a long awaited upturn in the market.

Membrane manufacturers, fabricators, and installers have all felt a strong increase in bidding and sales during the second and third quarters of 1983. Many large projects are being bid for 1984, ranging from 5 to 7 million square feet up to a 20 million square foot project. These indications point towards possible growth of 20 percent in 1984, reaching 185 million square feet of geomembrane installations. It appears that the corner could be turned by year end so that the market will once again be on a solid growth track. For those who kept up their committment to the geomembrane concept during the disappointing years of the early 80's, it's still too soon to tell whether renewed growth is here to stay. It's been a long wait for many, so sustained growth couldn't come too soon.

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General Concepts and Marketing Trends of Geotextiles Used to Protect Geomembranes

The geotextile and geomembrane markets have grown up independent of each other, but they are now being used in combination to provide the engineer with a safer and stronger liquid and solid waste containment system. Thick, needle-punched nonwoven geotextiles are being used to protect geomembranes from puncture and gas pressures. In 1984, the geomembrane market is projected to grow by 12% to 325 million square feet, while the geotextile market for protection of geomembranes will grow by 20% to 15 to 20 million square feet. Key technical questions relating to the continued use of geotextile/geomembrane systems revolve around the chemical resistance, gas transmission properties, and long-term performance results.

INTRODUCTION

INTRODUCTION

A marriage is occurring today between flexible membrane linings (geomembranes) and thick, needle-punched nonwoven felts (geotextiles). Geomembranes have been used as an impervious barrier for containment of liquids in ponds for over twenty years. By 1981, the area covered by geomembranes on engineered containment structures had surpassed the area coverage by local clays and bentonite. And this market for geomembranes continues to grow by 12% to 14% annually. It is currently being enlarged by the use of flexible linings in solid waste landfills. But, usage and growth have also made us aware of the frailities of geomembranes. They are often thin and unyielding to pressures. Also, they trap pockets of subgrade gas which cause them to lift, float, and eventually rupture. Geotextiles (porous, fibrous mats) are being employed in combination with geomembranes to provide puncture protection, gas relief, and abrasion resistance. Thick, needle-punched fabrics were first used in Europe (1) in the early 1970's and have now found wide acceptance in the U. S. market with geomembranes. In 1983, over 15 million square feet of heavyweight geotextiles were installed with geomembranes — both below and, sometimes, above the lining. They have also been installed as an intermediate layer between two membranes in order to separate them and act as a conduit for later transmission of fluids that may pass through the upper geomembrane structure. Projects as large as 70 acres have employed geotextiles in combination with geomembranes (2). This unique combination provides the engineer/owner with a superior installation.

1. High degree of impermeability (geomembrane).
2. Puncture protection (3) — from earthen particles below, and foot or vehicular traffic above.
3. Gas relief from beneath the geomembrane — by in-plane transmissivity within the compressed geotextile layer (4).
4. Clean environment for field seaming of the membrane lining panels — providing quality assurance of the seam bonds.

GEOMEMBRANES

The containment market can be readily segregated into two distinct groups:

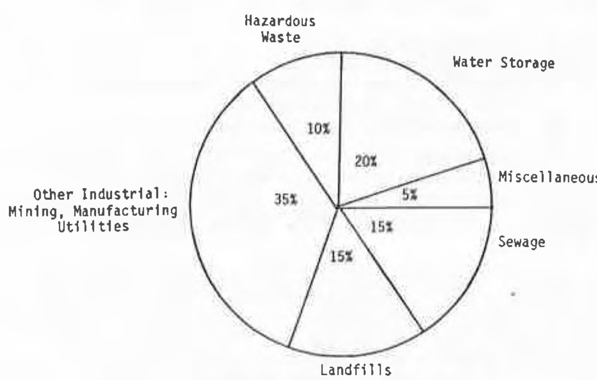
1. Natural — a. local clays
   b. bentonite
2. Synthetic — a. geomembranes

The natural structures have been in common use for thousands of years and generally quite economic to install. As our requirements for impermeability and chemical resistance have risen, particularly with toxic waste impoundments, the market has moved toward the synthetic (geomembrane) materials. The 1984 lining materials market is projected to top 325 million square feet with geomembranes capturing nearly 60%.

1.	Local Clays	45MM ft^2
2.	Bentonite	95MM ft^2
3.	Geomembranes	185MM ft^2
		325MM ft^2

This market is growing at 12% to 14% per annum.

The end use markets where geomembranes are employed are shown below.



The landfill segment is projected to experience the highest growth rate in the next five years. There are no clear geographic trends — although states with sandy soils will be the best markets, e.g., Florida, Texas, South Carolina, Arizona, New Mexico, and California.

The type of geomembrane selected for use appears to be shifting from the thin, low-engineered systems (e.g., PVC) toward the thicker, more durable products (e.g., HDPE). But, geotextiles are providing a new dimension here as they allow the use of thinner linings while providing puncture and gas relief protection.

GEOTEXTILES

The geotextile market is currently in excess of one billion square feet, but it is comprised of a wide range of fiber polymers, fabric properties, fabric constructions (i.e., woven or nonwoven) and thicknesses. The preferred products for use with geomembranes — because of availability, cost, and physical properties — are thick, needle-punched, polypropylene, nonwoven fabrics. Typical fabric physical properties (5) used with linings are as follows:

PROPERTY	VALUE
Thickness (mils), ASTM D-1777	110-150
Grab Strength (lbs), ASTM D-1682	260-320
Elongation (%), ASTM D-1682	90-150
Burst Strength (psi), ASTM D-751	400-500
Equivalent Opening Size (EOS), U. S. Standard Sieve	80-100
pH Resistance	1-13

The 1984 market for geotextiles used as an underlining (or in combination with a geomembrane) is projected to be 15 to 20 million square feet. Because the combination of geotextile and geomembrane is still a relatively small portion of the overall lining market (i.e., 6%), it will evolve and grow at an annual rate in excess of 20% for the next three to five years. Customers will select geotextiles on the basis of need and compatibility with due concern for the following:

1. technical experience/support,
2. site condition (e.g., rocky, soft),
3. gas/moisture permeability of soil,
4. price versus sand, and
5. availability.

Because the use of geotextiles is relatively new in structures with heavy loading (e.g., deep ponds), there will be a continuing need for technical information on a variety of subjects:

1. lifespan of geotextiles,
2. chemical resistance,
3. compression and permeability properties,
4. installation procedures, and
5. case histories of various uses.

It is expected that regulatory agencies, such as the EPA and State Departments of Natural Resources, will continue to experiment with geotextile/geomembrane systems as a means of providing safer, longer lasting containments.

Geotextiles are also commonly used as a drain wrap for washed stone in subdrain systems. These are quite common on pond sites as both water table control systems or as collection devices for liner leakage. The geotextile provides a filtration network around the stone to allow passage of water while stopping the movement of soil particles into the drainage network.

CONCLUSIONS

Geotextile fabrics of thick, needle-punched, nonwoven construction will increasingly be used in combination with geomembranes to provide:

- puncture protection from soil conditions or overlining traffic,
- gas relief through lateral transmissivity, and
- subdrain filtration efficiency.

Both geotextile and geomembrane markets are projected to grow at 20% and 12%, respectively, over the next three- to five-year period.

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Questioning the Twenty-Year Warranty

In any commercial or consumer sale transaction, the warranty of product quality or performance is one of the key items of interest for the purchaser, and in commercial transactions especially, it is usually one of the key negotiating items in the sale. The geomembrane industry is no exception.

This paper discusses warranties generally and warranties particularly in the synthetic lining industry, in contrast to the warranty practices of other industrial markets, and in the context of the legitimate, though often conflicting, interests of the purchaser and the seller in a synthetic lining system transaction. It suggests that the "twenty-year" warranty push in this industry is a dangerous and commercially and socially irresponsible trend which, due to the advantages it gives to the "gamblers" among us, and the disadvantages it places on those of us who are trying to bring stability and longevity to an industry plagued by past problems, could cause irreparable harm.

BACKGROUND

Probably for historical reasons, the geomembrane industry seems to have inherited some of the warranty practices of the roofing industry, which has frequently offered long-term (often twenty-year) warranties. In most respects, these warranties (usually underwritten by bonding companies) have provided significantly less protection for the purchaser than their "twenty-year" captions might suggest.Vague and numerous qualifications and conditions, coverage of materials but not installation, pro rata allocation of warranty repair costs and other avenues of escape often deprive the caption of substance.

It may be useful, before exploring the "twenty-year warranty" phenomenon, to lay out the statutory and contractual framework for warranty discussions.

In the United States, the sale of goods and related services is usually governed by the Uniform Commercial Code ("UCC"), which has been adopted, with minor variations, in all states except Louisiana. Under the UCC, the term "warranty" is used to refer generically to any promise, covenant, guaranty or assurance of the quality, characteristics, condition or performance of goods. The UCC identifies several types of warranties, the most common of which are express and implied warranties . (The UCC also deals with warranties of clean title and warranties against patent infringement, which are beyond the scope of this paper.)

Technically, an express warranty can be any of the following representations which is made part of the basis of the bargain:

- an oral or written promise or affirmation of fact made by the seller to the buyer which relates to the goods,
- any description of the goods,
- a sample or model of the goods.

In a commercial context, express warranties are generally those set forth by the parties in the contract documents. Statements in advertisements, oral assurances and other affirmations can, under certain circumstances, create warranties beyond those written in the contract if the seller is not careful to merge them into the resulting contract document.

The most common express warranty is one of freedom from defects in materials and/or workmanship, either at the time of sale or for some fixed period of time thereafter. Express warranties can, however, take innumerable other forms, depending on the characteristics of the product and the demands of the market.

In addition to any express warranties offered by the seller, the UCC provides that certain warranties may be implied even though the seller may not expressly offer them. The two most important implied warranties under the UCC (and those which can create the greatest potential for liability) are the implied warranty of "merchantability" and the implied warranty of "fitness for the (buyer's) particular (i.e., intended) purpose". (UCC Sections 2-314, 2-315) Both types of implied warranties can be included in the sale of goods.

Under the UCC, goods are impliedly warranted as "merchantable" unless the seller negates the implication through a disclaimer. (UCC Section 2-314) To be "merchantable", goods must meet several standards set forth in the UCC, which can be roughly summarized as "generally acceptable quality levels for the particular industry." Sellers learned quickly after enactment of the UCC that the "merchantability" of goods, which is often judged by a product liability jury with the benefit of hindsight, is a virtually limitless concept often used as a catch-all in product litigation where no liability can be attached to any express warranty.

The "implied warranty of fitness for the particular purpose" is created by UCC Section 2-315, which provides that:

> Where the seller at the time of contracting has reason to know any particular purpose for which the goods are required and that the buyer is relying on the seller's skill or judgement to select or furnish suitable goods, there is unless excluded or modified under the next section an implied warranty that the goods shall be fit for such purpose.

As explained in the Official Comments to Section 2-315:

> 2. A "particular purpose" differs from the ordinary

purpose for which the goods are used in that it envisages a specific use by the buyer which is peculiar to the nature of his business whereas the ordinary purposes for which goods are used are those envisaged in the concept of merchantability and go to uses which are customarily made of the goods in question. For example, shoes are generally used for the purpose of walking upon ordinary ground, but a seller may know that a particular pair was selected to be used for climbing mountains.

This,too, can create unknown pitfalls for the seller because the "particular" purpose can be quite unique to the purchaser and outside the seller's expertise and control. For example, a geomembrane supplier may know a great deal about the general qualities and technical specifications of his product and its general suitability for a variety of lining applications; however, he may not have sufficient expertise in the technical aspects of physical loadings to understand the ability of the product to withstand the strain of a 50-foot gypsum stack in a desert in the Middle East. He is not, therefore, really capable of warranting the "fitness" of his geomembrane for that particular purpose.

The UCC allows both of the implied warranties to be disclaimed in the contract if the disclaimer language is "conspicuous" (e.g., capitalized and/or bold face) and, in the case of the implied warranty of merchantability, if the magic word "merchantability" itself is mentioned in the disclaimer. (UCC Section 2-316) Because both of the implied warranties create potentially significant liability exposure, they are routinely disclaimed by sellers, particularly in a commercial or industrial context, in favor of simpler and more precise express warranties.

The UCC also allows a seller to limit the remedies available to its customer should the warranty be breached, and to limit the dollar amount it might otherwise be obligated to pay to fulfill the limited remedy. (UCC Section 2-718, 2-719) Accordingly, rather than assuming unlimited responsibility for all financial consequences caused by a breach of a warranty, most sellers limit their remedy to repair or replacement of the defective article. (This is particularly important in the context of major capital goods projects for a component supplier who cannot be expected to assume the financial responsibility for a malfunction of the entire system if his small component part should fail in use.) Limitations of remedies are fully enforceable, even when coupled with implied warranty disclaimers, provided that certain statutory language requirements are complied with, that certain contract formation formalities are met and that the seller, in fact, honors the precise limited remedy provided. (Otherwise, a court could strike the limitations and disclaimers for having "failed (of) their essential purpose" (UCC Section 2-719) and allow the buyer full access to all breach of warranty remedies provided by the UCC.)

Finally, the UCC also segregates damages (the compensation provided to a victim of a contractual breach) into several different types -- direct, incidental and consequential. (1) Although the distinguishing features are complex, the different types of damages can be conceptually separated.

From the standpoint of a seller's liability, direct damages are those required to put the buyer immediately back into the position he would have been in if the contractual promise (whether a warranty, delivery time commitment or other contractual assurance) had been met. For example, if a seller delivers defective goods, the buyer may be forced to have the goods repaired or to purchase other goods at a higher price, and the buyer is directly damaged by the cost of repair or price differential. He may also, however,have incurred "incidental" damages related to the replacement of the defective goods, such as freight and storage costs for the return and replacement of goods and other out-of-pocket expenses. Finally, the failure or temporary loss of the goods could have caused the buyer "consequential" damages -- lost profits during factory down-time, liability to third parties for failure to deliver, extra working capital and interest expense, liability for environmental damage to third parties because his supplier's product failed, or any other adverse consequences which the seller could reasonably foresee at the time the contract of sale was made. The two latter damages categories (incidental and consequential) are the most potentially expansive damage concepts, and liability for such damages is almost always excluded (as is permitted by the UCC) in a commercial contract of sale.

WARRANTIES IN THE GEOMEMBRANE INDUSTRY

With this statutory background in mind, let us consider warranties in the geomembrane industry in light of technical and commercial risks we encounter.

The most important issues we face and, not unexpectedly, the most common issues are:

1. What will be warranted;
2. For how long; and
3. What remedy will be provided for warranty breach.

Each commercial sale theoretically has its own set of facts and therefore should have its own tailored warranty. But humanity, to progress, builds on the experience and knowledge of the past. We develop useful generalizations, rules and forms for easily duplicating past learning. We often tend to copy each other's forms and follow common patterns of behavior, rather than continually re-invent the wheel.

And so it is with warranties: Where there have been common elements to transactions, we have tended to develop common approaches to warranties. The human knowledge-building process advances the condition of mankind so long as the duplicated experience proves to be beneficial. But it must be continually questioned and if it does not prove out, it must be removed from conventional wisdom, revised in light of experience, and replaced. Unquestioned duplication has not, for reasons explained below, been altogether wise as it pertains to geomembrane warranties.

Two distinct warranting patterns have emerged to date in the geomembrane industry. The first is a simple, but limited express warranty (such as freedom from manufacturing defects at the time of sale, freedom from defects in installation workmanship, compliance with agreed specifications, etc.) for a reasonable period of time. Standard characteristics are defined without regard to actual use, contractually placing the onus of ascertaining the fitness of the product for the particular applications (the conditions of which are beyond the geomembrane supplier's control) on the customer or its engineers, but a complete repair/replacement remedy for warranty breaches is amply provided. The second pattern, the catalyst for this paper, is the practice of offering long-term (often up to twenty years) warranties of "useful life", "weatherability" or similar durability-related concepts.

Warranties of the second type are highly unusual in an industrial equipment context (2), but alas, not in our industry. Unfortunately, they seduce our customers, especially those who are unsophisticated in warranty and liability concepts or unwilling to read the "fine print", and have become "straw men" of the trade in warranty negotiations on major projects.

The "twenty-year warranty" is usually extremely detailed, very conditional and, in actuality, of little protection for the buyer as compared to a fuller but shorter term warranty.

Here are the terms and conditions geomembrane customers generally encounter in the "twenty-year" warranty:

1. Only materials are covered. Repair or replacement materials will be delivered to the site, but labor and other costs for warranty repair installation services are at the expense of the purchaser.
2. The warranty is pro rated over its stated term so as to charge the purchaser for an increasing portion of repair or replacement materials as time goes on.
3. The liner normally has to be covered at all times and in all places.
4. The warranty applies only to significant defects actually impairing the functioning of the liner system.
5. The warranty only applies to defects occuring under specified service conditions. Excluded are damages or defects resulting from a variety of vaguely articulated unknowns such as "harmful chemicals", "excessive stresses", "acts of God".

While pro-rating the cost of replacement material over the twenty-year period obviously deprives the customer of almost half of the value of the warranty, if there is any coverage applicable after the loopholes of warranty scope (see point 5) are navigated, the limitation of the remedy to repair or replacement material-only is the real sleeper.

For example, if the lining system were to develop a leak in the tenth year, the customer would have to find it, decide that it was covered by the warranty, notify the lining supplier, and clean the area for inspection by the lining supplier, who would then decide whether the leak was caused by a "defect in the material" or "normal weatherization" (which often are the only causes warranted against). Assuming the material was defective, the lining supplier would be obligated to provide enough material (at half price (10/20) and at the day's prevailing prices) to patch the leak, often only a square meter of material. All related costs of installation and repair would be borne by the customer. Thus, for the ordinary leak resulting from a material defect, the 20-year warranty obligation is worth the prorated price of a square meter of lining -- which is an insignificant proportion of the actual cost of repair at any time during the warranty period.

The plain truth is that a material-only warranty of any length is a low risk proposition for a supplier of good quality linings, but it is a very high risk proposition for the customer who has been lulled by the twenty-year caption into the false belief that he has a meaningful warranty and consequently has provided for no other coverage against a lining failure.

When problems occur in lining systems, they usually have been caused by improper site preparation, material selection, installation or uses, and not by material defects or normal weatherization. The key to adequate warranty coverage, then, is to obtain warranties from the designers, construction engineers and lining installers for their respective roles in the performance of the lining system. More specifically, lining system faults are most likely to arise during installation and are usually attributable to defective joints or careless punctures, so it is particularly important for the customer to rely upon the warranty of the actual lining installer for the bulk of the warranty protection. In fact, the customer's principal interface should be with the installer, who should be held solely responsible for the future performance of the entire lining system.

The reason for single source responsibility is elementary: if the designer, construction engineer, lining manufacturer and system installer each have separate obligations to the owner or prime contractor, they can each argue that a lining failure falls under the responsibility of one of the others. Of course, to make this single source responsibility more palatable to an installer, the owner or prime contractor would have to require the others responsible for the system performance to indemnify the installer for any role they may play in any lining system failure. A material warranty given to the installer by the manufacturer then would be understood for what it is,a material-only warranty, and would create no illusions in the installer's mind.

How does the normal "twenty-year" warranty stack up against this standard? Not very well.

Because of the twenty-year warranty illusion created by some lining manufacturers, insufficient attention has been paid by purchasers to the background, integrity and staying power of lining systems installers: What kind of assurances does the customer have that the installer will be around to make repairs or pay for them two or ten years from completion? How long has the installer been in business? Is it a substantial corporation with history and evidence of stability that will be around twenty years from now, or was it formed yesterday as an operation owned by one man? Do the customer's procedures for approving an installer include determining whether the installer should be required to provide a bond for performing his warranty? How good are performance bonds anyway? What is the installer's track record and reputation for fulfilling his warranty obligation? What kind of company would readily give an owner increased warranty protection without increasing the price -- a financially prudent company? One that expects to fulfill its warranty? Would you rather have a two-year full warranty on a Mercedes-Benz or a twenty-year parts-only warranty on a De Lorean? In practice, the words on paper are less important than the intangibles that have always driven the engines of commerce: the manufacturer's own self-interest in protecting its reputation and the rewards associated with building a reputation for consistent value and quality.

The twenty-year warranty illusion has also masked another real issue, one which is difficult for a supplier to face. It is easier to give the customer what he wants -- a "twenty-year warranty" -- than to talk about sharing the risk. In the final analysis, however, shifting or sharing the risk is what warranties do -- they are insurance contracts of a sort. And facing the issue with the owner is the responsible course.

The extent of a vendor's responsibility to warrant environmentally-sensitive products depends on a number of factors, including the stage of development of the technology in the industry, the technological aspects of the particular application, and the value to the vendor of the particular contract. Of course, the customer has an obvious interest in the durability of the product and can legitimately expect the vendor to stand behind the product for some fixed period of time. In light of these competing interests, from both the vendor's and the customer's standpoint, the actual scope and strength of the product warranty should be more important than its stated duration. On close analysis, long term ("twenty-year") warranties really offer the purchaser little except a false sense of security. That is not to suggest anything intentionally deceitful on the part of those offering these warranties in <u>every</u> case; rather, it is simply recognition of the fact that warranty exposure so far in excess of that which is customary for other types of industrial components results in very precisely delimited responsibilities and the attachment of numerous conditions to the warranty obligations. More importantly it is easier to sell to those who care the least about, or are more willing to gamble with, the long term consequences it entails.

Responsibility for the product really has to be shared, particularly in a rapidly changing industry such as the lining industry, which has yet to reach technological maturity. The designer must be responsible for proper design, the material suppliers must be responsible for the quality of the materials, and the construction contractor and installers must be responsible for the proper performance of their functions, all according to the state of the art. But the owner -- who opted for an installation at a certain stage in technological development and who, by electing the type of facility for which the liner is to be used, has complete control over the risk to the public that any failure might create -- should assume the risk of the unknowns of technology and chance.

Significant material or installation defects are generally manifest within two years after installation, and no one can contest the justice of imposing on the installer, for that length of time, the responsibility for his performance or for the performance of the system if he in turn can look to his suppliers to fulfill their respective responsibilities. But after two years, it becomes quite difficult to allocate responsibility for failures between suppliers on the one hand and external or unknown factors on the other. Thus the risk of assuming warranty responsibility for an unknown over which the installer has no control is heightened after a couple of years and increases the insurance cost of the warranty, and the cost of the system to the customer as a result. (In most cases, it is probably cheaper for the customer to self-insure.)

Our industry must be more sentitive to its social responsibities than others may feel compelled to be. Acceeding to industry norms designed to evade responsibility for defective material, sloppy installation or poor design is socially irresponsible -- and the development of law in other areas of public concern proves that such evasions simply will not work in the long run. Creating the illusion of protection when none is given is equally irresponsible. And it is certainly not an appropriate selling tool for honest competition.

We believe that the industry should offer warranties that assure the public that what we undertake to do, we undertake to do well. Certainly there is no harm in giving the customer more if he demands it, but we should not be panderers of illusions. Even if it means that negotiating a contract will be harder, we should be forthright to a fault about what we do not cover -- leading the customer through the legalistic verbage if necessary -- so that the customer can seek elsewhere for additional protection, through insurance or otherwise, if needed.

The tendency of owners and prime contractors to push suppliers for twenty-year warranties will, in the long run, be counterproductive, because it favors less risk averse, and usually less stable, suppliers in an industry whose existence depends on the longevity of responsible and stable suppliers. The geomembrane industry, through candid and forthright negotiations between buyer and seller, can eliminate that potentially dangerous trend.

References

(1) We have addressed only the geomembrane supplier's contractual warranty liability to direct customers. The failure of the supplier's lining system may create other categories of liability to third parties (perhaps including the owner of the site if that is not the direct customer) under theories of negligence, nuisance, strict liability in tort or other non-contractual liability theories, including the expanding concept of "toxic tort" (ultra-strict, or "no fault", joint and several liability for those who contribute to pollution damage claims, regardless of their share in causation). In the third party liability area, protection for the geomembrane supplier can, in the last analysis, only be assured by comprehensive quality control techniques and a sound insurance and loss control administrative program.

(2) A recent Conference Board Report entitled "Industrial Products Warranties: Policies and Practices" summarizes the results of sampling over 350 U.S. manufacturing companies with respect to their warranties and warranty practices. Although the Report covers a variety of companies with a variety of products, it is interesting to note that, even in the industrial products field, average warranty periods are in the one to two year range, with an "extended" warranty considered to be one of up to five years maximum. In addition, in most cases, companies charge an extra fee for the extended warranty beyond the standard one or two year period.

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New Applications for Geomembranes: Lining Solar Ponds

Salt gradient solar ponds are a proven technology, even though their impact on future energy supplies is, at this time, speculative. This paper addresses issues of importance to the geomembrane industry concerning this technology, including:

- An explanation of the salt gradient phenomenon;
- An assessment of the role to be played in the technology by geomembranes; and
- A discussion of the potential for broad-based, commercial-scale usage of the technology.

The Phenomenon of the Solar Pond

What is a solar pond? Webster offers no definition of this term. It is generally accepted, however, that a "solar pond is a body of liquid which collects solar energy and stores it as heat."(1) One might assume that this means that a common surface impoundment could be used to collect solar energy, but there are innumerable natural and man-made impoundments that are not effective collectors of solar energy. How then can a simple pond be made to collect and store the energy provided by the sun?

Many approaches and techniques have been studied and modeled with varying degrees of success. Most are a modification of the classical flatplate solar collector which results in a system "limited in size to a few square meters; large collecting areas are only possible by connecting an assembly of these units. Thus, to collect solar energy on a really large scale requires a radically different approach."(2)

To many in the scientific community, the greatest potential for large scale solar energy collection is held by the phenomenon known as the salt gradient effect. "Imagine an energy producing technology that uses neither oil nor gas nor uranium but common salt, a more abundant and less expensive mineral. In this technology, a pound of salt supplies as much electricity or three times as much heat as a pound of coal burned in a combuster. Yet miraculously, after the energy is produced the salt remains, while the coal is used up and its by-products scattered to the four winds, the water, and the soil."(3) Such is the promise of the salt gradient solar pond.

The salt gradient solar pond is not merely a feasible technology; it is a naturally occurring phenomenon. "It is found in a few places in the world where a rare combination of circumstances results in a body of water being more saline at some depth below the surface than it is at the surface."(4)

A naturally occurring salt gradient solar pond was first observed by the Russian scientist, von Kaleczinsky, who published the first study of naturally occurring, solar-heated, salt-gradient lakes in 1902. He measured unusually elevated temperatures of more than 70°C (126°F) in isolated areas of Lake Medve, in Transylvania. The bottom of the lake was saturated by a bed of salt, while freshwater runoff and rainfall fed the surface waters. The resulting concentration gradient of dissolved salt, from the bottom to the surface, provided the vertical density gradient that allowed the solar energy to be absorbed and stored.

In the years since von Kaleczinsky's observations, naturally occurring sites have been found in other areas of the world. Science has actively sought to understand and harness this source of energy, but natural sites are not always conveniently located for effective study and utilization. As a result, scientists have sought to mimic nature by fabricating salt gradient solar ponds of their own. "In Israel, studies of solar ponds began in the late 1950's. The first experimental pond - a 600 square meter pond built at the Dead Sea Potash Works in 1960 - attained a temperature of 96°C (205°F), proving that the concept was workable."(6) Since then the number, size and complexity of such ponds has grown. In addition to proving that the salt gradient solar pond concept is practical, most of these ponds serve as useful sources of energy. Indeed, there are many examples today of this technology producing useful work with energy from the sun.

In the U.S., pilot-scale ponds provide heat for crop drying, space heating and, in one case, heating for a municipal swimming pool. In 1975 a 100 square meter pond was built at the University of New Mexico where, during the summer of 1980, a record temperature of 108°C (227°F) was attained. "Heat extracted from this pond would have been sufficient to provide the requirements of a single family home."(6) A pond built at Louisiana Tech University in 1982 is now providing heat for greenhouses on the Agricultural Engineering campus. It is also the basis of study under a grant from the Louisiana Department of Natural Resources. The study's goal is a design/construct manual for salt gradient solar ponds in the Gulf Coast region.

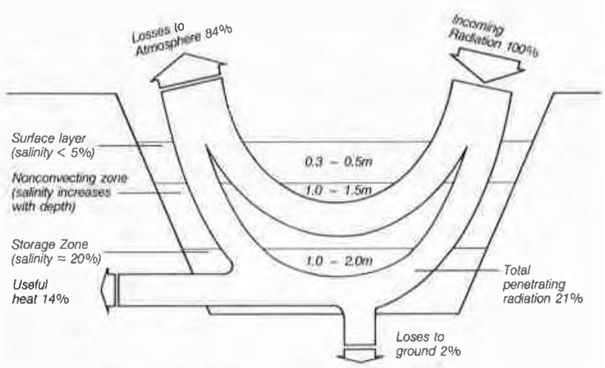
In addition to the U.S. and Israel, Australia, India and other countries of the world are conducting similar work. "The list of countries active in solar-pond research is growing, indicating a high level of interest in this technology. Experience totals 10 to 15 pond years in

Israel, 20 to 25 pond years in the United States, and about 15 pond years in the rest of the world."(6) This work has resolved many major questions and seems to be paving the way for the construction of commercially scaled facilities. While financial, political and technical uncertainties remain, the basis of this "energy from the sun" technology is now proven.

The Salt Gradient Effect

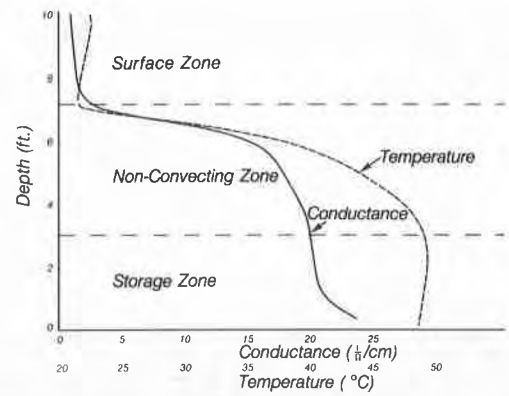
The absorption of solar energy by water depends upon a number of factors, water clarity and incidence angle for example. When heated, fresh water expands and becomes less dense, rising to the surface of a pond and losing its heat to the atmosphere. Wisps of vapor rising from a pool on a cool morning are evidence of heat loss. Natural convention causes this constant energy exchange during daily cycles. This general state of equilibrium must be altered if solar radiation is to be collected and stored.

Dissolved salt can be used to create layers of water with different densities that will impede the normal convection process. By feeding clear fresh water to the surface of a pond and heavier saturated brine to its bottom, a concentration gradient can be established. The percent salt concentration near the bottom can range in excess of 20%, with gradually lessening concentrations nearing the surface. With an established salt gradient the pond will exhibit three zones, as shown in Figure 1. Wind, waves, evaporation and surface cooling at night create a surface layer. Below the surface layer and isolated from surface effects, is the non-convecting zone where salinity and temperature increase sharply with depth (see Figure 2). The bottom of the pond is the storage zone.



A salt gradient solar pond has three zones: a surface zone, a non-convecting zone where salinity and temperature increase with depth, and a storage zone at the bottom of the pond, from which energy is withdrawn.

FIGURE 1



Temperature and conductance profile in a salt gradient solar pond. September 22, 1983, Louisiana Tech University.

FIGURE 2

Much of the sunlight striking the surface of a salt gradient solar pond is reflected. Another significant portion is absorbed by the surface layer and readily returned to the atmosphere. The remainder of the solar radiation penetrates the surface layer and warms the brine in the non-convecting and storage zones. The warmed brine would normally expand and begin to rise, but it is denser than the water above and cannot. The dissolved salt has created liquid stability, with concentrated density gradients that are more stable than the thermal density gradients that ordinarily promote convection. Thus, the salt gradient solar pond is indeed non-convecting; the warmed brine is trapped below.

"Depending on location, water clarity, and temperature, the solar pond can capture 10 to 20 percent of the solar energy hitting its surface."(3) Temperatures in the storage zone may reach and even exceed the boiling point of pure water. Even under less than ideal operating conditions, large quantities of low-grade heat are available. The problem then becomes one of containing the contents, maintaining operating conditions, and extracting the energy of the pond.

Construction and Operation of a Solar Pond,

and the Need for a Geomembrane

In spite of its simple description, the salt gradient solar pond is a complex physical system which interacts strongly with local meteorology and geology.

Sound solar pond design recognizes and makes best use of site specific data in order that the finished product may collect and store energy most efficiently. Local topography and groundwater conditions will largely determine pond depth, relative to grade, and perimeter configuration, although in general a compact shape and uniform depth help maximize energy collection.

The ideal salt gradient solar pond site would have these characteristics:

- Free draining, dry soil
- Flat land, to minimize earthmoving
- Low cost salt nearby

- Access to water
- Good soil cohesion
- Easily compacted soil
- High incident solar radiation
- Low wind
- Low evaporative rate
- Deep groundwater table

Unfortunately, some of these items are contradictory. Free draining soils, for example, usually have low cohesive properties and are difficult to compact properly. Areas with a high incidence of radiation do not usually experience low evaporative rates. Therefore, the best available compromise is necessary. The actual earthworking requirements are site dependent and best left to the recommendation of those familiar with local civil engineering practices.

With a site selected and the design process begun, one of the major questions to be asked is whether to use a synthetic liner or a natural material. The use of natural site materials alone certainly lowers the capital cost of construction. Indeed, a prevailing opinion among solar energy pundits seems to be that "the economics of using solar ponds to generate power depend largely on the ability to operate without a synthetic liner."(3) Some experts suggest that "low cost containment methods not employing liners, such as lining with impervious clays, can be developed for high temperature use."(5)

Such opinions erroneously assume that natural materials are always and everywhere conveniently located and are available at low cost. This is simply not the case for many parts of the United States, where the purchase, transport and installation costs for natural materials, including clay, exceed that of most geomembranes.

It is the further opinion of this author, however, that the long term economics of operating solar ponds to generate power depend largely on the use of a suitable geomembrane. Geomembranes have long been at work isolating the contents of handling, treatment and containment facilities in industry. The containment problem posed by salt gradient solar ponds seems ideally met by geomembranes. Pond leakage causes the loss of heat energy and ecological damage to the immediate surroundings. The best natural soils and clays exhibit permeabilities of only about 10^{-7} cm per second. Permeability recommendations from researchers involved in the design, construction and operation of these ponds range from 10^{-8} cm per second (1) upwards, with permeability of 10^{-12} cm per second considered desirable. Such enhanced energy containment and environmental protection is attainable through the use of a geomembrane.

In addition to environmental and operational concerns, incomplete containment can have significant economic consequences. If salt is purchased, the use of clean sodium chloride is recommended. In the U.S. the delivered cost of salt varies from $25.00 to $56.00 per ton. A salt gradient pond will require from 30 to 100 lbs per square foot of surface area.(1) The actual weight per acre depends on the design depth selected. A pond containing 66 lbs per square foot at $35.00 per ton delivered would have an investment of $1.40 per square foot in salt alone (1), a substantial investment which should be protected.

In addition, the absence of a geomembrane can have a direct impact on a critical operating parameter: brine clarity. Fine silty clays can settle out in suspensions at different density levels; the saline and thermal environments can nurture a flourishing halothermophilic floral community (organisms that grow in salty environments at elevated temperatures).(2) Both events can absorb solar radiation which reduces pond efficiency and may hamper energy extraction. Neither is a problem in the presence of a suitable geomembrane.

A far more catastrophic problem can arise when heat storage in the bottom of an unlined pond causes the generation of biogenic or other gases in the subgrade. These gases collect and bubble up to the surface resulting in turbid conditions which destroy operating conditions. This actually occurred at a large salt gradient pond built at Atlith, Israel.(6) A geomembrane coupled with proper gas venting can avoid such disturbances.

Considering the capital construction costs associated with energy extraction, the use of a geomembrane to protect the salt investment and to enhance pond operations seems almost mandatory. Indeed, to date, every site-built pond in the United States employs a geomembrane. The materials that are now in use for this application are HDPE (high density polyethylene), CPE (chlorinated polyethylene), CSPE (chlorosulfonated polyethyelene), and a liner made from unknown base materials that goes under the trade name of XR-5[R].

This author is in full agreement with Short and Fynn's recommendations regarding the necessary characteristics required of a geomembrane in lining a solar pond:

> "Color - Black is generally most desirable because of its resistance to ultraviolet radiation. However, liner color has little effect on pond performance since the deposition of dust and biological material on the pond walls and bottom gradually change all colors to a dark gray/green.
>
> Temperature - The liner must be able to withstand temperatures from -30°C (-20°F) to 95°C (200°F) while remaining flexible and retaining strength.
>
> Weather - The liner must have the ability to withstand ultraviolet light while under thermal and chemical stresses. The most vulnerable area is the material around the pond surface and berms.
>
> Chemical - The liner must have the ability to withstand saturated brine (NaCl) solution at 95°C (200°F) with ions of free chlorine, copper, iron, aluminum, and sulphate present in the solution.
>
> Micro-organisms - The liner must remain impervious to attack from algae and other water- or soil-borne organisms.
>
> Rodents - The liner and foundation should resist rodent attack. Rodents will burrow alongside a warm solar pond during the winter.
>
> Wicking - The liner fabric should not wick appreciably, or delaminate (if laminated), if a raw edge is exposed to saturated brine at 95°C...
>
> Welds - Welds should be at least 5 cm in width and as strong as the parent material. They will need to be rigorously tested and inspected for weaknesses or potential leaks.
>
> Permeability - The liner should be impermeable to hot brine. A permeability of less than 10^{-8}cm/sec should be acceptable.
>
> Mechanical Specifications - The material must be highly resistant to abrasion, puncture, and tearing and must maintain its flexibility over its useful life..."(11)

This author only notes, however, that a black liner is not necessarily resistant to UV radiation. Only if the liner is black as the result of the addition of carbon black, or another UV stabilizer, can the geomembrane be considered UV resistant.

The best source for comparing a material's properties with these guidelines for a geomembrane considered for a solar pond is the recently published Standard No. 54 on Flexible Membrane Liners by the National Sanitation Foundation.

The Future For Solar Ponds

Worldwide experience has proven the salt gradient concept a workable and practical solution to many energy needs. As energy costs rise, there will no doubt be increasing use of solar ponds for site specific purposes, such as space heating of buildings, industrial process heat, agricultural process heat, and water desalination. For broad-based commercial-scaled utility, however, the electric power sector must find economic viability in the technology.

The U.S. Department of Energy published a study of the "Regional Applicability and Potential of Salt-Gradient Solar Ponds in the United States" in 1982, concluding that abundant resources exist in the United States for developing salt gradient solar ponds to supply electric power and low temperature thermal energy. "The total energy supply potential of solar ponds in the United States in the year 2000 is estimated at 8.94 quadrillion BTU's per year. This amounts to 7.2% of the projected national energy demand for that year. An estimated four million acres of solar ponds will be required to produce 8.94 quads per year."(8) The study, performed for DOE by the Jet Propulsion Laboratory at the California Institute of Technology, reported that more than three quarters of the nationwide electricity-generating potential for salt gradient solar ponds is in the Gulf Coast/ Red River Valley regions.

It is little wonder then that a recent prospectus from the Gulf South Research Institute entitled "Salt Gradient Power Ponds" states, "It is not unreasonable to envision the startup of a 50MWe (megawatt of electrical power) salt gradient power pond in Louisiana by the year 2000."(7)

To achieve this goal, Gulf South Research Institute has outlined a phased implementation plan, with five major milestones:

- 1983 - Salt gradient solar pond design and operating manual
- 1984 - Demonstration center operating in Louisiana
- 1984 - Siting study for the Gulf Coast/Red River Valley region
- 1990 - Utilities/industries operating 5MWe power plant modules
- 2000 - Commercial 50 MWe power plants on-stream

This somewhat conservative time table could be speeded considerably by shifts in world politics and economics. Indeed there is some evidence that commercial scaled electricity generation may be closer at hand than this timetable might suggest.

Spurred by the energy crisis of the early 1970's, the nation of Israel committed itself to commercial development of salt gradient solar ponds. In December 1979, Ormat Turbines, Ltd., Yavne, Israel, began producing electricity from the largest salt gradient power pond in the world. The pond is located near Ein Bokek, Israel, adjacent to the Dead Sea. It has a surface area of 1.5 acres and has produced as much as 150 KWe in a peak load configuration. Another 50 acre pond was scheduled for completion in late 1983. It will have a capacity of 5 MWe operated for base load, and 10 MWe for peak load, producing an estimated 25 million KWh of electricity per year.

By the turn of the century, the Ein Bokek facility is expected to encompass 125,000 acres of solar ponds with 50 MWe modular plants totaling 2000 MWe in generating capacity.(7)

For those involved in the development of this technology the successes in Israel are milestones of great importance. However, the long term utilization of all alternative sources of energy will depend on the supply and demand of conventional sources; as the cost for conventional energy rises, the demand for renewable, locally controlled, and low cost energy will rise also. Among such sources, salt gradient solar ponds rank highly.

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Suggested Further Reading:

1. Fynn, R.P., Short, T.H. and Roller, W.L., "Necessary Liner Characteristics For Use With Salt Gradient Solar Ponds," SERI Workshop on Impermeable Barriers for Solar Ponds and Biomass Systems, (Snowmass, Colorado, August 27-28, 1981).

FIGURE 3 - A salt gradient solar pond at Louisiana Tech University was lined with a 2.5 mm high density polyethylene geomembrane. The pond is 75' x 75', 1:1 side slope, 12' vertical depth. Capacity of the pond is 350,000 gallons of brine.

FIGURE 4 - This pond design required 140 tons of salt (NaCl) shown here being placed in the pond.

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Field Studies of Geomembrane Installation Techniques

Fourteen construction sites where geomembranes were being installed were visited to observe subgrade preparation and liner installation techniques. These sites were visited during a study conducted for the U.S. EPA, Solid and Hazardous Waste Research Division. The sites included mine tailing disposal impoundments, landfills, potable water reservoirs, geothermal brine impoundments, evaporation impoundments, and industrial wastewater treatment impoundments.

Six generic types of geomembrane materials were included in this study. They were (1) polyvinyl chloride (PVC); high density polyethylene (HDPE); (3) chlorosulfonated polyethylene (CSPE); (4) chlorinated polyethylene (CPE); (5) Neoprene®; and (6) ethylene propylene diene monomer (EPDM). Observed subgrade preparation procedures and geomembrane installation techniques are described in this paper.

INTRODUCTION

Geomembranes are presently being used to contain fluids in hazardous waste surface impoundments and landfills. They function as a low permeability barrier to fluid flow. Many factors contribute to the overall effectiveness of the geomembrane system.

The proper planning, design, and construction of lined containment facilities involve numerous steps, including the following:

(1) Defining facility function and geometry;

(2) Designing a liner system that is compatible with the substances to be stored or treated;

(3) Planning suitable subgrade preparation, seepage monitoring, and collection systems (if appropriate);

(4) Planning proper liner installation; and

(5) Developing appropriate post-installation operation, maintenance, and closure plans.

This paper describes subgrade preparation techniques and placement procedures for geomembrane liner materials observed during a study performed by Southwest Research Institute for the U.S. EPA.

FIELD OBSERVATIONS

Fourteen construction sites where geomembranes were being installed were visited during the study. The type of facility and type of liner for each site is shown in Table 1. The installation of geomembranes requires proper planning before construction. Important elements of installation are adequate subgrade, onsite storage of materials, installation equipment, manpower requirements, procedures for liner placement, field seaming procedures, sealing around structures or penetrations, quality assurance/quality control procedures and soil cover requirements. Observations made at these sites regarding each of the major elements of installation procedures are described in the following sections.

TABLE 1. GEOMEMBRANE LINER CONSTRUCTION SITES VISITED DURING THE STUDY

Facility Type	Liner Type
Mine Tailings Storage	Polyvinyl chloride/ chlorosulfonated polyethylene
Municipal Landfill	Polyvinyl chloride
Evaporation Pond	High density polyethylene
Potable Water Storage	Chlorosulfonated polyethylene
Brine Storage	Chlorinated polyethylene
Tailings Storage	Chlorosulfonated polyethylene
Potable Water Storage	Chlorosulfonated polyethylene
Evaporation Pond	Chlorinated polyethylene
Hazardous Waste Landfill	Chlorosulfonated polyethylene/clay soil
Spill Containment	Neoprene®
Industrial Waste Storage	Ethylene polyethylene diene monomer
Municipal Landfill Cover	Polyvinyl chloride
Municipal Wastewater Storage	Chlorosulfonated polyethylene
Spill Containment	Polyvinyl chloride

Subgrade Considerations

For an earthen structure, the subgrade must be firm and dry, free of all rocks, roots, debris, or other objects that might tear or puncture the liner. Backfilling is recommended if necessary to meet these conditions. Where vegetation has been cleared to prepare the site, or soil has been brought in to provide a bed for the liner, soil sterilization is specified to prevent grasses from growing through the liner. This is especially true in areas where prior growths of nut or quack grasses have existed. Areas where excavated soil is deposited to create subgrade may also require sterilization.

Compaction of the subgrade is normally specified to provide a firm support for all membrane lining materials. Generally, a fill subgrade is built up in a series of compacted layers, whereas an excavated subgrade is compacted only at the surface. Usually, the minimum compaction of the subgrade material will be specified. Most liner installations specify that the density of the subgrade be at least a specified percentage of that obtainable by the Standard Proctor Test, ASTM D698, with 90 percent of Proctor being the most frequently specified relative compaction. Some contracts will specify the compaction equipment which is to be utilized, number of equipment passes per layer, layer thickness, permissible water content range at placement, and method and location of water addition.

The regularity and texture of the surface is critical to a successful liner installation. A plane surface is the most desirable for liner placement, but is not always achievable or specified. In many installations, soil clods or local surface irregularities will be flattened (further compacted) by the overlying weight of the stored material after the facility is filled. Further, it is thought that the polymeric membrane liners will adjust their shape over any clods so that no detrimental effects will result. Nevertheless, rocks or irregularities with sharp edges should be eliminated from the finished subgrade prior to liner installation.

Compaction densities were at least 90 percent of Proctor for all but one installation listed in Table 1.

In this installation, soil was placed over compacted refuse. A liner was then installed to prevent rainfall infiltration.

The installation contractor at the second facility, a municipal landfill, covered sharp sandstone subgrade with a geotextile before liner placement. The fabric served as a substitute for a sand bed.

Three facilities incorporated a cushioning medium over the native subgrade. At the second facility, 0.15 m (0.5 ft) of sand was placed over a rough sandstone. At the fourth and tenth facility, 0.30 m (1.0 ft) of sand was placed over concrete subgrades prior to liner placement. Liner installation was performed at three sites where the subgrade was saturated. However, the liner was not installed over standing water at any of these sites. Soil sterilization was accomplished at two of the 12 sites with earthen subgrades. At both of these locations, vegetation had existed prior to excavation. The subgrade at the eighth installation was not sterilized and, subsequent to liner placement, salt grass grew through the liner, necessitating significant liner repair.

Onsite Storage of Material

Membrane materials are normally shipped to a construction site, and must be stored prior to placement. Most materials are folded in panels and shipped on wooden pallets or in rolls. Protection of the liner materials from the effects of heat and from vandalism is recommended. These are the two most important storage considerations. Excessive heating can degrade the surface of the material, causing problems with field bonding. Covering the material with white plastic or storing out of direct sunlight is recommended for all materials. If the material is shipped in cardboard boxes, plastic shielding is not required. Placing the material in a secure area is recommended to prevent vandalism.

Onsite storage of liner material was observed at eight of the fourteen sites visited. All of these sites were being constructed within a secured area. Therefore, storage of material at the impoundment site was considered to be in a secure area. Protection from direct sunlight was provided at two sites. Field crew members at the remaining sites indicated protection was not needed due to either the short time between delivery of the material and placement, or existing ambient temperatures did not warrant protection.

Installation Equipment

Equipment often required to install a membrane liner includes a fork lift truck, backhoe, or front end loader for material placement, and various tools necessary for material positioning and field seaming. A fork lift truck with large rubber tires (not warehouse type), is most often recommended for material placement, because most material is shipped to a site on wooden pallets. The pallets can easily be maneuvered with a fork lift truck.

The equipment needed to seam the material together is basically similar for all types of material, with the exception of high density polyethylene. Table 2 lists the equipment commonly used by the industry. High density polyethylene is welded together and requires special equipment.

Manpower Requirements

Manpower requirements for the installation of liner materials is a function of the rate that the installer wants to place panels and accomplish field seaming. Typically, installation contractors will recommend five to ten people onsite when placing one panel at a time. Generally, a crew foreman will direct the activities of the field crew. He may not directly participate in the unrolling and positioning of panels or in field seaming. However, it is recommended that he be experienced in the installation of the specific liner material.

TABLE 2. EQUIPMENT AND MATERIALS FOR INSTALLATION OF GEOMEMBRANE LINERS

Item	Use
Fork lift truck	To move liner panels into position.
Backhoe	To backfill anchor trenches.
Tires, sandbags	To anchor unseamed panels to prevent wind damage.
Proper adhesives	To make field seams and seal liner around concrete or steel penetrations.
Portable electric generator	To operate heat guns or lighting for working at night.
Air lance	Quality control testing of field seams.
Hand-held earth tampers and rakes	To smooth and compact subgrade as necessary.
Ladders	For access to side walls/steep slopes.
Miscellaneous materials:	For field seaming.
- Adhesive applicators (paint brushes, caulking guns, rollers, etc.).	
- Liner preparation equipment: clean rags, scrub brushes, scouring pads, pails for solvent, hard surface rollers, seaming support board, heat guns, crayons for marking, dowels for pulling panels, stakes and chalk line, steel measuring tape, scissors and utility knives, electrical extension cords (for heat guns).	
Field crew equipment	For field crew when making seams.
- Safety goggles, solvent resistant gloves, knee pads, respirators, soft soled shoes.	
Portable tent/shed	To allow seaming activities in colder/windy weather.
First aid kit	In case of accidents.
Air compressor	Supply air that might be needed when working with solvents, and for air lance.

Crew size recommendations also depend on the complexity of the installation and the experience of the field crew. If the majority of the crew members are recruited locally, more members may be needed due to lack of experience. At the present time, the trend is toward having installation contractors retain field supervisors who travel from job site to job site. Large jobs where crews perform specific tasks may involve many people.

Field personnel observed at the study sites ranged in number from five to thirty. Generally speaking, the larger the site, the larger were the numbers of field personnel. At all but one location, crew foremen representing the installer were onsite to direct installation. Workers were hired locally as required. The exception was a high density polyethylene installation. In this case, all crew members were employees of the manufacturer and installer.

Liner Placement Procedures

Important considerations that should be followed in placing a membrane liner are as follows:

(1) Follow the design engineer's installation plan showing the location and position of liner panels;

(2) Follow manufacturers' recommended procedures for adhesive system, seam overlap, and sealing to concrete;

(3) Use a qualified installation contractor having experience with membrane liner installation,

preferably the generic type of liner being installed;

(4) Plan and implement a quality control program which will help ensure that the liner meets specification and the job is installed per specifications;

(5) Document inspection for review and record-keeping;

(6) Conduct installation during dry, moderately warm weather (above 45°-60°F depending on material); and

(7) Subgrade should be firm, flat, and free of sharp rocks or debris.

Before moving panels from the storage site to the installation location, a number of tasks must be performed. The anchor trench around the perimeter of the installation should be completed. The dirt excavated from the anchor trench should be raked smooth so that the panels can be unrolled along and parallel to the anchor trench in the width direction. Other things that must be accomplished prior to panel positioning are: (1) the subgrade should be raked smooth or compacted if necessary (2) there should be no standing water in the impoundment; (3) any concrete structures that must be seamed around should be prepared prior to unrolling any panels; (4) if skirts are to be used around footings on concrete structures, these must be in place prior to the beginning of panel placement; and (5) any outflow or inflow structures or other appurtenances should be in place.

Placement often begins with the unfolding or rolling of the panels in a lengthwise direction. The panels are then unfolded in the width direction, either down the side slope or across the floor. The panels are normally unrolled on the inside of the anchor trench, eliminating the need to move the liner across the trench. The field crew then begins to position or "spot" the panel into its proper location according to the installation plan. As panels are spotted, sand bags are placed along the edges to prevent uplift and subsequent wind damage. The instructions on the boxes containing the liner must be followed to ensure that the panels are unrolled in the proper direction with the correct side exposed for seaming. The panels should be pulled relatively smooth over the subgrade. If the subgrade is smooth and compacted, then the liner should be relatively flat on the subgrade. However, sufficient slack must be left in the material to accommodate any possibe shrinkage due to temperature changes. The amount of slack required depends on the material being installed. Each manufacturer has specific recommendations regarding the amount of slack that should be left during installation.

Unfolding of the panel is normally begun at the anchor trench around the perimeter of the facility. Once the panel is unfolded, it is pulled across the subgrade by the field crew. Workmen grip the panel using dowels which are rolled from the edge of the panel toward the center enough to provide a handle. The edge of the panel is raised and flipped to pump air under the liner. This allows the liner to be pulled into position without damage. This prevents stretching of the materials. Windy conditions may cause serious difficulties for a field crew installing a panel in this manner.

Once the panel is unfolded, the crew positions it properly, providing the necessary slack and overlap of seams. Sandbags are placed along the edges to prevent wind damage. At this point, field seaming crews begin joining the panels together. Table 3 presents a summary of liner placement activities observed at the sites visited during the study.

Field Seaming Procedures

During liner placement, panels must be positioned so the recommended seam overlap will be maintained along the entire edge of the pond. Generally, overlaps were 0.05 m (2 in.) to 0.3 m (1 ft) depending on the material. After correct overlap is achieved, field seaming crews begin inspecting the liner surface to be seamed together. Foreign material, such as dirt, or moisture is wiped off with rags or brooms. A surface oxidation or "cure" has to be removed from certain types of membranes. This is accomplished using a solvent such as trichloroethylene or methyl ethyl ketone. Workers normally wear protective gloves whenever working with these solvents. After removing the surface oxidation, a liberal quanity of solvent is applied. The material is allowed to soften at the surface. Solvent is applied to both surfaces to be seamed together. The two surfaces are pressed together with rollers. A board is often placed under the seam to provide a firm surface for rolling the seam. This procedure is typical for those materials seamed with a solvent. PVC, CSPE and CPE are commonly field seamed with solvents.

Contact type adhesives are used with vulcanized liner materials, such as EPDM. To seam these materials, the surface is prepared with a petroleum distillate, such as fuel oil. Then a contact adhesive is applied to both surfaces. When a tacky surface is produced, the two are folded over and pressure is applied. Often, a caulking compound is applied at the edge to complete the seam. A cap strip is then applied over the seam.

High density polyethylene liners are bonded together by welding. The automatic machine applies a ribbon of hot HDPE approximately 0.05 m (2 in.) wide between the two liner surfaces which have been preheated. Pressure is applied to the seam at the rear of the unit. The entire machine moves under its own power at a rate set by the worker. This procedure requires a skilled operator to achieve a successful weld. Power is supplied by an onsite portable generator set.

Solvent seaming produces a physical bonding between materials by chemically dissolving the surface layer of the material. Heat seaming simply melts the surface layer of the material to allow the bond to be achieved. A variety of heating devices are used to heat seam membrane liners. Liester guns provide non-contact heating of the material. Direct contact heating elements are also used. Heat seaming is difficult and does not necessarily provide a superior field seam compared to other available methods. Direct contact heat seaming was observed at only one site visited during this study.

Sealing Around Structures/Penetrations

Various approaches to bonding liner material to concrete, metal, or plastic structures were observed at the majority of sites visited. Generally speaking, where the structures were the same material as the liner, bonding appeared to be a simpler task requiring fewer steps. High density polyethylene and PVC liners can be bonded directly to pipes made of the same material. However, basic resins used to make these materials may have different thermoexpansion characteristics. Therefore, the installer should be sure they can be bonded together prior to installation.

The design of most installations visited included some type of structure that penetrated the liner system. These structures included inlet/outlet pipes, concrete support piers, drain flanges, gas vents, and concrete emergency overflow abutments. For those cases where liner material and structure material were incompatible, several approaches to sealing were observed. For sealing liner material to concrete, a combination of a mechanical and mastic system was used at one site. Anchor bolts

TABLE 3. SUMMARY OF LINER PLACEMENT ACTIVITIES OBSERVED AT 14 LINED FACILITIES

Facility Type	Mode of On-Site Material Storage	Air Temperature During Installation (°F)	Spotting Techniques	Overlap Width (in.)	Seam Width (in.)	Sealing System	Method to Seal Penetration	Gas Vents	Mode of Final Securing of Liner
Tailings Storage	Cardboard Boxes and Pallet	80-100	Ten-Twelve Man/Panel Machine Assisted	6	3	Solvent	Not Observed	Yes	Vertical Slit Anchor Trench Filled with Soil
Municipal Landfill	Unprotected on Pallets	35-50	4 Man/Panel	2-6	1-2	Heat Gun	Not Applicable	No	Anchor Trench Filled with Soil
Evaporation Pond	Unprotected Rolls	60-80 (Night)	Mechanized Due to Panel	6	1	Heat Gun	Not Observed	No	Half Anchor Trench Filled with Soil
Potable Water Storage	Cardboard Boxes on Pallet	50-65	8 Man/Panel	12	2-4	Solvent	Mastic with Mechanical Seal	No	Batten Strips to Concrete
Brine Storage	Cardboard Boxes on Pallet	80-90	Four-Eight Man/Panel Manual	6	2	Solvent	Mastic with Overlying Weight	No	Vertical Slit Anchor Trench Filled with Soil
Tailings Storage	Cardboard Boxes and Pallet	80-95	Not Observed	6	2	Solvent	Not Observed	No	Vertical Slit Anchor Trench Filled with Soil
Potable Water Storage Concrete	Cardboard Boxes and Pallet	60-95	Six-Eight Man/Panel	4	2	Solvent	Not Observed	No	Vertical Slit Anchor Trench Filled with Concrete
Evaporation Pond	Reflective Material Over Cardboard Boxes	70-950	Six-Eight Man/Panel	4-6	2	Adhesive	Adhesive and Tensioned Band	No	Half Anchor Trench Filled with Soil
Hazardous Material Landfill	Unprotected on Pallets	60-80	Not Observed	4-6	4	Solvent	Not Applicable	No	Half Anchor Trench Filled with Soil
Spill Containment	Cardboard Boxes and Pallet	60-80	Four-Six Man/Panel	4	2	Heat Gun	Adhesion, Mastic Tempered Band	No	Batten Strips to Concrete
Industrial Waste Storage	Burlap Wrapped Rolls	60-70	Six-Eight Man/Panel Machine Assisted	6	4	Adhesive and Lap Sealant	Not Observed	No	Half Anchor Trench Filled with Soil
Municipal Landfill Cover	Unprotected on Palletts	40-60	Six Man/Panel	2-6	1-2	Solvent with Heat Gun	Solvent with Mechanical Seal	Yes	Half Anchor Trench Filled with Soil
Municipal Wastewater Storage	Cardboard Boxes and Pallet	70-85	Four-Six Man/Panel	4	2	Heat Gun	Not Observed	Yes	Vertical Slit Anchor Trench Filled with Soil
Spill Containment	Unknown	90-95	Three Man/Panel	6	4-6	Adhesive and Lap Sealant	Adhesive with Mechanical Seal	No	Half Anchor Trench Filled with Soil

were placed into the concrete to secure steel strips. The steel strips were placed over the liner. Between the liner and the concrete, a mastic was applied to make a tight seal. A mastic recommended by the liner manufacturer that is compatible with the membrane and the waste was chosen for this purpose.

Liner materials and structural materials, such as concrete, are also joined using a combination of mechanical devices and boots made of the liner material. At one location, an inlet pipe made of fiberglass was isolated by the use of a boot constructed in the field with a mechanical clamp sealing off the pipe to fluid movement. The boot was bonded to the side slope of the impoundment to complete the seal. For a large number of penetrations, the installer generally will work with a fabricator to construct boots in the factory rather than in the field.

All polymeric liners are anchored at the berm or top of a landfill or surface impoundment. The edge of the liner is placed in a trench typically one foot wide and one foot deep. A backfill of earth or concrete secures the edge in the trench.

Quality Assurance/Quality Control

With respect to geomembrane liner system installation, quality assurance/ quality control (QA/QC) activities should be implemented to increase the confidence that the liner system was installed as specified. The more extensive the QA/QC program, the higher degree of confidence the owner would likely have that the system will function as intended.

Observed activities in the field which were intended to be QA/QC procedures ranged from sampling field and factory seams from every panel placed to visual inspection of field seams. At two sites, independent third party contractors provided checks of all installation activities and prepared a summary of compliance with installation plans and procedures. These contractors were onsite during the entire installation period. At one of these sites, samples of each seam were taken and sent to a laboratory for testing. Laboratory results were reviewed by the QA/QC contractor. If the samples did not meet minimum requirements, the seam was remade. The decision to remake the seam was not subject to interpretation. If problems were identified, resolution was the responsibility of the QA/QC contractor. At both of these sites, the seam testing performed was destructive testing in the sense that the sample was cut from the field seam. This method of testing meant that for each field sample taken, a patch was installed in the liner system. Both QA/QC inspectors believed the benefits gained from sample testing outweighed the drawbacks of having to patch an equal number of cuts in the liner.

Table 4 presents a summary of various QA/QC elements observed at the sites visited during this project. Personnel were assigned and wholly dedicated to the job of inspecting the quality of work at seven sites. As previously indicated, the owners of two sites contracted third parties to conduct QA/QC onsite. At the five other sites, QA/QC inspection was performed by either the installer's personnel or the manufacturer's personnel. The installation field chief provided QA/QC at the other seven sites.

Subgrade inspection was performed at every site, since this is critical to the success of a liner system. Testing to determine adherence to density requirements was performed at eight sites. Two of the remaining six involved concrete subgrades, and were not tested for that reason. No testing was performed at the remaining four sites.

The quality of the liner material in terms of visual appearance was determined at every site at the time the material was placed onto the subgrade. Quantitative, non-destructive testing was performed at two sites. Methods used were ultra-sonic testing and tensile testing of field seams. Destructive testing of seam samples was performed at two sites which were previously discussed.

Soil Covers

Several manufacturers of geomembrane liners suggest that at least a 0.15 m (6 in.) soil cover be placed over the liner after installation is completed. This practice is recommended to provide a protective barrier against weathering, mechanical, and other environmental damage. Ultraviolet light for extended periods can degrade certain polymeric materials. Most liners made of PVC are especially susceptible to UV light. Generally, PVC manufacturers suggest placing a soil cover over material exposed to sunlight for extended periods (>6 months). A soil cover also minimizes mechanical damage from landfilling operations, wave action, freeze/thaw cycles, vandalism, hail, and wildlife.

Soil covers were installed over the liner systems at seven of the fourteen sites. Three of these sites specified PVC geomembranes which required a soil cover for ultraviolet light protection. The remaining four sites installed covers for protection against heat or mechanical damage. Equipment to accomplish this task included bulldozers, pan scrapers, and front end loaders. Soil covers were not installed at seven sites. Manufacturers of these materials did not require soil covers as a condition of material warranty.

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TABLE 4. SUMMARY OF FIELD QUALITY ASSURANCE OBSERVED AT 14 LINED FACILITIES

Facility Type	Wholly Dedicated Inspection Personnel	Subgrade Inspection	Subgrade Testing	Liner Material Exam	Field Seam Examination: Visual Exam	Field Seam Examination: Physical Testing: Qualitative	Physical Testing: Quantitative: Non-Destructive	Physical Testing: Quantitative: Destructive
Tailings Storage	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes
Landfill	Yes	Yes	No	Yes	Yes	Yes	No	Yes
Evaporation Pond	No	Yes	Yes	Yes	Yes	Yes	Yes	No
Potable Water Storage	No	Yes	No	Yes	Yes	Yes	No	No
Brine Storage	No	Yes	Yes	Yes	Yes	Yes	No	No
Tailings Storage	No	Yes	Yes	Yes	Yes	Yes	No	No
Potable Water Storage	Yes	Yes	Yes	Yes	Yes	Yes	No	No
Evaporation Pond	Yes	Yes	Yes	Yes	Yes	Yes	No	No
Hazardous Materials Landfill	No	Yes	Not Known	Yes	Yes	Yes	No	No
Spill Containment	Yes	Yes	No	Yes	Yes	Yes	No	No
Industrial Waste Storage	Yes	Yes	Yes	Yes	Yes	Yes	No	No
Municipal Landfill Cover	Yes	Yes	No	Yes	Yes	Yes	No	No
Municipal Wastewater Storage	No	Yes	Yes	Yes	Yes	Yes	Yes	No
Spill Containment	No	Yes	No	Yes	Yes	No	No	No

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Containment of Magnesium Chloride Brine Solutions Using Hot Sprayed Elastomeric Membranes

Presented is the 3.5 years case history pertaining to the design, installation and service performance of hot sprayed elastomeric membranes to serve as evaporation pond liners for magnesium chloride brines. The particular membranes discussed are those based upon styrene-butadiene block copolymers and asphalt, containing other fillers and additives for specific tasks. The finished membrane blends, which comprise a thermoplastic mixture, were manufactured on site with a semi-portable batch plant and spray applied with specialized mobile spray equipment to a geocloth covered earthen base. Application temperatures were approximately 193°C (380°F). Application rates of 3763 m^2/hr (0.93 acre/hr) were achieved for placing a 2.29 mm (0.090 in) thick membrane in two passes. Overall, performance of the membrane has been good, however, two design items requiring further improvement are discussed.

CONTAINMENT DESIGN CRITERIA

The final design criteria required careful consideration of a large number of parameters including, but not limited to: location; local history and prior land use; climatological, geologic, hydrologic, soils and seismic conditioning; process parameters; economics and installation constraints; operation and maintenance directives; regulatory requirements, approvals, monitoring results; performance evaluation.

Each of the above parameters is sufficiently unique to require discussion of the considerations made. With two minor exceptions, the original containment system design has proved adequate for the intended service.

Location.

The project area is just north of Henderson, Nevada; on the north side of the Boulder Highway, east of Pabco Road. The location is approximately 19.3 km (12 miles) southeast of downtown Las Vegas, in the lower end of Las Vegas Valley, and 9.65 km (6 miles) from where Las Vegas Wash enters Lake Mead. See figure 1 for regional orientation and figure 2 for detailed location. The ponds are not visible from the Boulder Highway, but are very visible from the air as they are directly under the approach to McCarran International Airport from the east.

Local History and Prior Land Use.

Industrial use of the current project area dates from the early 1940's when an industrial complex for the manufacture of magnesium for incendiary bombs was constructed by the Defense Department. The plant area is south of the Boulder Highway and west of Water St. in Henderson. Approximately 4.05 km^2 (1000 acres) of shallow, unlined ponds were constructed simultaneously with the plant. Very few of these ponds were used, but they are still a prominent feature of the area.

The entire industrial area is now managed by Basic Management, Inc. with the main parts of the old magnesium plant being occupied and operated by three separate companies. TIMET, one of the three companies, occupies approximately the eastern one third of the main plant area.

There are a couple of sewage treatment plants in the area that have in years past operated with unlined effluent systems. There were also several open ponds, ditches, gravel pits and throughout a several square kilometer area that have contributed to the very complicated hydrologic condition which exists in the lower Las Vegas Valley today.

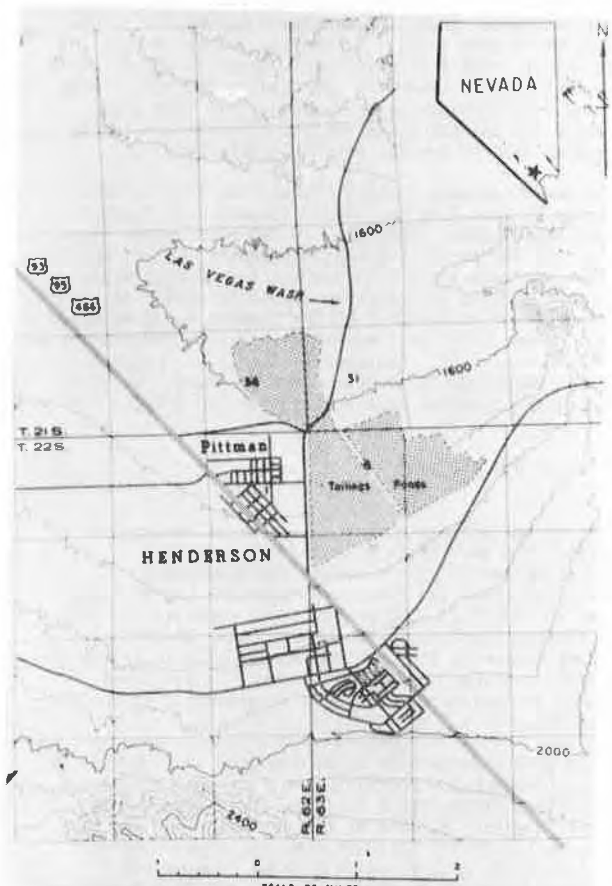

Figure 1. Map of lower Las Vegas Valley.

Climate.

The area is typical of many high desert areas in the western United States and is characterized by low rainfall and humidity, high evaporation, abundant sunshine, mild winters, hot dry summers and a wide range of temperatures both daily and seasonally (1). Elevation in the vicinity of the containment ponds is 533 m (1750 ft) Annual mean precipitation is 11 cm (4.33 in); most of which occurs in the winter. Relative humidity ranges from an average of 20% in the summer to 44% during the winter. While the normal high temperatures during the summer months are in the 38-41°C (100-105°F) range, extremes of 46-47°C (115-117°F) are not uncommon. However, the overnight lows during the summer are usually a comfortable 20-24°C (68-75°F). Minimum winter temperatures average 1.7°C (3°F) above freezing, but freezing temperatures are not at all uncommon. Annual pan evaporation rates for water average 284 cm (112 in) with summertime rates reaching 38cm/month (15 in/month). The area receives 262 clear days per year. Mean wind speed is 0.046 m/s (9.1 mph), although, winds over 0.254 m/s (50 mph) occur occasionally and are particularlty troublesome because of blowing sand. The prevailing wind is from the southwest.

Geology and Seismic History.

The project area is situated in Las Vegas Basin which is a "northwest trending structural depression formed by right lateral crustal distention and subsequent tilting of large crustal blocks. Stratigraphy of the alluvial floored basin and surrounding mountains range in age from Precambrian to Holocene"(1). The area geology and the following subject of seismic activity are described in further detail in reference (1).

A review of seismic history reveals the area to be classed as Zone 2 on the USGS'S Seismic Risk Maps of the United States. More than 10 000 microseisms have been recorded in the region since Hoover Dam was built. These were associated with the filling of Lake Mead, starting in 1936 and reaching a peak in 1954. An earthquake of magnitude 5 on the Richter Scale occurred near Boulder City, 16.1 km (10 miles) from the project site, in May, 1939. Subsequent aftershocks ranging between magnitude 3.5 and 4.0 were recorded. It is reasonable to expect continued seismic activity.

Soils and Hydrology.

Soils in the project area are classed as Quaternary-Alluvial Fan and Valley Fill Mixtures (Qsmg), "characterized as poorly sorted and moderately stratified sandy mud and gravel that are tan to light brown in color"(1). Generally these deposits are limy and average about 50 to 60 percent sand, 40 to 50 percent medium to fine subangular gravel with traces of nonplastic fines. Localized lenses of gravels and cobbles are present. Thickness is estimated up to 18.3 m (60 ft).

In the pond construction area, the Qsmg is 6 to 9 m (20 to 30 ft) thick and contains 2 to 3 percent nonplastic fines (4). Slopes dip 2 to 10 percent to the north. In the immediate as well as the general area, the Qsmg rests on top of the Tertiary - Muddy Creek Formation (Tmc), a light gray to grayish brown crossbedded conglomerate which is moderately to very well indurated with limy sand and silt. Scattered sandstone lenses are present.

The current hydrologic conditions of the project area and the surrounding vicinity are varied, changing and complicated. They are also well studied, being the subject of several studies for a Colorado River Basin Salinity Control Project by the U.S. Bureau of Reclamation (1), (2), (3), (5). In the study area, there are approximately 91 shallow and 58 deep observation wells. Several concentration contour maps of the area have been prepared which plot total dissolved solids and several individual cations and anions. However, a detailed discussion of such data is beyond the scope of this paper.

In spite of the large amount of data available on the near surface hydrology and salinity of the area, an understanding of the salt transport and ground-water dynamics has been slow in development. While much still lacks an adequate explanation, one point on which a concensus has been reached is that most of the salt now reaching the Colorado River from the study area is from vadose zone salts being dissolved by rising water tables and point source aquifer recharge. The degree of saturation of the near surface aquifer varies from a few to 100 percent, depending on location. The complex situation resulting from all of the various past uses of the area and all of the current influencing variables makes cause and effect assumptions hazardous at best. The situation is slowly improving and many small locations within the study area are now yielding to predictive techniques.

Process Parameters.

Seven evaporation ponds have been constructed since 1980 for the production of Bischofite from leaching effluent by solar evaporation (6). Those ponds have a total surface area of 182 070 m^2 (45 acres). The leaching effluent is generated during the production of

Figure 2. Aerial photograph of process pond area.

titanium metal (sponge) by the modified Kroll process. As the size of the ponds had been previously specified based on TIMET's expected production rates of leaching effluent, the only process parameter left to consider, from a liner design standpoint was effluent composition.

As generated, the leach liquor contains about 17% magnesium chloride, 2.5% magnesium nitrate, 1% HCL and traces of sodium, ammonia and sulfate. As the liquor moved through the first four ponds it was expected to concentrate to 50% of original volume before evaporation rates dropped to zero. Normal composition at this point would be 29% magnesium chloride, 6% magnesium nitrate, 2% calcium chloride, 1% sodium chloride, Sp. Gr. = 1.30 to 1.40. The liquid in the ponds up to this point was to be 30 to 122 cm (1 to 4 ft) deep with peak summertime liquid temperatures in the 4th pond expected to be 50°C (122°F) and peak liner temperatures expected to be about 66°C (150°F). Along the way the leach liquor is neutralized in a two-step process. It is first passed through a shallow lined pit containing coarse dolomitic limestone before the first evaporation pond. The final neutralization, to pH 5, is accomplished by the addition soda ash down stream from the first pond.

In 1982, the final three (of seven) shallow ponds were added. While the first four ponds were 2 each at 24 276 m^2 (6 acres) and 2 each at 36 414 m^2 (9 acres) in area, the three final ponds were each 20 230 m^2 (5 acres) in areal extent. The liquid in the final ponds is intended to be 30.5 cm (1 ft) deep. Actual summertime liquid temperatures in these ponds have been measured at 57°C (135°F) with liner temperatures in the 68 to 71°C (155 to 160°F) range. Bischofite crystals ($MgCl_2 \cdot 6H_2O$) saturated with an invariant brine are formed in the final ponds.

Other Design Constraints.

There were several other constraints that required consideration before establishing the final design criteria, not the least of which was economics. While prior installations of hot sprayed membranes had cost well under what a comparable synthetic rubber installation would cost, the past projects had not been as demanding in required chemical resistance and resistance to ambient environmental extremes. It was hoped that a product with much better performance characteristics could be installed with only a small incremental cost.

Another design constraint was brought about by the desire that the ponds be easily accessible to mobile equipment for maintenance and/or precipitate recovery. This required that the maximum slope gradient be 4:1 (run to rise).

An additional maintenance requirement was imposed specifying that the membrane to be in-service patchable.

Contemplating all the previous considerations, the final design criteria evolved:

- The liner system must be capable of hot spray application to 4:1 slopes and require minimum soil preparation.
- The liner must resist the ambient environmental extremes involved including membrane temperatures ranging from -18 to 76°C (0 to 160°F), winds to 0.254 + m/s (50 + mph), high intensity sunlight - especially the U.V. component.
- The liner must be able to resist reasonable seismic forces and displacements.
- The liner must adequately resist the expected chemical environment imposed by the leach liquor composition.
- The liner must have very low permeability and be capable of being installed leak-free.
- The liner must be able to withstand the stresses generated by operating heavy equipment directly on it as in maintenance operations or crystal harvesting.
- The liner must be in-service patchable.
- The liner must be economically competitive.
- The liner must have reasonable service life.

EARTHWORK.

The ponds were excavated using conventional earth-moving equipment. Water was added where necessary for dust control and compaction. Final contouring and grading were best accomplished with a motor-grader. Compaction was obtained with a vibrating roller (see figures 3 and 4). The actual degree of compaction was not critical, surface uniformity being the prime criterion. Of course, compaction was continued to the point that the surface would no longer yield under the wheels of heavy mobile equipment. The alluvial soil contained sufficient fines such that good compaction and smoothness were easily obtained.

LINER BLENDING AND INSTALLATION.

Liner material blending was done on-site by the manufacturer - installer in a semi-portable bulk

Figure 3. Earthmoving for a crystal harvesting pond.

blending plant. Raw materials for the blends were shipped to the site either in bulk or in palletized bags. Capacity of the bulk blending plant is approximately 8165 kg/hour (18 000 lbs/hour) which will cover from 3520 to 3763 m^2 (0.87 to 0.93 acre) in a two pass 2.29 mm (0.090 in) total thickness. This production rate just happens to be same as the hourly spraying capacity of the application truck.

Both the bulk blending plant and the application truck were custom built to the owners specifications. The blend tank is an insulated, trailer mounted tank heated through coils with heat exchange fluid from a separate, diesel fired heater. The tank is equiped with high shear turbine mixers and a high capacity pump.

The application truck (see figure 5) contains a heavily insulated 8896 L (2350 gallon) tank. This size tank was selected because it was the largest one that could be mounted on a single rear axle truck along with all of the other required equipment. The single rear axle truck was selected because it would cause less squirming under the tires than would result from a tandem rear axle truck when driving and turning on the geotextile or finished membrane. Other features of the truck include an auger mixer, diesel fired firetube heater with air gap, hydraulic drive for all application equipment, oversize pumps, 20.3 cm (8 in) pump suction lines, 3.66 m (12 ft) side boom in addition to the 4.88 m (16 ft) main spray bar. Many of the special features of the truck were required because of the very high viscosities involved, up to $8 \cdot 10^{-3}$ m^2/s @ 177°C (3500 SSU @ 350°F).

In practice, the geotextile is first placed on the soil base and then covered with two coats of the hot sprayed thermoplastic, elastomeric membrane. Either a 100% overlap or an alternating 50% overlap pattern can be used to build up the two coats. The slopes are usually finished first (see figure 6) followed by the bottom (see figures 7 and 8). The geotextile, which is a light weight non-woven nylon cloth on 2.54·2.54 cm (1 x 1 in) polyester reinforcing, serves several purposes. First of all, it keeps dust and soil moisture from causing a porous membrane. Secondly, it gives the membrane some dimensional stability in areas of high stress. However, if large movements occur as with seismic displacement, the geotextile is free to tear and allow the 1300% elongation capacity of the membrane material adjust to the new conditions and maintain the integrity of the liner. Lastly, the cloth gives the thermoplastic material on the slopes increased flow resistance during the summer and provides something to anchor the top of the slopes. The anchor originally was formed by keying the top of the geotextile in a trench around the top of the berm and burying it. The anchor is now formed by extending the geotextile 1/2 to 3/4 of the way across the roadway on top of the berm and covering it with 15 to 30 cm (6 to 12 in) of soil. Filling of the pond can start as soon as the spraying is complete.

While the two-coat application system usually provides a liner free of pinholes, occaisionally a few small leaks will be caused by one reason or another. These

Figure 5. Application truck.

Figure 4. Same as figure 3, showing compactor.

Figure 6. Applying second coat to a slope.

leaks have been very easy to detect by inspection or when filling a pond with a low pH solution if a carbonate soil is under the liner. These leaks have been successfully patched in depths of 1.22 m (4 ft) of liquid by using a section of large I.D. pipe or a barrel as a cofferdam, bailing and mopping the inside, drying and heating the membrane with a torch and pouring fresh molten material on the bottom.

After the ponds were filled to their operating level, the slope areas that remained above the water line were sprayed with a commercial grade of white Latex paint to provide protection from ultraviolet (U.V.) radiation, see figure 9 for U.V. cost application. Figure 10 shows a backhoe entering one of the 36 414 m (9 acre) ponds to harvest some crystals from the bottom.

Figure 7. Partially complete bottom.

Figure 8. Applying a bottom.

MEMBRANE FORMULATION.

Membrane No. 6 as manufactured and applied by the Deery Oil Co. of Mack, CO has a variable composition with specific properties tailored to specific end uses. The characteristics and properties of formulations for less severe service have previously been described (7), (8). However, the results of laboratory exposure tests consisting of boiling coupons of test formulations in samples of leach liquor for several days suggested that changing to a high density filler and adding carbon black would give increased chemical and U.V. resistance. Accordingly, several of the properties of formulations used on this project are different from those previously given:

Table 1. General physical properties of high performance membrane No. 6.

Test	Result
Specific gravity @ 15.5°C, ASTM D - 70	1.05 - 1.35
Viscosity, m^2/s @ 177°C, ASTM E - 102, $\cdot 10^{-3}$	2 - 8
Cone penetration @ 25°C ASTM D - 217	15 - 60
Softening point, $^{\circ}$C, ASTM D - 2398	82 - 110
Resilience @ 25°C, ASTM D - 3408	50 - 90
Flow, 70°C, 72 hrs., @ 75° angle, cm	0
Service temperature, $^{\circ}$C	118 - 93

Note: Other properties remain as previously reported.

In addition to the above, the preferred geotextile for this application is a 7g/m^2 spun bonded nylon on a

Figure 9. Applying U.V. protective coating.

2.54·2.54 cm polyester (500·500 denier) reinforcing. Milliken "Rockweave"® is a suitable product in the above grade as are others.

PERFORMANCE EVALUATION.

The first four leach liquor ponds have been in service since December, 1980; the final three since June 1982. Overall the performance of the membrane No. 6 liners in this service has been good to excellent - as is the current condition of the liners. Two items have been found, however, that need further improvement. First, on the south slopes, wind action has raised some of the nails and/or staples used to anchor the geotextile to the soil to the extent that the fasteners have penetrated the membrane. This only occurs above the waterline and the holes are easily puddle patched. An improved anchoring system or a system to dissipate pressure differential across the membrane would solve the problem. Secondly, the paint used to provide U.V. protection above the waterline lasts only 1.5 to 2 years before it must be reapplied. A longer lasting material that does not greatly increase the cost needs to be found.

The membrane system is approved for use in Nevada as a single layer liner provided pond liquid level monitoring devices and down-gradient monitoring wells are in place. In spite of the interpretation problems with the wells, no leakage from these ponds has been detected to date.

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Figure 10. Equipment entering a pond to recover salt.

Figure 11. Jan., 1984 view of evaporation pond.

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An Unprotected HDPE Liner in a Subarctic Environment

Two large brine storage ponds were constructed with clay liners in 1971-72. One pond exhibited leakage after four years of operation. A study was carried out to determine the sources of leakage, as well as causes, and mitigative action. Several factors contributing to the localized failure of the liner, included the existence of silt layers in the clay subgrade, desiccation and weathering of the liner due to lowered pond levels, freeze/thaw cycling and exposure to concentrated brine contributing to a secondary structure in the clay, and the burrowing of rodents within the dikes. Repair of the clay liner in the area of the seepage was effected as a temporary service. A study of the physical characteristics and costs of numerous alternatives resulted in selection of an unprotected high density polyethylene (HDPE) liner for the entire pond. Performance of this liner has been satisfactory. The second pond, completed and placed in service with a clay liner in 1972, still exhibits no sign of seepage.

A major oil company operates a gas fractionating facility near Ft. Saskatchewan, Alberta, Canada. In this operation, underground storage caverns are utilized to store the liquid petroleum gases produced at the facility. These underground caverns have been located in salt deposits, and hence the surface storage of significant quantities of highly concentrated brine, primarily sodium chloride, is necessary. In order to store this brine, two large storage ponds were designed and constructed immediately adjacent to the plant site. Each pond has a capacity of approximately 160 megalitres of concentrated brine. Each pond is about 120 metres by 300 metres in plan area (interior of top of berm).

An extensive geotechnical evaluation was carried out in 1971 to provide input to the design of the ponds. Initial design recommendations called for non-balanced cut and fill construction with pond bases being maintained above observed groundwater levels. This design included embankment sections constructed, to a relatively homogeneous configuration, of relatively impervious cohesive material excavated from the reservoir area. However, due to lack of sufficient suitable borrow on site, the owner opted for a balanced cut and fill design and construction approach. The pond bases were accordingly lowered to facilitate this.

Eight groundwater observation wells were installed around the perimeter of the two ponds, as a permanent monitoring system. Sampling and chemical analyses have been performed on a regular basis by the operator ever since, and the findings reported to Alberta

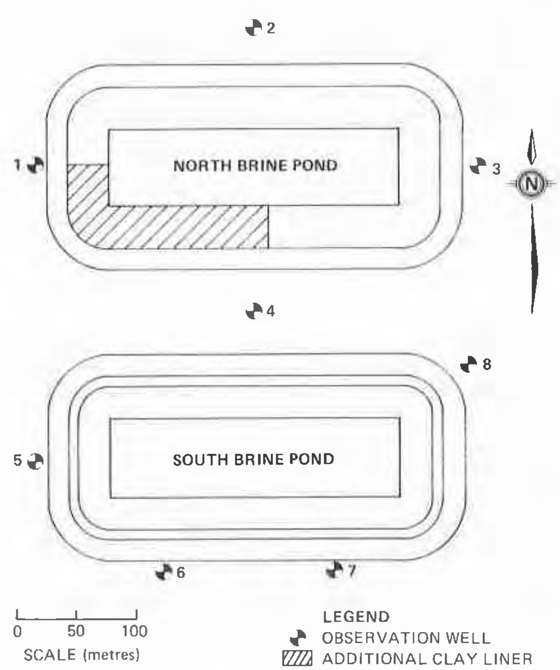

FIGURE 1 BRINE PONDS PLAN AND WELL LOCATIONS

Environment, the responsible regulatory agency. Figure 1 shows the plan of the ponds, and layout of these wells.

Construction of the ponds was halted for the 1971-72 winter, during which time the details for liner design were developed by the consultant and operator. The south pond was considered to be more of a concern, due to higher groundwater levels in this location. Consequently, the liner was constructed to be 1500 millimetres thick on the base, tapering to 750 millimetres at the top of the interior slope. The north pond liner was 750 millimetres thick throughout. Pond construction was completed in July 1972.

Operations were commenced immediately upon completion. The south pond was filled to its maximum level immediately, whereas the north pond was maintained at varying levels around the halfway point until fall 1976, at which time it was filled. Within a short time, an increase in salinity was noted in one of the observation wells (No. 2) adjacent to the north side of this pond. In fall 1977, a significant seepage zone was noted on the exterior face of the berm of the north brine pond, in the southwest corner. It appeared that this seepage had occurred in response to the raising of the pond level, a fact substantiated by the high salinity of this seepage water. The pond level was subsequently lowered to the previously maintained mid-level, and the seepage stopped. It should be noted that this was the only noted pond seepage from either pond. Similarly, only the one well ever showed a salinity increase, and no unusual "makeup" water was required for operations.

INVESTIGATION

An investigation of the problem was initiated by the consultant, at the operator's request, in mid 1978. This program was intended to examine in-depth, the nature of the seepage and its causes, as well as to provide recommendations for mitigative action. This included a review of the available literature, field and laboratory testing, and extensive examination of the pertinent data accumulated through operations.

The investigation concluded that the primary causes of the leakage related to the operating conditions of the pond. A fractured, blocky, secondary structure was noted to have become quite dominant within the liner materials, with definite vertical and horizontal fissures and fine fractures. Some minor voids were noted in samples recovered from the liner, presumably formed during the placement and compaction process.

The contributory factors were considered to be as follows:

1. The liner was placed under similar conditions in each pond. The south pond was, however, provided with a thicker base liner which has presumably contributed to higher quality retention capability.

2. The south pond has been maintained at the maximum level since commencement of operations. Drawdown for pressure injection of brine occurs in winter, however, so some cyclic freeze/thaw would occur. In the north pond, however, not only does this drawdown occur, but the pond was maintained at low levels for four years. The freeze/thaw, in conjunction with summer exposure, have greatly contributed to the secondary soil structure, in addition to significant surficial desiccation.

3. The brine is stored in near saturation concentration levels and has a freezing point of about -10°Celsius. It is, therefore, conceivable that under exposure to winter temperatures, the soil in the slopes and base could be subjected to freezing even when covered with brine. Conversely, however, due to geothermal heating of the brine, temperatures approaching 40°C are possible at the base.

Figure 2: View of pond in southeast corner. Note the wave- cut benches illustrating different pond levels, and desiccation of surficial clay.

4. Due to the lower and varying levels maintained in the north pond, the slopes could be exposed to concentrated brine, with some penetration, alternated with flushing with fresh water from snowmelt and precipitation.

5. Summer wetting and drying will also promote both erosion and desiccation, again creating a surficial pervious structure. See Figure 2. Wave action on the slopes created benching of the clay slopes.

6. No evidence of piping was noted in the south west corner where high levels triggered substantial seepage. What was apparent, however, was an extensive network of gopher burrows in the dike. It is possible that this could provide a contributory source for the degree and rate of leakage reported.

REMEDIAL WORKS

In view of the somewhat restricted seepage on an areal basis, and in view of a strong opposition on the part of the operator to take the north brine pond out of service for any length of time, a short-term repair measure was advocated, pending further study of potential solutions to the problem. Repair of the pond consisted of reconstruction and enhancement of the clay liner in the south west corner. This was carried out in the summer of 1979, utilizing locally (on-site) available clay. This program was not satisfactorily completed, however, as the quality of clay deteriorated rapidly with progression into the pit, becoming very silty and with significant silt layering, and hence the quantity required was not available. Rather than proceeding with off-site borrow, the remedial works were terminated incomplete. The area treated is shown on Figure 3.

After considerable discussion, the owner, under continuing pressure from Alberta Environment, decided to proceed with a "once and for all" solution. This decision resulted in a commission to undertake an evaluation of available materials, including a completely new clay liner, synthetic geomembranes, spray-on membranes, and other seepage mitigation techniques such as a cutoff wall.

Figure 3: View of south side of pond. Note darker area in southwest corner of remedial clay liner, untrafficable brine sludge on base of pond.

GEOMEMBRANE DESIGN CRITERIA

Selection of the appropriate geomembrane, and its incorporation into a design section which would provide a relatively maintenance-free facility was dependent, at least in part, on external criteria.

The rather harsh climate in the Ft. Saskatchewan area was a major concern, and a review of liners installed in similar facilities was undertaken. The maximum and minimum air temperatures vary from lows below -45°C to highs in excess of 35°C. Precipitation is low, and especially in an exposed location, snow cover is frequently nil. Periodic mid-winter thaws with highs of 10°C are common. In addition, due to geothermal heating of the brine, temperatures at the base of the pond in excess of 40°C are possible. Consequently, the liner is likely to be exposed to a temperature variation of over 80C°.

In general, the requirements for the replacement lining were identified as:

- resistance against the contents of the pond: concentrated sodium chloride brine, liquid petroleum gases (primarily butane), and fresh water;
- resistance against temperature fluctuations ranging from -45°C to 40°C. In addition, in winter, sudden temperature changes (either way) of 30C° to 40C° in as little as 24 hours are not uncommon;
- the ability to accommodate frost action by remaining relatively flexible during cold weather. It must be able to tolerate local tensile stresses due to differential movements across cracks;
- the lining should be ultraviolet (UV) stable if unprotected;
- it must be resistant to damage due to runoff water, wave action, and scraping of log booms;
- it must be properly anchored and/or of sufficient weight to resist high winds;
- it must possess the ability to tolerate burning gas spills, which are a possibility;
- resistance to uplift due to gas buildup below the liner; and,
- it must be protected from damage from animals, hail, or even vandalism.

LINER SELECTION

Given that the soils at the site are predominantly comprised of (relatively) impermeable clay soils, it was necessary to recognize that a clay liner remained a viable alternative. One of the major criteria for liner selection is the expected lifetime. Notwithstanding any other criteria for selection, estimated life expectancies of different liner materials are shown on Table 1. Although several other materials were considered in the overall study, discussion herein is restricted to those materials indicated in Table 1.

TABLE 1
MINIMUM LIFE EXPECTANCY OF LINER MATERIALS
(YEARS)

Liner Materials	Unprotected	Protected
Clay	< 7	10
Chlorinated Polyethylene (CPE)	20	40
High density Polyethylene (HDPE)	25	50
Polyvinyl Chloride (PVC)	< 3	20
Chlorosulfonated Polyethylene (Hypalon) (CSPE)	20	40
Oil Resistant Polyvinyl Chloride (Oil PVC)	< 3	50

As can be seen, the protection of a liner generally increases the life expectancy by a factor of at least two. In some cases, the benefits of protection of a liner are such as to change it from an impractical to a feasible alternate. This tends to reflect the fragility of some polymers in exposed conditions. Protection normally comprises soil cover.

In light of this, the CPE, Hypalon, and HDPE liners in an unprotected configuration seem to satisfy reasonable expectations for durability. In view of the problems experienced with the north brine pond in the first years of operation, and in view of the fact that remedial works were necessary, the clay liner really never was very desirable as a candidate, but was included in ballpark costing for comparative purposes.

Numerous other criteria are naturally included in the liner selection process, as cited previously. Table 2 summarizes these physical and chemical considerations which input to the evaluation and selection of the liner. It should be noted that many of these comparisons are purely qualitative, and have been constructed from the technical and product literature.

Prior to the selection of the most desirable liner system, costing of the various alternatives was carried out. It should be noted that this paper presents the findings from a selection of the alternatives considered in the study. In all, eleven different systems were considered in both protected and unprotected modes. A total of twenty seven systems were priced by various contractors. In addition, spray on systems were considered but shall not be discussed herein. Cutoff and slurry trench walls were also considered.

Table 3 presents the results of a comparative cost assessment of the alternatives. The last entry in Table 3 is a composite of two polymers which was recommended as an alternative. In addition to it, the HDPE liner and oil resistant PVC liner were presented as recommended alternatives. In view of all of these considerations, the high density polyethylene (HDPE) geomembrane was selected for use by the owner.

DESIGN AND CONSTRUCTION

The detailed design of the pond liner was carried out subsequent to the geomembrane selection. It was naturally desirable to retain the existing configuration to avoid unnecessary earthworks. The slopes on the inside of the pond were therefore retained at an angle of 3 horizontal to 1 vertical (3:1). This did, as a result, present one of the constraints on the liner, and was consequently considered in the selection process.

The major consideration with regard to adaptation of the existing pond to a suitable condition for construction of the liner, was the treatment of the sludge which had accumulated in the bottom of the pond (see Figure 3). In the existing (after draining the pond) state, the base of the pond was untrafficable, and some treatment of the sludge was necessary. After a program of laboratory testing had been carried out, it was determined that mixing a cement mixture into the sludge optimized the strength gain with time. Subsequent to this treatment, access of equipment to the base of the pond for construction of the liner system would be possible.

The criteria for the design of the pond are the requirements for interception of any seepage from the pond, and collection of this fluid for replacement in the pond. Similarly, prevention of gas buildup below the liner is a necessary requirement. A suitable venting system was therefore incorporated below the liner.

The top 300 millimetres of the pond base was treated with the cement mixture, and after a relatively short period, the base was capable of sustaining traffic. A non-woven polyester geotextile was placed directly onto the pond base, and covered with a graded sand base course. The drainage and vent pipes were placed in the sand blanket, and the liner finally placed on top. This sequence is illustrated on Figure 4.

Figure 5 illustrates the placement of the sand bed onto the geotextile prior to completion of installation of the ventilation and seepage collection systems. Figure 6 shows the liner in-place. An aerial view of the two ponds is presented on Figure 7, taken in February 1984.

CLOSURE

In this case, the failure of the clay liner does not necessarily indicate the unsuitability of clay liners as a viable alternative for pond linings. Even though geomembranes are considered to be the preferred means of retaining these fluids, in certain circumstances, owners can elect to proceed with the substantially cheaper alternative. The onus is squarely on the designer, however, to identify the constraints.

Clay liners do tend to leak. However, the nature of the contained fluids clearly dictates the degree to which leakage can be tolerated. For ponds containing toxic materials, the risk of a clay liner leaking is unacceptable, in our opinion. In the subject case, for the original design, the lower cost and higher risk of leakage was an acceptable alternative, especially in view of the faithfulness with which the operator regularly monitored the wells on site. Reaction to evidence of the leakage was timely, and measures could be taken which alleviated the problem prior to any damage.

The selection of the geomembrane for the north brine pond was a consequence of the desire to minimize the time the pond was out of service, optimize costs, and respond to the need for a positive solution to a lingering problem. It is possible that at some point in time a similar decision will have to be made for the south brine pond. The twelve years of operation of the south pond, without evidence of leakage is probably pushing the operational life expectancy of a clay liner, in this environment.

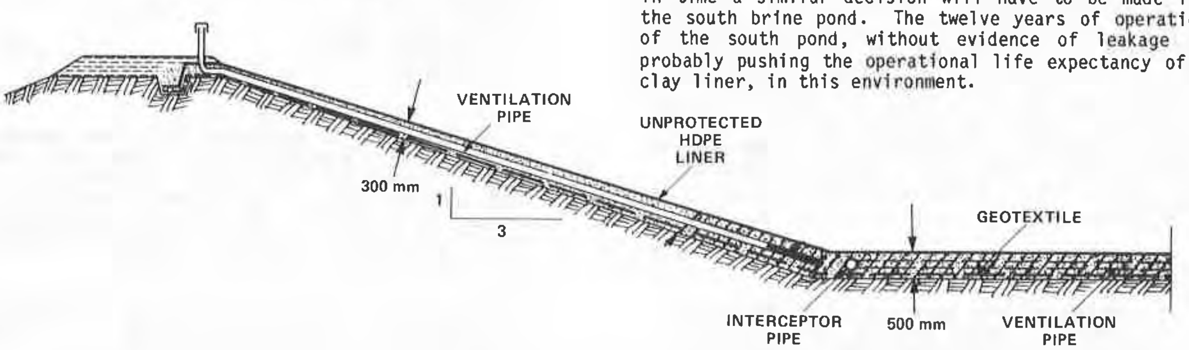

FIGURE 4 CROSS SECTION OF LINED POND

TABLE 2: COMPARISON OF LINER MATERIALS

	CLAY	CHLORINATED POLYETHYLENE (CPE)	HIGH DENSITY POLYETHYLENE (HDPE)	POLYVINYL CHLORIDE (PVC)	CHLOROSULFONATED POLYETHYLENE (HYPALON)	OIL RESISTANT POLYVINYL CHLORIDE (OIL PVC)
A. OVERALL HYRDOCARBON RESISTANCE	Good	Fair - Poor	Good	Fair - Poor	Poor (*)	Excellent - Good
B. MECHANICAL PROPERTIES						
Puncture	Poor	Good	Excellent	Excellent - Good	Good - Poor	Excellent - Fair
Tear	N/A	Good	Excellent	Excellent	Good	Excellent
Elongation	N/A	Excellent	Excellent	Excellent	Excellent	Excellent
Abrasion resistance	N/A	Good	Excellent	Good	Good	-
C. OVERALL WEATHERING RESISTANCE	Poor(*)	Good	Good	Poor(*)	Good	Poor(*)
D. THERMAL CHARACTERISTICS						
Low temperature flexibility	N/A	Good - Poor(*)	Good	Fair - Poor(*)	Excellent - Good	Fair - Poor(*)
Dimensional stability	N/A	Good	Good	Excellent	Poor (*)	Excellent
Minimum field handling temp.	5°C(*)	-12°C(*)	-18°C(*)	-10°C(*)	5°C(*)	5°C(*)
E. OVERALL FIELD SPLICING	N/A	Good - Poor	Good	Good - Poor	Excellent	Good - Poor
Solvent	N/A	Excellent	Good	Excellent	Excellent	Excellent
Heat	N/A	Poor(*)	-	Poor(*)	Good	Poor(*)
Adhesive	N/A	Good	-	Good	Good	Good
Minimum field seaming temp.	N/A	-7°C	10°C	-7°C	-7°C	5°C
F. COEFFICIENT OF PERMEABILITY (m/s)	10^{-10}	10^{-14}	-	7×10^{-15}	3.6×10^{-14}	10^{-14}
G. LIMITING SLOPE ANGLES (vert:horiz)	3:1	2:1	vertical	1:1	1:1	1:1
H. RELATIVE COST	Low	Medium	High	Low	High	Medium

(*) CRITICAL FACTOR GOVERNING USAGE — RELATIVE SCALE: Excellent-Good-Fair-Poor

N.B. The relative suitability of each category is a qualitative assessment of the generic product, based on the literature available at the time. This should not be considered as either a condemnation or an endorsement of any product for any particular use.

TABLE 3: COMPARISON OF LINER COSTS

	PROTECTION	LINER THICKNESS SLOPE	LINER THICKNESS FLOOR	CONSTRUCTION COST*	RELATIVE MAINTENANCE COST	MINIMUM EXPECTED LIFESPAN (YRS)
Clay	none	0.75 - 1.5 m	1.5 m	1.00	High	< 7
	0.7 m sand	0.75 - 1.5 m	1.5 m	1.56	High	10
Chlorinated Polyethylene (CPE)	none	36 mil (reinforced)	36 mil (reinforced)	2.42	Medium to High	20
High Density Polyethylene (HDPE)	none	80 mil	60 mil	2.67	Medium	25
Polyvinyl Chloride (PVC)	0.7 m sand	20 mil	20 mil	1.82	Medium to High	20
Chlorosulfonated Polyethylene (Hypalon)	none	36 mil (reinforced)	36 mil (reinforced)	2.88	Medium	20
Oil Resistant Polyvinyl Chloride (Oil PVC)	0.7 m sand	30 mil	20 mil	2.00	Medium to High	50
Polyvinyl Chloride/ Chlorinated Polyethylene (Reinforced) (PVC/CPER)	0.7 m sand	36 mil (CPER)	20 mil (oil resistant PVC)	2.22	Medium	20

* Costs shown are relative to the cost of the clay system, unprotected. Costs were based on non-competitive pre tender estimates by suppliers/contractors.

Note: 1 mil = 25.4 microns

Figure 5: Placement of sand bed onto geotextile, and installation of ventilation and collection piping.

Figure 6: In-place high density polyethylene (HDPE) liner.

In general, given the availability of a wide variety of specific-application geomembranes, and the increased awareness of designers to the environmental consequences of liners which leak, the use of clay liners as an acceptable alternative for storage of toxic and even innocuous fluids, will decrease. Hopefully, the transition from soil liner to synthetic liner will not result in the neglect of the geotechnical engineering component of pond design. If

Figure 7: Aerial view of brine ponds (February 1984)

it does, there could be non-liner failures of ponds due to this neglect, with potentially more catastrophic consequences than "mere" leakage.

Acknowledgement

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Floating Reservoir Cover Designs

Over the years various designs for Floating Reservoir Covers have been used with variations in performance characteristics. This paper presents an overview of the evolution and state-of-the-art in the design features of Floating Reservoir Covers. Various design features appropriate for inclusion in a floating reservoir cover design are presented, and descriptions of the effects they have are covered. Reservoir geometry and configurations relative to the design approach appropriate are described. Functional considerations are covered.

Floating cover designs started with the simple deployment of a sheet of impervious sheeting over a body of water. These early trials were generally considered unsuccessful in that the slack produced by the sheeting as the reservoir went from an empty state to a full state would wrinkle randomly all over the reservoir and would not allow for any means of rainwater collection and disposal. Obviously, there could be no access to the top of this cover when in operation, as the slack immediately would be drawn toward the weight of the individual traversing the cover and he would be immediately enveloped in the slack of the floating cover - obviously an extremely unsafe situation.

SLACK ACCOMMODATION

In virtually all floating cover designs, the most significant design issue is how to handle the slack produced in the floating cover membrane as the cover rises on top of the surface of the water, from an empty reservoir state to a full reservoir state. In a slope-sided reservoir, this is commonly known as "slope-allowance." It may be geometrically described as the difference between the horizontal projected distance (or width) of a slope and the slope-distance of that slope (the hypotenuse of the triangle formed by the horizontal projected distance, or width, of the slope and the depth of the reservoir). This "slope-allowance" is the extra material which forms slack, and which must be dealt with by the various floating cover design approaches.

In a vertical wall reservoir, this slack is equivalent to the height of the reservoir walls or the depth of the reservoir.

UNTENSIONED, CENTRALLY FLOATED, PERIPHERAL SUMP RESERVOIR COVER

A material improvement in the evolution of floating cover design occurred when Globe Linings Inc. of Long Beach, California patented and started installations of a new design of reservoir cover. This design may be briefly described as an untensioned, peripheral sump floating cover. This design deploys a sheet of flexible impervious material anchored at the periphery of the reservoir, cut and installed to fit the reservoir when it is in an empty condition. In the central portion of the reservoir, usually the entire bottom of the reservoir, a series of parallel floats, emanating from a central, perpendicular float are arranged beneath the cover and bonded to the bottom side thereof. A typical reservoir of this type, which I shall call "Untensioned, Centrally Floated, Peripheral Sump Reservoir Cover" (UCFPSR Cover, hereafter), is shown in Figure 1 and Figure 2. Figure 1 is a plan-view of a sloped-sided rectangular reservoir which employs such a cover. The network of floats beneath and attached to the cover are indicated by the bold lines in the center. Usually the central float is placed down the longest axis of the reservoir, with the parallel floats running perpendicular from the central float, spaced 20 to 30 feet apart. It is intended that this central portion will form a series of parallel paths which direct rainwater to the peripheral space outside of the centrally-floated area and inside the perimeter of the reservoir. This is intended to form a peripheral sump between the central float portion and the anchor means at the perimeter of the reservoir.

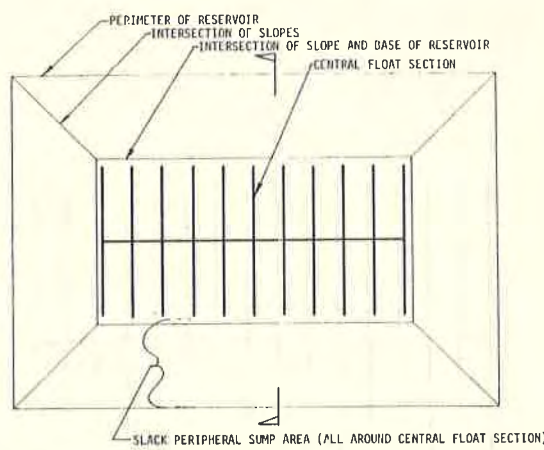

FIGURE 1

Figure 2 is a cross-sectional view through this reservoir, showing in the bottom view the UCFPSR Cover with the reservoir empty, and in the top view and middle view the UCFPSR Cover with the reservoir full. In this view, it is apparent that the slack formed in the cover membrane as the reservoir cover goes from the empty state to the full state, due to "slope-allowance," provides the extra material to form a rainwater sump between the centrally floated portion and the perimeter anchor means.

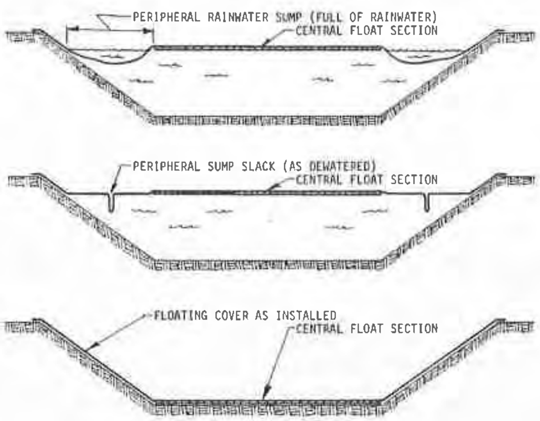

FIGURE 2

In theory, this design was meant to form a common rainwater sump all around the periphery of the centrally floated portion which would facilitate the collection and removal of rainwater from the top surface of the floating cover. The spaces between the floats were intended to form paths directing the rainwater falling on the central portion to the peripheral rainwater sump portion. The top view of Figure 2 shows the reservoir cover with the peripheral sumps substantially full (the rainwater not removed from the sump area). The middle view shows the same cover, but with the rainwater substantially removed from the peripheral sump portion. Although, for purposes of illustration, the location of the slack forming the rainwater sump is shown as indicated, it is possible that this slack could form folds at any location between the perimeter of the reservoir and the central portion which has floats.

Figures 3 and 4 show a similar UCFPSR Cover utilizing a central grid of floats, much as described above, in a vertical wall reservoir configuration. Figure 3 is a plan-view of that reservoir. The typical layout of floats in this system is shown. It is noted that they project out, to within a few feet, of the walls of the reservoir. Figure 4 is a cross-sectional view, showing various states of the reservoir from empty to full. The bottom view shows the reservoir and floating cover with the reservoir empty. The middle view shows the reservoir

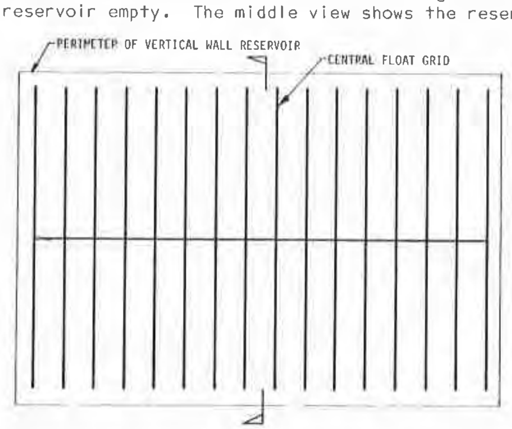

FIGURE 3

with the reservoir full and the rainwater sump dewatered. The top view shows the reservoir cover when the reservoir is full and the rainwater sump has not been dewatered. It is noted that the slack produced from the side-wall material, as the reservoir cover goes from the empty state to the full state, hangs essentially adjacent to the wall of the reservoir and acts in a rolling diaphragm type of response.

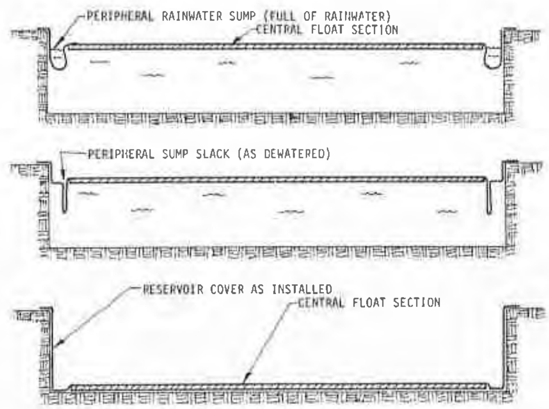

FIGURE 4

For many years this basic design approach was the preferred system of choice for floating reservoir covers as it materially improved the state of the art and created a functionally acceptable floating cover where none previously existed. Most of the early reservoir covers of this design were made of a vulcanized, synthetic rubber sheeting which was extremely difficult to seam in the field. Additionally, the factory fabricated panels available at that time were quite small by today's standards; i.e., typically 22 Ft. by 100 Ft. This necessitated a large amount of field seaming per unit surface area covered. These field seams were best described as post-vulcanized bonds, utilizing agressive bonding adhesives. True vulcanized field seams or welds were not possible with this type of material, therefore the seams were difficult to make and subject to potential disassociation over a period of time. These post-cure adhesive bonds were particularly susceptible to disassociation in the presence of flexure or stress. It was the state-of-the-art regarding the then available, most-favored-materials used in the early evolution of the floating covers that predominately dictated the need for an unstressed, untensioned floating cover.

This UCFPSR Cover had some drawbacks. The membrane between the perimeter and the central section was completely slack. This slack caused several undesirable conditions to exist. An individual could not go out onto the surface of this floating cover safely. If an individual should attempt to walk on the surface of the cover in the peripheral area, the slack in that area would immediately be drawn to the weight of the individual, which would cause him to effectively sink into the folds of the slack of the cover. This was an extremely unsafe, dangerous potential situation.

Another problem caused by the slack peripheral area was that in the presence of high winds, the central portion, which had floats connected beneath the cover, would be blown toward the leeward side of the reservoir. This would remove all slack (which formed the rainwater sump) from the windward-side of the central portion and force that extra slack into the leeward-side of the central portion. This would disrupt and randomize the original peripheral rainwater sump area. There would effectively be no rainwater sump on the windward-side area and in the spaces between the windward and leeward-side, the cover would typically form random, disconnected folds which were not any longer a common, peripheral sump. This materially detracted from the ability to remove rainwater from the sump area with a common rainwater removal pump or system. If the rainwater removal suction hose or pump was located in the windward area, of course it would be "high and dry" and all of the rainwater sumps would be on the opposite side of the reservoir.

Because no access could be safely gained to the surface of the reservoir it was virtually impossible to perform routine inspection,cleaning, maintenance or repair of the floating cover without taking the reservoir out of service. Access could be laboriously and precariously made to the central float section utilizing float pads and/or gangway planks in an appropriate manner; however, once on the central portion, the only areas that could actually be traversed safely was the top of the floats. If an individual should walk within the space between the floats, his weight would be sufficient to draw the peripheral slack toward him and he would, again, soon be enveloped in the folds of the slack of the reservoir cover.

Another negative factor which became apparent was that it was not uncommon for wind-driven rain to form large puddles, which would collect along the central float section and between the floats radiating therefrom, because there was no driving force to cause these puddles to move to the peripheral sump area. It was not uncommon for these puddles to remain as large, deep, undewatered areas, even after the peripheral rainwater sump had been dewatered to the best degree possible.

Another problem which would occur with this UCFPSR Cover was that wind-driven debris, dirt, sand, etc. would blow on to the top of the cover and collect in one or more local spots in the peripheral sump area. This would cause the slack to be drawn to the area(s) weighted with this debris. This would cause the peripheral sump to be broken up into several separate disconnected sumps. Again, this makes it very difficult to dewater the surface of the cover of rainwater.

FLOATING COVER MATERIAL DEVELOPMENT

In 1968 a new type of geomembrane was developed which was based on a synthetic rubber, produced by Dupont, known as Hypalon , a commonly known flexible membrane material today. After several years of product development incorporating reinforcing scrims, the first use of this material in a floating cover project utilizing above described UCFPSR Cover design occurred in early 1973 in the Alcoa, Tennessee floating covers.

Certain very salient improvements in the nature of the membrane and its suitability for floating cover use were observed. The ultimate configuration of membrane developed for floating cover used was a five-ply construction of membrane consisting of 3 plys of Hypalon synthetic rubber inter-layered with 2 plys of reinforcing scrim, totaling 45 mils in thickness. Flexures, and wrinkling which occur in a floating cover membrane in operation, dictated that the material should be principally a highly flexible, rubber sheeting material as opposed to being a cloth material.

Another salient advantage of this new membrane was that it was produced and deployed in an unvulcanized state; it was essentially a thermoplastic elastomer form of Hypalon synthetic rubber. This allowed the use of heat or solvent welds in both the factory and in the field, which would produce completely homogenous seams, containing no dissimilar material, and which could be considered as a monolithic material, containing no seams, once thus joined together. The seams would no longer be a "weak link" in a floating cover, as they were now as strong as the parent material itself. This superior material became the material of choice for subsequent UCFPSR Covers as described above for many years thereafter.

DEFINED SUMP RESERVOIR COVER

The special qualities available in the newly developed materials, together with the inherent problems of the then-current design approach, invited consideration of new and improved design methodology which would overcome these problems, and function more predictably and reliably. What evolved was a reservoir cover design which was a "Defined Sump Reservoir Cover" wherein the rainwater sumps were caused to be defined by the judicious placement of flexible, segmented weights and floats in geometrically considered configurations so that the rainwater sumps would occur at precise locations and the rainwater sumps thus formed would be precisely defined as to their location and size. Figures 5 through 8 show a classic form of this reservoir cover design system deployed on a slope-sided rectangular reservoir. The various configuration elements are drawn slightly over-sized or out-of-scale in order to demonstrat the design principles involved. Figure 5 is a plan-view of the reservoir when it is empty. Figure 6 is a section-view through the middle of the reservoir when the reservoir is empty. Figure 7 is a plan-view of the reservoir when it is full. Figure 8 is a section-view through the middle of the reservoir when it is full. The bottom view shows the rainwater sump in a dewatered condition and the top view shows the rainwater sump containing rainwater (before dewatering).

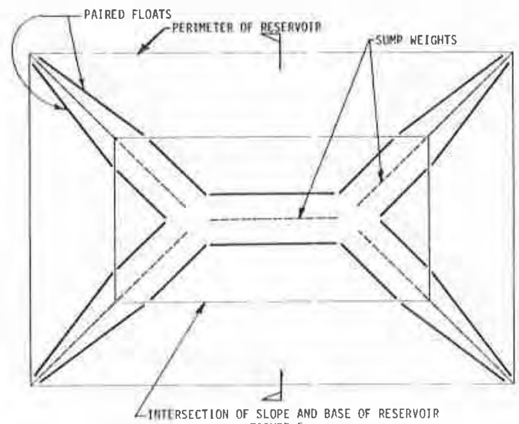

FIGURE 5

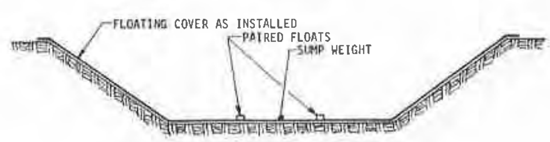

FIGURE 6

In Figure 5 the floats are denoted by bold lines and are flagged accordingly. The flexible segmented weights are denoted by a dashed line and are also flagged accordingly. The weights run down the four corners of the slopes of the reservoir and project out into the center of the reservoir from each corner. There is a line of weights connecting these two intersection points running down the middle of the reservoir. Typically these weights are made of sand filled tubes of the same flexible material as the cover. These weights are attached to the top side of the reservoir with straps of the same material as the cover. The floats are generally encased in the same material as the floating cover and may be either installed on top of the floating cover or beneath the floating cover. The weights are centered in between a pair of floats. This is often referred to as a paired-float/weight system.

As water is introduced into the reservoir beneath the floating cover, the cover is lifted off of the floor of the reservoir as are the segmented weights. The segmented weights create a discrete, low level tension in the floating membrane which causes a localized sump to form. As the "slope-allowance" provides more and more slack as the reservoir gets fuller and fuller, the depth of the sump continues to increase using all of the slack provided by the "slope-allowance". The slight tension produced by the weights in the floating cover membrane continuously works to cause the horizontal surface portion of the reservoir cover to achieve an absolutely flat surface. As rainwater falls on the surface of the reservoir, it forms a thin layer of an even thickness on the horizontal planes of the reservoir cover. This water fills in the space between the floats, the rainwater sump, as shown in Figure 8. A common rainwater removal means such as a sump pump or suction hose from a self-priming pumps is deployed in the rainwater sump area to remove the rainwater as it collects in the rainwater sump.

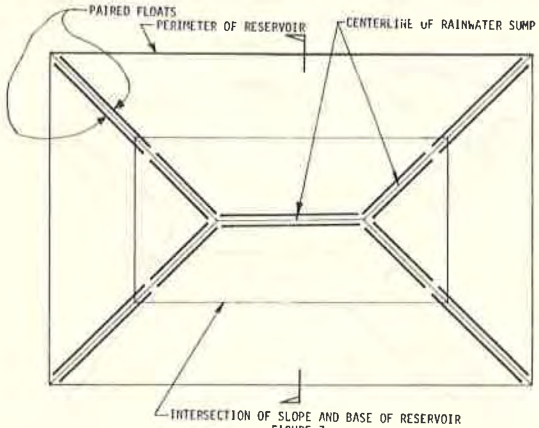

FIGURE 7

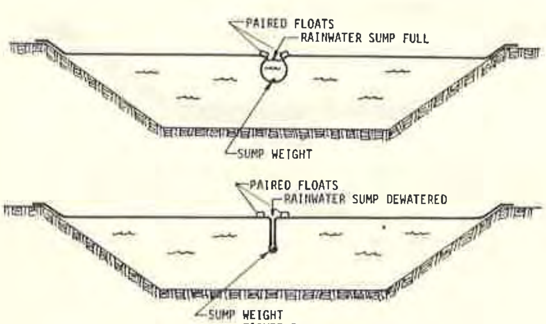

FIGURE 8

In 1976 a patent was issued covering the "Defined Sump Reservoir Cover." Since that time many reservoirs have been installed which embodied the "Defined Sump Reservoir Cover" design approach. A few of the reservoir covers which have employed this system are as follows: Hinkle Reservoir, Folsom, CA; Manchester Water Works (3), Manchester, NH; P. T. & O. H. Reservoirs, County of Ventura, Ventura, CA; City of Clifton Reservoir, Clifton, CO; Bingham Water Dist. Reservoir, Bingham, ME; P. G. & E. Reservoir, Tuolomne, CA; Cayman Water Reservoir, Cayman Islands; U. S. Dept. of Energy Reservoir, Fenton Hills, NM; Thompsonville Reservoirs (3), Thompsonville, CT; Mill Rock Reservoir, New Haven Water Works, Hamden, CT.

By having rainwater sumps, with weights in them, projecting up the slopes of the reservoir at the intersections of the slopes as shown, tension in two directions in the horizontal planes of the reservoir cover is achieved. By having tension in two directions in these horizontal planes or plates of the floating cover, the cover is sufficiently stable for a workman to walk out onto the floating cover. It is possible to walk anywhere on the surface of the Defined Sump Reservoir Cover. This access capability overcomes the limitations of the previous reservoir cover designs described. This access capability makes the reservoir cover complete accessable for routine inspection, cleaning, maintenance and repairing. In addition, the tension in the reservoir cover, which is always working to produce an absolute flat surface, prevents the formation of any wind-driven puddles (other than very thin, small, incidental puddles, a fraction of an inch in depth) from forming.

This Defined Sump Reservoir Cover absolutely fixes the location of the sump and is resistant to movement by the wind. As long as the floats and weights are properly attached to the cover, there is no translational movement of the sump in the presence of high winds. There is no disruption or discontinuation of the rainwater sump due to winds. All of the drawbacks present in the Untensioned, Centrally Floated, Peripheral Sump Reservoir Cover are overcome by the Defined Sump Reservoir Cover.

In Figure 6 it may be noted that during installation the floats are placed apart a predetermined distance equivalent to the slope allowance of both sides of the reservoir plus an additional amount to provide the desired width of the rainwater sump when the reservoir is full. If the floats were placed too close together, they would bump together as the reservoir became full, thus negating the rainwater sump formation characteristics of the cover. As can be seen in Figure 7, the floats which were spaced apart during installation have drawn close together as the reservoir has become full. A comparison of Figures 5 and 7 will indicate the change in configuration of the floats as the reservoir progresses from the empty state to the full state. In the top view of Figure 8 the shape of the rainwater sump may be seen when the rainwater sump is effectively full of rainwater. In normal operation, this degree of rainwater sump fullness is never achieved, in that automatic dewatering systems are commonly employed which prevent the rainwater sumps from ever becoming full.

Figures 9 through 12 show a use of the defined sump reservoir cover on a vertical wall, rectangular reservoir. Figure 9 is a plan-view of the reservoir when it is empty. Figure 10 is a plan-view of the reservoir when it is full. Figure 11 is a section view of the middle of the reservoir when it is empty and when it is full. Flexible, segmented weights are also deployed, running vertically, in each of the corners of the reservoir.

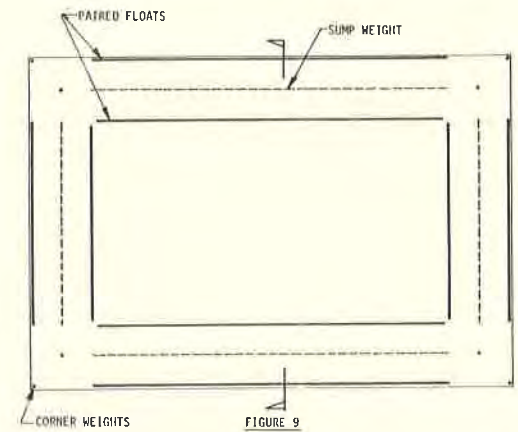

FIGURE 9

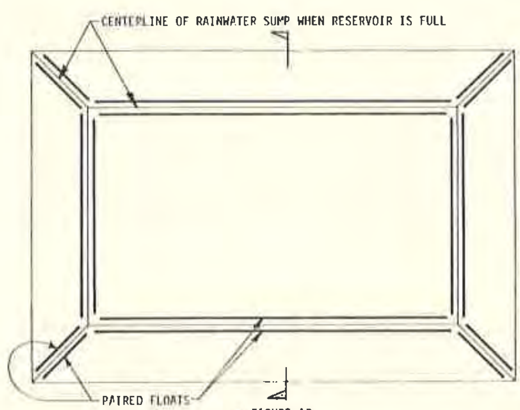

FIGURE 10

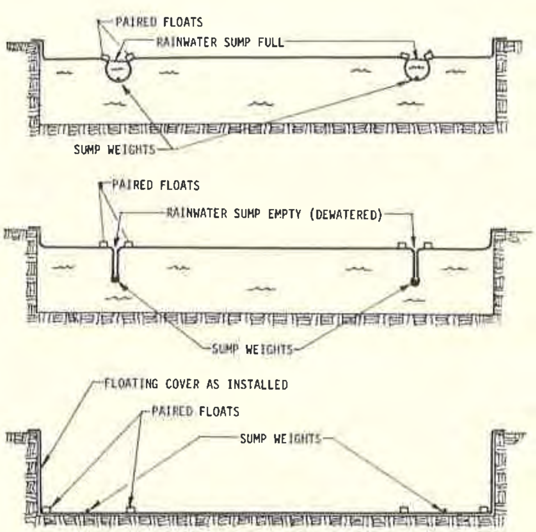

FIGURE 11

Although the judicious placement of weights on a reservoir cover will define the location of rainwater sumps, in their own right, leaving off the paired floats on either side thereof can cause a potential unsafe condition. Without the floats, as rainwater enters the sump area, the sides of the sumps will be an ever-increasing-in-width radius projecting up from the sump weight. Anyone approaching a rainwater sump without paired floats which was filled with rainwater would be likely to slip into the sump due to the shape factor of the rainwater sump and the extreme slipperiness of the floating cover material when wet. Furthermore, it is not possible to visually see where the rainwater sump physically starts, or stops. Figure 12 is a cross-section view of an undewatered rainwater sump which does not have paired floats on either side thereof. It can not be over emphasized that the extreme slipperiness of the floating cover material, when wet, is such that an individual slipping into such an undewatered sump may very well not be able to crawl out of such a sump.

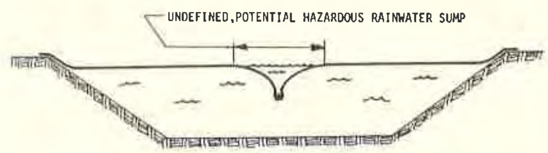

FIGURE 12

DESIGN FEATURES

Perimeter Attachment

By far the most common form of attachment at the perimeter of the reservoir is the simple use of a batten bar which clamps the reservoir cover to the edge of the reservoir. Normally a rope hem is included in the edge of the reservoir cover to further insure against pullout. Proper edge burial in an anchor trench around the perimeter of the reservoir is also an acceptable method of perimeter attachment.

Water Inlet Provisions

Where water will be discharged into the reservoir, it is common to provide some form of accommodation to prevent high velocity water from impinging directly on the bottom surface of the floating cover during the filling or normal operation processes. These have taken many forms, including deflector plates, stand-off frames, an extra layer of floating cover material attached to the underside of the cover in the impingement area, extra thick floating cover material in the impingement area, and the judicious use of flotation material in the impingement area. Various combinations of these options have also been utilized.

Water Outlet Provisions

It is very important that provisions be made to prevent the reservoir cover from being drawn uptight or "sucked into" the water outlet of the reservoir, particularly as the reservoir is emptied. This requires two special design provisions. The reservoir cover must be physically prevented from closely approaching the reservoir outlet fitting or sump area and the area affected should be vented to break any vacuum which may form in that area.

Overflow Provisions

Reservoirs are normally fitted with overflow provisions which must be accommodated by the floating cover. Depending on the type of floating cover design selected, these provisions may include the need for stand-off frames to physically hold the cover away from the overflow area, or the simple use of flexible stand-off flotation members may be sufficient.

Chafing Buffer Provisions

Normally, the area contiguous to the perimeter attachment, where the cover enters the reservoir, is provided with a chafing buffer pad beneath the cover where it is in contact with the reservoir; this protects the cover from chafing damage. Any reservoir projections when the cover may contact repeatedly should be "softened" as necessary and thoroughly protected with a chafing buffer provision. This is often accomplished with the geotextiles or geomembranes.

Access Hatches

Reservoir covers are normally provided with access hatches. Generally these are prefabricated hatches which are mounted on flotation means at prescribed locations on the floating cover surface. Generally these hatches are placed at strategic locations over sumps, valve areas, etc. within the reservoir which may require access at some future time. It is common to place an access hatch directly over the water outlet area of the reservoir so that this hatch may be opened during the final draining of the reservoir to prevent any vacuum from forming in the area. Some reservoirs have been fitted with extra flotation leading from the perimeter of the reservoir out to the location of the hatches. These are called hatch access path floats and, although they are not absolutely required, do offer certain advantages as to cover access, convenience, protection, and stability. Where very large areas containing equipment or the like must be accessed, very large hatches have been designed utilizing a simple batten bar principle located and attached on top of flexible flotation means.

Rainwater Removal Means

Rainwater removal is commonly provided by the simple use of a suction hose in the floating cover reservoir sump leading to a self priming pump on the edge of the reservoir. A preferred method is the simple deployment of a sump-pump type pump in a small plastic tank, fitted with flotation means and located in the rainwater sump area. This assembly must be made without projections or features which could damage the floating cover. Another method for removal of rainwater from the sumps is the use of a submersible pump in an appropriate water collection piping and receiving network.

Another preferred form of rainwater removal, when such adaptation can be made, is a self-draining provision. This system is most appropriately fitted to a Defined Sump Reservoir Cover in that the reservoir cover rainwater sumps form in an absolutely predictable, recurring position at all times. This system was first developed at the Hinkle Reservoir in Folsom, California. In this system, the bottom of the rainwater sump is fitted with penetration fittings and collection piping to which flexible hoses are connected beneath the cover. These flexible hoses lead to the base of the reservoir where they penetrate the floor of the reservoir leading to drainage piping. The flexible hoses connected between the drainage piping and the bottom of the floating cover rainwater sumps allow the reservoir cover to rise and fall in normal operation with no restriction and continuously drain the rainwater sumps of the reservoir as rainwater is collected therein.

Reservoir Structure Provisions

Many times there are one or more large projecting structures existing in a reservoir which must be accommodated. There is no prescribed common method for handling these, as each case must be considered on its own as a special design project. Generally, the judicious placement of flotation means or special structural provisions or modifications will accommodate these requirements. Consideration of handling the excess slack produced by these projecting structures as the reservoir goes from empty to full is the central design issue which must be considered.

Inflation Provisions

Generally it is not necessary or advisable to inflate a floating cover. There are, however, exceptions to this and, with proper design and safeguards, certain floating covers can be inflated temporarily for cleaning and maintenance beneath the covers. Where this is anticipated, pre-installed inflation fittings may be provided on the floating cover to which blowers can be connected for inflation. Additionally, consideration of access to the cover in an inflated state would be made and access hatches of appropriate design be provided at appropriate locations. Ladders or steps on the surface of the reservoir beneath the cover may be useful or necessary.

Precautions against damage to the cover or injury to personnel in the event of sudden high winds must be provided. Provision for the rapid "dumping" of inflation air in the event of sudden high winds should be provided. The cover should not be inflated any higher than absolutely necessary to accomplish the objective beneath the cover. The cover should not be "fully inflated," but inflated minimally to keep the profile as low as possible. Rather than the cover being inflated to some predetermined pressure, the cover should be inflated to some predetermined physical state, i.e., "x" number of feet off of the floor of the reservoir.

The stresses imposed on the floating cover during inflation can be significant and proper advice should be sought from qualified consultants regarding this matter. The Defined Sump Reservoir Cover which employs a pattern of weights on the surface of the cover is particularly well suited for inflation, in that the weights hold the reservoir cover down, stabilize it, and keep it in a low profile during inflation.

PATENTS

There are many, many patents which have been issued which are related to floating cover design and design features. No attempt can be made here to delineate or describe all of these patents. Information presented in this paper is not intended to advise the reader regarding patents or invite infringement of any existing patents. The reader is advised to make his own investigations as to the scope and content of existing patents relative to floating reservoir covers.

CONCLUSION

Obviously it is not possible to cover the entire field of floating reservoir cover design in a relatively short paper such as this. There is a great deal of floating cover design technology/know-how and there are a large number of design options available to the floating cover designer which can be adopted or not, depending on the site-specific conditions, the economic environment, and many other factors. This design information is readily available to engineering firms or departments who are designing floating covers. There is an on-going, continuing evolution in the state-of-the-art regarding floating cover design and their unique features. For example, a patent recently has been granted covering a subsequent improvement wherein sump forming weights are added in a geometrically considered manner, which will change the Untensioned, Centrally Floated, Peripheral Sump Reservoir Cover previously described into a Defined Peripheral Sump Reservoir Cover. Special design features for ice resistance are available. Special design features and materials are available for reservoir covers containing fluids other than potable water. Therefore the potential user should investigate the field as to the then-current state-of-the-art and specify those design approaches or features that are most appropriate for his needs.

KITTREDGE, DAVID
Manchester Water Works, Manchester, New Hampshire, USA

Performance of Flexible Membrane Floating Covers

Some of the first floating covers ever installed in the United States were at Manchester, N. H. in 1970. Three open water reservoirs used for distribution storage and owned by the Manchester Water Works (MWW) were covered with reinforced EPDM to reduce high levels of coliform bacteria and maintain chlorine residuals in the water system. Although design objectives were achieved, cover maintenance and repairs proved difficult and costly over the years due to excessive amounts of surface water, ice damage and vandalism. These conditions resulted in rapid deterioration of the EPDM covers and by 1981 new floating covers made of reinforced hypalon had to be designed and installed. These second generation floating covers have given very satisfactory performance during the first two years of operation. The MWW attributes this improved performance to basic improvements in floating cover design, material properties and operating procedures followed by the utility during winter months.

Founded in 1874, the Manchester Water Works is today the largest municipally owned and operated water utility in New Hampshire serving a resident population of 90,000 and a regional population of about 115,000 persons. Water quality at the consumer tap is considered to be exceptionally high at Manchester for several closely related reasons. One is the excellent quality of the City's drinking water source at Lake Massabesic where vast amounts of watershed land are owned and tightly controlled by the MWW. A second reason is the use of complete chemical pretreatment, plus sand filtration and carbon filtration which combine to raise the quality of treated water to a point which is well above minimum drinking water standards established by the U. S. Environmental Protection Agency. The final reason for production of high quality drinking water at Manchester is the existence of three covered distribution reservoirs with a combined storage capacity of 155,185 m^3(41 MG). Two of these reservoirs provide treated water storage for the High Service System of the City and are 64,345 m^3(17 MG) and 15,150 m^3(4 MG) in capacity. The third and largest reservoir provides treated water storage for the Low Service System and is 75,700 m^3(20 MG) in capacity.

Prior to 1970, these reservoirs were uncovered and the only method of treatment afforded to the City's drinking water was simple chlorination and corrosion control. This limited application produced a number of water quality problems, particularly during summer months when chlorine demand was high and aftergrowths of coliform bacteria could not be effectively controlled due to sunlight penetration and algae development within the open reservoirs. Thus, large quantities of chlorine in dosages up to 10 mg/L had to be applied at times in order to meet drinking water standards and, as a result, the esthetic quality of the drinking water became unacceptable to consumers much of the time. To counteract a deteriorating water quality and poor economy of treatment, the MWW made a decision in 1970 to cover its three open reservoirs by installing flexible membrane floating covers utilizing an innovative design developed by Globe Linings, Inc. of Long Beach, CA. These first generation type floating covers featured a 60 mil thick, nylon reinforced EPDM flexible membrane with tongue and groove joints bonded together in the field using a butyl rubber cement adhesive. Individual panels of EPDM 6.1 m (20 ft.) by 30.5 m (100 ft.) formed a rectangular grid shaped pattern on the empty reservoir bottom and foam floatation material was used beneath the panel joints to provide alignment and buoyancy for the cover when it was deployed in a floating position. The outermost perimeter float of the cover formed one side of a collection channel or sump for rainwater which, generally speaking, ran parallel to the exterior perimeter wall of each reservoir with about 9.1 m (30 ft.) of separation distance between the wall and float. An aluminized paint was applied to the black colored EPDM material to enhance its appearance and longevity and also, to provide for a cooler and more palatable water during summer months.

Soon after completion of the first floating cover in mid-1970, it became apparent to MWW maintenance personnel that excessive amounts of surface water were collecting on the cover due to the design configuration of perimeter floats. These floats created a barrier or small dam against natural runoff to the collection channel area of the covers which made surface water removal costly, time consuming and extremely difficult. (This perimeter float design was later modified by removing the end sections of floats, but unfortunately the remaining double layer of EPDM material still produced a ponding effect over much of the cover surface.) Portable gas driven float pumps were used to pump off the grid portion of the covers while a combination of electric submersible pumps and manually operated siphon hoses were used to drain the main collection channels. At this time, pumping operation at the three reservoirs required a crew of two or three men working at least three days a week over eight months of every year.

By the early winter of 1970-71, considerable amounts of surface water had collected at all three reservoirs and despite major pumping efforts the water soon turned into ice and dewatering operations had to be abandoned. It then became apparent as the winter progressed that ice on top of the cover was having a devastating effect on the cover membrane. This was particularly true at the two largest reservoirs where huge ice blocks would form around the inner perimeter of sloping walls due to water level variations of several feet per day. (See Figure 1). Ice also formed beneath the floating covers which produced a "shear effect" on the membrane as it became sandwiched between the upper and lower layers of broken ice along reservoir sidewalls. As a result, several large membrane tears were discovered at the Low Service Reservoir during this first winter of floating cover operation. These tears ranged from about 10 cm (4 in.) in length to one of over 6.1 m (20 ft.) which was found near the midpoint of the east wall. Additionally, numerous field seams were observed to have failed along the top section of floats due to a "buckling action" created by differential ice movement over the cover surface. Once the ice and snow had melted away the following spring (1971), an inspection was performed at the three reservoirs to assess the degree of winter damage. Over thirty separate areas were identified to be in need of repair. Fortunately, however, none of this damage was serious enough to warrant draining the reservoirs and repairs were completed in roughly a months time by the installation contractor and by MWW maintenance personnel.

During the summer of 1971, someone managed to penetrate a security fence at the large High Service Reservoir causing substantial damage to the floating cover by cutting it with a knife or other sharp instrument in several locations. One of the cuts formed a near perfect circle about 1.2 m (4 ft.) in diameter and was large enough to necessitate complete removal of the floating cover from service for repairs. It was at this time that repair crews experienced the extreme difficulty of performing repairs to a floating cover which had become partially submerged. Several innovative repair schemes were attempted, e.g., air inflation and use of SCUBA divers, but few ever had any measurable degree of success. It was found however, that repairs to sidewall areas of the covers could normally be accomplished satisfactorily by lowering the reservoir water level to a point where damage areas became accessible just above the waterline.

In spite of the relatively severe damage to the EPDM covers as a result of this first winter of operation, a substantial improvement in both water quality and economy of treatment was noticed during the months which followed. Average chlorine dosage, for the two years prior to installation of the covers (1968-1969) had measured 5.9 mg/L and 6.3 mg/L respectively. After covering the reservoirs, chlorine dosages and costs were reduced by approximately one-third during the first full year of cover operation in 1971. (At today's prices, the annual cost savings for chlorine would amount to roughly $23,200.) Fewer consumer complaints were also received after the reservoirs were covered due to the reduced chlorine concentration and reduced standard plate count (biological conditions) measured in the stored water. These reductions, in the opinion of the author, can be mainly attributed to the elimination of sunlight and airborne contaminants by the covers.

Experience with these first generation floating covers over the years which followed was very similar to that which has been previously described. However, as time progressed, the ability to make satisfactory repairs to the covers became increasingly difficult as patches frequently came loose and tear damage due to ice action increased accordingly. Over $12,000 in cover maintenance and repair costs were incurred during 1978 alone. The following year it became necessary to remove the floating cover from the large High Service Reservoir due to its deteriorated condition and install temporary chlorination equipment for prevention of bacterial aftergrowths in the system. An engineering study was then performed by the MWW comparing new floating covers to alternative types of rigid frame cover structures, e.g., prestressed concrete tanks, reinforced concrete covers with column supports and aluminum deck covers with structural supports. This study concluded that installation of new floating covers would produce the most cost effective solution to the problem, and in late 1980, a decision was made to proceed with final design plans and specifications. The new cover project was let to bid in February, 1981 with the low bidder being Sta-Flex Corp. of Greenland, N.H. who submitted a base bid price of $527,000 for 34,782 m² (374,400 ft.²) of hypalon material and floats at the three reservoirs. Cover installation work began in late May, 1981 and was completed in November, 1981 at a final contract price of $550,100.

The design of Manchester's new second generation floating covers was based on a patented cover system offered by Burke Industries, Inc. of San Jose CA. This system layout incorporates a 45 mil, 5 ply reinforced hypalon membrane with a weighted sump and parallel float arrangement which provides necessary alignment for the cover and also helps to facilitate surface water runoff. Floats are 10.2 cm (4 in.) by 30.5 cm (12 in.) in cross section, encased in 30 mil thick reinforced hypalon and then solvent welded to the top of the cover membrane. Weights are constructed of 30 mil reinforced hypalon tubing filled with sand. These are deployed between the floats to provide positive submergence of the sump area. (See. Figure 3.) The hypalon membrane was joined together in the field using a standard bonding solvent (xylene) adhesive system. A 15.24 cm (6 in.) minimum overlap and 7.6 cm (3 in.) minimum bonded seam width from the outside edge of factory assembled panels was specified. Additionally, the MWW required that representative samples of flexible membrane be removed from factory shipped panels at the job site and then tested for conformity with the specifications. This work was done by the materials testing laboratory at the University of New Hampshire. A summary of the UNH/FML test data is shown in Table 1.

TABLE I
SUMMARY OF PHYSICAL TEST DATA -45 MIL CSPER
MANCHESTER WATER WORKS

Property	Specification Requirement	Test Samples
Weight	1.62 kg/m² (0.332 lbs/ft²)	1.66 kg/m² (0.34 lbs./ft.²)
Thickness	0.10 cm (0.041 in. min.)	0.11 cm (0.045 in.)
Puncture Resistance	74.84 kg (165 lbs. ave.)	107.5 kg (237 lbs.)
Breaking Strength:		
Fabric- Longitudinal	45.36 kg min. (100 lbs. min.)	76.2 kg (160 lbs.)
Rubber- Longitudinal	68.04 kg min. (150 lbs. min.)	73.9 kg (163 lbs.)
Elongation:		
Fabric-Longitudinal	15% min.	35%
Rubber-Longitudinal	150% min.	209%
Tear Strength: Longitudinal	13.61 kg ave. (30 lbs. ave.)	39.5 kg (87 lbs.)
Cold Bend	No cracks visible at 7X	Pass
Seam Strength	To exceed parent material	Pass

Note:
Testing done by Center for Institutional and Industrial Development, University of New Hampshire, Durham, N.H.

Operational experience with these new second generation floating covers of hypalon began in June, 1981 upon completion of the cover installation at the large High Service Reservoir. A brief summary of operating experiences and some general observations about cover performance are presented below.

June, 1981: The installation contractor completed work at the large High Service Reservoir in about five weeks time. During installation, trees surrounding the reservoir were completely stripped of foliage by a regional infestation of gypsy moth caterpillars. Millions of these insects climbed the outside wall of the reservoir and upon reaching the top of the perimeter wall then slid down the hypalon membrane to the bottom. Workmen had to remove the insects by sweeping, hosing and pumping to prevent septic odors and sludge buildup on the bottom. The insect problem persisted for about a month after the reservoir was returned to service. It was later concluded that had this condition occurred a year earlier when the reservoir was uncovered it would have been necessary to completely drain and clean the reservoir to avoid a major water quality problem.

The MWW designed and installed a new electric submersible pump system for more effective removal of surface water from the floating covers. The pump and floating support platform is lighter in weight, easily placed and removed, and less expensive than the original design by MWW. (See Figure 4.)

July 1981: The contractor removed the old floating cover and installed the new hypalon cover in about five weeks time. However, repairs first had to be made to a 60 mil thick non-reinforced EPDM liner installed in 1970. This liner had failed at several locations on the reservoir bottom due to expanding gasses from organic sludge decomposition. MWW engineers concluded that some organic matter had escaped removal prior to liner placement, thus creating the failure. However, no sign of any leakage had occurred before the reservoir was removed from service and overall the liner was considered to be in excellent condition after eleven years of service.

Severe damage was done to the new floating cover at the large High Service Reservoir by vandals who penetrated a chain link security fence and proceeded to cut a large square hole in the cover about 2.4 m (8.0 ft.) on each side. A large portion of the cover thus became flooded with surface water making in-service repairs impossible. Thus, the reservoir had to be drained and the cover repaired at a total cost of about $4,900. (This included the loss of treated water in the reservoir.) Additional security fencing was later installed and police patrols increased at the reservoir. However, these measures have not proven totally successful in preventing further incursions into the area.

October, 1981: Large flocks of seagulls began showing up at the reservoirs leaving pollutants on top of the cover and causing some physical damage to the flexible membrane as well. A nearby City landfill seemed to be the primary attraction. MWW officials contacted the U. S. Fish and Wildlife Service (USFWS) and several techniques were used to frighten the gulls away with little success. One method which did meet with measurable success was the occasional shooting of gulls using 12 gauge shotguns loaded with No. 6 birdshot. Shooting was done in a tightly controlled manner by MWW and City police department personnel operating under a special permit issued by the USFWS. Federal officials have since embarked on a major study of the inland gull population in this region and although no definite conclusions have yet been drawn, preliminary evidence suggests that definite control measures will be needed in the future to ensure public safety.

November, 1981: A new operating procedure to control reservoir levels during winter months, thereby reducing ice action along reservoir sidewalls, was implemented by the MWW. Pumping plant operators were given special instructions to closely monitor reservoir levels by operating high lift pumps at the main treatment plant to provide a 0.3 m (12 in.) maximum operating range in the reservoirs for the entire winter period. This procedure has since proven to be very beneficial in reducing winter damage to the FML material.

February, 1982: Extremely low temperatures and heavy snowfall occurred with no visible sign of any ice damage to the cover membrane at the three reservoirs. Although some ice thrust was still observed along sidewall areas, the amount and size of fractured ice was substantially reduced from previous years by the new operating procedure as documented by photographs taken in 1978 and 1983 and shown as Fig. 1 and Fig. 2 respectively. Inspection hatches at the Low Service Reservoir were opened on 2/3/82 and 0.2 m (8 in.) of ice thickness was measured beneath the cover membrane.

April, 1982: Once the ice and snow had melted away, electric submersible pumps were reinstalled at the reservoirs and the covers were effectively dewatered in roughly two weeks time. Inspection of possible winter damage disclosed only several small tears about 2.5 cm (1 in.) long at the large High Service Reservoir and a single tear of about 15.2 cm (6 in.) at the Low Service Reservoir near the outside corner of a concrete gate structure. It was concluded that major improvements in the cover design to handle surface water runoff, improvements in the physical properties of the flexible membrane, particularly bonded seam strength, and the new operating procedure followed by MWW during winter months had combined to produce a very positive effect on operating performance.

April, 1983: Inspection of the floating covers disclosed that no physical damage had occurred as a result of the winter of 1982-1983. A single float pump was again deployed at each reservoir and surface water from accumulated snowmelt, etc., was totally removed in about two weeks time.

May, 1983-January, 1984: Floating cover operation has been very routine and very positive in terms of protecting treated water quality at minimum cost. Each reservoir cover is given a close inspection monthly and any surface debris, e.g., rocks or bottles, are promptly removed. Aging and weathering characteristics of the hypalon are currently being monitored on an annual basis by the MWW.

In conclusion, the Manchester Water Works believes that it has solved most of the problems associated with the old EPDM floating covers and that with regular inspection and maintenance these second generation covers of hypalon should provide low cost and reliable service for many years to come.

Fig. 1. - EPDM Floating Cover at 20 MG Reservoir
March, 1978

Fig. 3 - Photo of typical rainwater collection channel at 4 MG reservoir
October, 1982

Fig. 2 - Hypalon Floating Cover at 20 MG Reservoir
March, 1983

Fig. 4 - Photo of 4 MG reservoir cover and dewatering system
October, 1982

The above photographs show before and after effects of reservoir level control procedure followed by MWW during winter months. Note reduced ice breakup along reservoir wall in 1983 photo.

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Foam Rubber Covers for Evaporation Control

Floating covers, constructed of low-density closed-cell synthetic rubber sheeting (EPDM), were first used in 1971 to control evaporation from vertical walled water storage tanks. The covers are fabricated from roll-stock, generally 1.22 m (4-ft) wide by 6.4 mm (1/4-in) thick by bonding cut sheets together with contact cement. A 9-m-diameter cover can be constructed in about 3 hours by two people. The covers are susceptible to wind damage, but practices have been developed to minimize the problem. At least 100 covers are presently in use on tanks in the southwestern United States. Procedures for estimating the cost of saving potentially evaporated water are presented.

In many areas new water supply development costs such as hauling, collecting (harvesting), pumping, or piping long distances are generally high. Evaporation rates from the stored water can be well over two meters per year in many parts of the world which many times could be more than the depth of water held in a tank. In these high evaporative demand areas, evaporation control measures should be considered, since it is generally more cost effective to conserve water already in storage than to develop more.

Floating covers are one of the most effective ways of controlling evaporation from open tanks. These covers generally reduce evaporation in proportion to the water surface area covered. Cooley (1) found that evaporation reduction ranged from 36% for floating foamed wax blocks to 84% for floating covers made of foamed rubber sheeting. The foamed rubber cover in Cooley's study was made of the material first evaluated and tested in Utah in 1971 (4). This report summarizes the application of this material when used as a floating cover for controlling evaporation from vertical walled water storage tanks.

MATERIAL

The foamed rubber covers are made of low-density (110-130 kg/m^3), closed-cell synthetic rubber sheeting (EPDM) in roll-stock form. It comes in various roll widths up to 1.22 m (4-ft) and from 3.2 mm (1/8-in) to 12.7 mm (1/2-in) thick. The roll-stock material is fabricated into a cover by lap jointing, using contact cement. A cover constructed of 6.4 mm (1/4-in) sheeting that fits a 9 m (30-ft) tank, weighs about 52 kg (115 lb), making it easily transportable to the field.[1/]

COVER FABRICATION

The three original covers (4) installed in 1971 were fabricated from 4.8 mm (3/16-in) roll-stock, one of which is still in use. Later work showed that 6.4 mm (1/4-in) material is less susceptible to being blown around and is now recommended over thinner sheeting. Actual fabrication details are only briefly described. For more detail, refer to Dean (2) and Dedrick (3).

Since few tanks are really round, several tank diameter measurements should be taken. The final cover diameter should be the same as the minimum tank diameter. Originally we recommended that the cover be 2.5 to 5 cm (1 to 2 in) smaller than the minimum tank diameter (3). However, because the material shrinks with time, it is recommended that the cover be made as large as possible. Some covers have been constructed slightly oversized but float freely after initial shrinkage. Covers generally are assembled on hard, flat surfaces. Panels are cut from the roll-stock to the appropriate lengths, allowing for overlap (Fig. 1). The panels are bonded together using a contact cement available from the manufacturer of the sheet stock. Surfaces to be bonded must be properly cleaned. In laboratory tests the cover material, rather than the joint, would fail if the lap width exceeded 5 cm (2-in). Hence, lap width should be 5 to 6.5 cm (2 to 2 1/2-in). Seaming is shown in Fig. 2.

Foamed rubber stripping, either rod stock or rectangular, has generally been bonded around the edge of the covers, on the water side. The edging provides some stiffening and assures complete edge contact with the water which helps prevent wind from getting underneath the cover.

Small holes, about 1 cm in diameter (Fig. 1), should be cut about 1 to 1 1/2 m apart along each roll-stock strip to allow the cover to self-bail rainwater and snow melt collected on top. The holes also help during installation of the cover. Sinking the cover by pouring water on top will allow all entrapped air to escape. The cover will then ride directly on the water surface without air pockets.

1/ Material available from Inmont Corp., St. Louis, MO. Listing of the company name is for the readers' use only and does not imply endorsement of the products or the company by the U. S. Department of Agriculture.

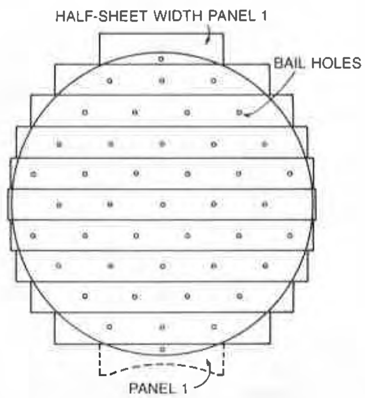

Fig. 1. Layout and procedure for cutting panels from roll-stock. Material must be lapped when cut. Bail holes, 1-cm diameter, should be cut every 1 to 1 1/2 m.

Fig. 2. Lapjointing sheet stock during the fabrication of a floating cover. Sheets must be carefully aligned.

Fabrication time depends on the cover size, sheeting width used, and number of covers being constructed. Generally, on a one-cover-at-a-time basis, two people can assemble a 9 m (30 ft) diameter cover using 1.22 (4-ft) wide material in about 3 hours.

INSTALLATION

All plumbing within the tank should be located so that the cover can float freely. If the water supply to the tank is clean, inlet water can go directly on top of the cover since the cover will self-bail. Inlet water carrying debris and/or sediment should be directed down the side of the tank to avoid deposition on the cover. The tank should be equipped with an overflow to prevent filling to the top (Fig. 3). A 30 cm (1-ft) freeboard is recommended.

The cover should be carefully handled during the installation process to avoid tearing. Damage to the cover can be minimized by (a) folding the cover from each side while on the ground next to the tank, then (b) lifting the cover by the folded edges and sliding it

Fig. 3. Floating cover on water storage tank near Safford, Arizona. The inflow pipe conveys water from a water harvesting catchment. The tank is notched (upper left side) to provide an overflow. This floating cover was constructed and installed in 1974.

across the water (Fig. 4), and (c) unfolding the cover onto the water surface. When the cover is in place, water should be poured on the surface as described earlier.

The covers can be damaged by wind and are most susceptible when the water surface is either very near the top of the tank or the tank is less than half full. The following measures should be taken to reduce this risk: (a) maintain the freeboard discussed, (b) stretch wires diametrically across the top of the tank, and (c) use an edging material to assure good cover contact with the water surface (discussed previously) and to provide some rigidity to the cover. Some users have used polyethylene pipe to provide the rigid edge and have weighted it with water to assure contact with the water. Such an edging could also be weighted with sand, etc., which would turn the edge of the cover down into the water. Upon freezing, the edging would be entrapped in the ice which would then anchor the cover in place. Several ranchers have been innovative in making some of these improvements.

COVER PERFORMANCE

Several experimental floating covers were installed during the 1971-to-1974 period, specifically for field evaluation. One of the original covers, installed in southwestern Utah in 1971, is still being used. The cover was 4.7 mm (3/16-in) thick, and about 9 m (30-ft) in diameter. The cover has become quite stiff, but as long as it is not handled, it continues to function properly. The cover has shrunk over the years as follows: original diameter (November 1971), 8.99 m; September 1975, 8.56 m (4.8% shrinkage), January 1984, 8.38 m (6.8 % shrinkage). Others installed in 1974 continue to function satisfactorily.

Many operational water tanks have been fitted with floating covers since 1974. Most covers of which the author is aware are on tanks used to store water developed on land managed by the Bureau of Land Management and Bureau of Indian Affairs in Utah and Arizona. The largest numbers currently in use are near Safford, Arizona--twenty-five--and nearly fifty on tanks on the Arizona Strip (area north of the Grand Canyon). These

Fig. 4. Floating cover being lifted and slid across the water surface (top). Once on the water, the cover is unfolded (bottom). Folding helps prevent puncturing of the cover during installation.

covers were fabricated and installed and are maintained mostly by Bureau of Land Management personnel.

Of the approximately one hundred covers that the author knows of, no seaming problems have been encountered. Unsatisfactory jointing was experienced by a company planning to manufacture covers as part of an already established business. Efforts were made by two Agricultural Research Service stations to bond the material, supplied from the company, without success. Exact reason for the problem was not determined. Since this incidence, many covers have been made by others without problem. Birds have been suspected of pecking the covers, causing a few holes, but this is generally a minor concern. The covers can be punctured by thrown rocks. Dust tends to accumulate on floating covers and many times clogs the bail holes. The holes are, however, generally reopened when water wets the surface of the cover.

Wildlife managers are enthusiastic about the covers since they serve as safe watering "rafts" for birds. Literally hundreds of dove have been observed around and on tanks equipped with floating covers. Floating covers prevent algae buildup, typically a nuisance in tanks in warm climates.

COST

Cost of water saved with a foamed rubber floating cover depends on the evaporation for the area, initial material and construction costs, evaporation control attainable with the cover, life of the cover, and interest rates. The cost per unit volume of water saved by controlling evaporation from a circular above-ground tank can be estimated by:

$$C = \frac{(CRF)}{14 e_r E} \left[C_e + \frac{18\ C_m}{D^{1/8}} \right]$$

where C = cost per unit volume of water, $\$/m^3$

CRF = Capital Recovery Factor = $\frac{i(1 + i)^n}{(1 + i)^n - 1}$

i = Interest rate in decimal form
n = Life of cover, yrs
C_e = Hourly labor cost, \$/hr
C_m = Sheeting cost, $\$/m^2$
e_r = Evaporation reduction efficiency in decimal form
E = Lake evaporation for area, m/yr
D = Tank diameter, m

Multiply cost in $\$/m^3$ by 3.79 to convert to \$/1000 gal.

From the experience gained using these covers since 1971, it appears that cover life is at least 8 years, and in many instances it has been 10 or more. The following example illustrates the use of Equation 1. If $C_m = \$4.50/m^2$, $C_e = \$10/hr$, $i = 0.12$, $n = 10$ yrs, $E = 2$ m, and $e_r = 0.85$, then the cost per unit volume of water saved for a 10-m-diameter tank would be:

$$C = \frac{0.177}{(14)(.85)(2)} \left[10.00 + \frac{(18)(4.50)}{10^{1/8}} \right]$$

$$C = \$0.52/m^3 \ (\$1.98/1000 \text{ gal})$$

This is inexpensive water when compared to the costs often encountered getting it there in the first place.

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Design of a Geomembrane Floating Cover to Contain Methane Gas for Positive Energy Applications

Installation of what is believed to be the first high density polyethylene hybrid floating cover took place in August 1983. The Heyburn, Idaho, facility of the J.R. Simplot Company's Food Division converted an activated sludge treatment system for potato process water to an anaerobic/aerobic treatment system. The company plans to recover and use methane in the power requirements of the plant.

The selection of a 2.5mm (100 mil) HDPE liner took advantage of the material's chemical and physical characteristics, as will be explained. The application did, however, pose some atypical design and operating issues. This paper discusses those issues and describes the design details that integrated material properties with such functions of the cover as year-round exposure to the weather; the hermetic seal necessary to prevent a possible explosive gas mixture; a gas transmission system; and provisions for cover stability, and rainwater and snow melt collection.

METHANE GENERATION FROM DOMESTIC AND INDUSTRIAL WASTE

One of the byproducts of an industrialized society is waste. When domestic and commercial waste is disposed of in a sanitary landfill, it will release methane as organic matter decomposes. This gas, while usually not hazardous to nearby populations, is almost always offensive because of its odor. Some municipalities have turned the irritant into a positive energy source by containing the gas, thus eliminating the odor, but further by utilizing the gas to provide energy, which is then either sold to a utility or consumed by the municipality.

When a geomembrane is employed in this application, it typically is the middle layer of a sandwich that consists of compacted waste covered by the geomembrane, covered by an overburden such as sand or dirt. The area is then revegetated for aesthetic and environmental reasons. Because the geomembrane is protected by the overburden from exposure to the weather, and from excess pressure from within the landfill because gas is withdrawn before pressure builds, it is common to specify a liner in the thickness range of 0.5mm to 1.0mm (20 to 40 mil).

Industrial plants, and particularly facilities engaged in food processing, brewing, paper making and meat packing, generate organic waste in the form of spent process water. Before this process water is released for municipal treatment or discharged directly into natural surface waters, it must be treated for BOD, COD, suspended solids removal and other contaminants particular to the process. Such treatment has traditionally been done with a sequence of primary clarifiers, aeration basins and secondary clarifiers. It is a familiar technology. Two major disadvantages are, however, associated with this process: it generates large amounts of sludge that must be separately handled and disposed, and it requires rather high energy and operational costs.

Some plants have accordingly changed their method of treatment to one of anaerobic digestion. In this process the decomposition of organic wastes is accomplished by micro-organisms, or "bugs" as they are affectionately termed, that function in the absence of oxygen. Under controlled atmospheric and temperature conditions, micro-organisms proliferate and generate methane.

The authors wish to note that biogas, not methane, is the initial result of organic decomposition. Biogas typically consists of 60 percent methane, 35 percent carbon dioxide, three to four percent moisture, and about one percent other gases. But because methane is valuable, and recoverable, that term is used throughout this paper to describe the byproduct of an anaerobic treatment system.

An anaerobic digestion system is not without hazard or handicap. Not only is the methane potentially explosive, with the system thus requiring skilled operators, but initial capital costs can be quite high. It is difficult to postulate a threshold beyond which an anaerobic digestion system is economically unfeasible, but generally it is a function of the availability of land and the volume of daily discharge water.

Where land is readily available, however, to treat the volume of discharge, an anaerobic digestion system offers a marked decrease in sludge volume and a significant reduction in BOD and COD content which can cut the time needed for subsequent aeration by as much as 90 percent. Furthermore, while methane can be vented and burned, it may also be utilized for positive energy applications. It is entirely possible that the payback period for such a treatment system might be as short as a few years.

J.R. SIMPLOT COMPANY, FOOD DIVISION

Until the 1983 conversion, the Heyburn, Idaho, potato processing facility had used an activated sludge treatment system. Although fully cognizant of the advantages associated with anaerobic digestion, the director of the company's environmental affairs considered that the volume of process water, 3 MGD at peak flow, would require a prohibitively expensive $2.5 million concrete tank.

A further study, however, recommended a direct conversion of the existing aeration basin, 104m x 121m (342 x 396 ft.), into two cells: the smaller unit, 104m x 30.5m (342 x 100 ft.), would function as a secondary aeration pond, but the remaining area of 104m x 90m (342 x 296 ft.) would be covered and sealed with a geomembrane in order to contain and collect methane; a vacuum pump would provide constant venting. The cover would be exposed to the weather year round. A concrete ringwall would be constructed on three sides of the anaerobic pond; the fourth side, which demarcates the cells, would be a baffle wall constructed of treated timber.

The opportunity to turn the treatment system into an energy producer rather than a consumer, along with the coincidence of a rare plant shutdown, were the final deciding factors in the decision to proceed.

MATERIAL SELECTION AND DESIGN OF THE COVER

The conversion proposal presented serious issues in terms of the physical and chemical properties required of the material, as well as the engineering details critical to the design of the cover and operation of the anaerobic system. Specifically, the cover had to accommodate:

1. Surface wind velocities as high as 129 km/hr (80 mph);
2. Ambient air temperatures of -34 °C to 43 °C (-30 °F to 110 °F);
3. Wastewater that contains vegetable oils, grease and residual hydrocarbons, as well as, eventually, methane and hydrogen sulfide;
4. Under-cover pressure of up to .0005 kg/cm^2 (.07204 psi or 2" WC) in the eventuality of a vacuum pump failure;
5. A system for the removal of rain water and snow melt;
6. The installation and operation of a gas transmission system under the cover;
7. A hermetic seal around the pond's perimeter;
8. As few seams as possible in the joining of the cover material, so as to minimize the probability of seam failure and consequent hazardous condition; and
9. Continuous operation during an anticipated 20 year design life.

CHEMICAL COMPATIBILITY TESTING

The environmental engineering staff of J.R. Simplot Company initially evaluated both high density polyethylene (HDPE) and hypalon liners. Chemical compatibility testing for both liners was conducted by Schlegel Lining Technology, Inc., in accordance with its laboratory test procedure LP 0420.

This test method was developed because at present there is no ASTM test procedure for evaluating the chemical compatibility of HDPE liners. EPA's P9090 test procedure is not yet standardized.

The LP 0420 test is intended to determine the resistance of geomembranes to chemical attack through an evaluation of changes, if any, in tensile properties. Although this method does not, and cannot, duplicate environmental conditions perfectly, it does indicate which liners are more suitable than others for exposure to specific reagents. The test is run at high temperatures in order to accelerate the detrimental effects, if any, of the chemical reagent(s) in question.

Weight changes are recorded at seven day intervals, and the total percent weight change after 28 days is calculated. After a 24 hour acclimatization to the laboratory environment, each sample is evaluated and compared with the tensile strength and elongation of the control samples, according to ASTM D638.

For a material to be considered compatible with the test media, the following criteria have been established:

1. A weight change no greater than ± 3%.
2. A change in tensile properties no greater than ± 10%.
3. Those samples not meeting criteria 1. and 2. above, could be considered compatible only if both of the following conditions are met:
   a. weight gain has stabilized and begun to recede by the 28th day, AND
   b. tensile values for exposed samples are within the "minimum specification" range for the unexposed material.

In accordance with the LP 0420 test procedure, five samples of each liner material were immersed for a period of 28 days in a leachate solution provided by Simplot. An additional five samples of each liner were retained for control purposes.

The average change in weight for the exposed five samples for each material is shown in Table I below.

TABLE I
WEIGHT CHANGES AND % WEIGHT GAIN
(Values averaged from five samples)

	Day 1	Day 7	Day 14	Day 21	Day 28	Total % Weight Change
(Schlegel) HDPE:	4.017226 g	4.023682 g	4.033022 g	4.023776 g	4.023018 g	0.14%
Hypalon:	2.122580 g	3.092516 g	3.758410 g	4.022692 g	4.791688 g	125.00%

Table II shows the tensile properties of the HDPE liner as average values for the five exposed specimens. Tensile testing could not be performed on the hypalon samples, because the material had curled to the point where such testing was impossible; by the 21st day of immersion, the samples had also swelled enough that most of the scrim-reinforcement fibers were no longer visible.

TABLE II
TENSILE PROPERTIES
(Values averaged from five samples)

	Ty (psi) Tensile Yield	Elongation Yield (%)	Tb (psi) Tensile Break	Elongation Break (%)
HDPE Control	2916	15.00	4045	802
HDPE Exposed	2935	15.25	3962	802
% Change	+ 0.65%	+ 1.6%	- 2%	- 0.74%

Based on the results of the chemical compatibility testing, J.R. Simplot Company issued a specification for the cover that called for a 2.0mm (80 mil) liner of HDPE. The specification was changed to 2.5mm (100 mil) shortly thereafter, when a number of calculations demonstrated that the extra thickness increased the safety factor by 50 percent.

Other factors that influenced the company to specify the HDPE material in the 2.5mm thickness included:

1. A -56.6 °C (-70 °F) cold crack temperature for the material, combined with the thickness, indicated good resistance to climatic conditions at the site.
2. The manufacturer confirmed the chemical resistance of the material against the leachate used in laboratory testing.

3. A value of 24N/mm ^{2} (3500 psi) at break indicated that the material would withstand vacuum pump failure and consequent generation of gas at greater than .0005 kg/cm ^{2} (2" WC) on an intermittent basis.
4. Density of 0.940 g/cc for the material meant that the cover would remain floating, even if it were punctured.
5. A cover of HDPE could be designed to accommodate surface water removal and a gas transmission system beneath it.
6. A system for hermetic sealing around the pond's perimeter was available.
7. Linear footage of seams for HDPE panels in a 10m (33 ft.) width was only 945m (3,100 ft.), excluding the perimeter.

GAS TRANSMISSION SYSTEM

Gas production under the cover will vary somewhat from one area of the basin to another. The variability results from changing concentrations of micro-organisms, the temperature and concentration of waste in the incoming process water, the flow rate of the process water, its point of introduction into the basin, and the amount of mixing within the basin.

It was necessary that the cover provide both containment of the gas and a means of channeling it to various draw-off points. From the containment standpoint, the variable gas production rate will exert uneven pressures on the underside of the cover, with the potential to cause the cover to balloon in spots. The lining industry often refers to these areas of uplifted geomembrane as "whale-backs." The possibility of whale-backs raised two concerns: the ability of the liner to withstand high winds in such a condition, and the need to vent excess pressure quickly so that the cover does not rupture. As already noted, the cover designed for J. R Simplot Company will withstand a pressure of .0005 kg/cm ^{2} with the 50 percent safety factor provided by the increased thickness. The strength of this material in this thickness, and a system of cables, described elsewhere in this paper, secured to the top of the cover, are the primary means of resisting undercover pressures.

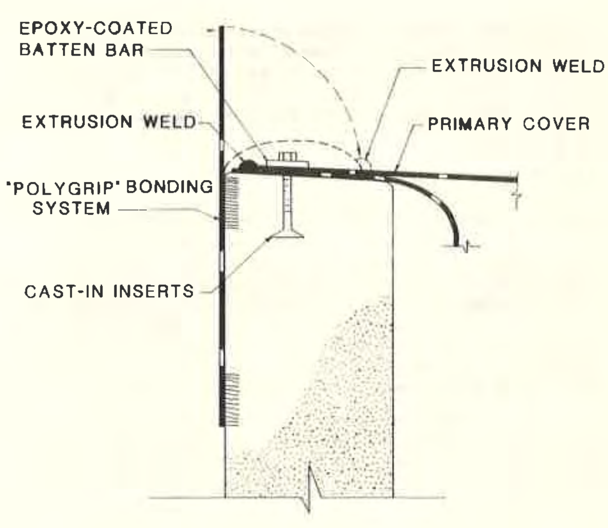
The other consideration for the cover was the channeling of the gas to specific draw-off points. A series of undercover float pipes were installed to provide this channeling. After allowing for the displacement of the pipe floating in the wastewater, a void space is created on either side of a pipe, and the gas flows through these channels to collection points. Vacuum pumps draw off the gas under controlled, powered conditions. The vacuum pump system is designed and controlled to produce a maximum of .0025 kg/cm^2 (10" WC) vacuum at each of several connection points.

A further safety system was designed. If the vacuum pump fails, as could happen through loss of power or human error, pressure rise under the cover cannot be allowed to continue unabated, as the cover could rupture and fail. To prevent this, a series of pressure relief valves, located on the main collection manifold, were engineered and installed. The valves are set to relieve at + 2" WC.

HERMETIC SEALING

From a design standpoint, the method of attaching and sealing the cover at the perimeter of the pond proved to be the greatest concern. If any leakage develops, negative pressure from beneath the cover will draw in fresh air, not only reducing the effectiveness of organic decomposition, but far more seriously, creating the potential for an explosive mixture of oxygen with methane. Other areas with potential for air infiltration, albeit not as grave as that described above, were the dirt or freeboard area between the top of the water level and the base of the concrete ringwall, and the expansion joints and any cracking that might occur in the concrete wall. These were addressed during the design stages of the project.

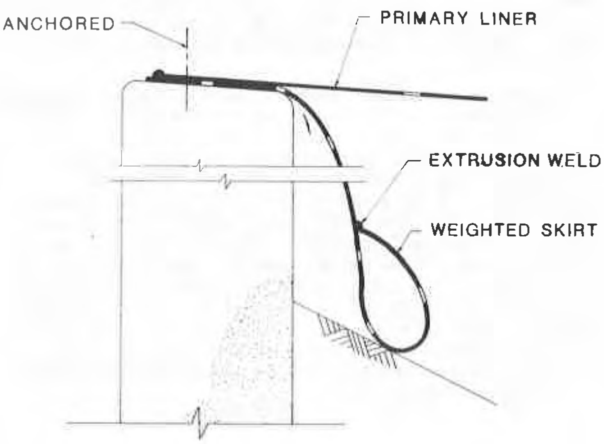
To ensure the necessary hermetic seal,a concrete ringwall was first constructed around three sides of the pond. (The fourth is a wooden wall.) Incorporated into this ringwall is a seal, proprietary to Schlegel Lining Technology, called the "polygrip" bonding system. Essentially, it is a mat of fibers integrally bonded to HDPE liner. For the Idaho installation, concrete was poured around the "polygrip". When the concrete had cured, and the formwork was removed, the "polygrip" was bonded within the wall. Sufficient HDPE liner was left to extend above the top of the wall and along its entire length. This material was folded over the cover after it was in place and extrusion welded directly to it. HDPE can, characteristically, be joined to other HDPE at any time (Figure 1).



'POLYGRIP' BONDING SYSTEM DETAIL

FIGURE 1

To seal the freeboard and expansion joint areas, more liner was attached to the wall and extended down the side slopes of the basin, forming a skirt. Because of its lighter-than-water density, the skirt was weighted at its bottom (Figure 2).



WEIGHTED SKIRT DETAIL

FIGURE 2

A battening system of plated steel bars was also employed: to secure the sidewall skirting; as an extra measure of security before the lip of the "polygrip" bonding system was folded and welded; and to secure the cover to the fourth side of the pond, the treated timber wall.

ATTACHMENTS TO THE SURFACE OF THE COVER

The final concern in regard to the issue of cover stability and operational safety was how to mitigate the effects of a vacuum pump failure and ensuing inflation of the cover. As previously explained, the HDPE material in the 2.5mm thickness will withstand at least a 2" WC pressure, but even so, a pump failure would result in pressure under the cover of more than 500,000 kg (1.1 million pounds) before the valves in the piping system release and vent gas to the atmosphere.

The cover design addressed this eventuality with a provision for securing cables to the top of the cover. These cables are designed to distribute uniformly any lift from under the cover until the relief valves come into operation.

The cables perform a number of other functions as well. Secured to concrete deadmen, they assist the cover in resisting lateral wind loads and, by keeping the cover longitudinally stable, keep the rain water collection channel in place.

In the early design stages, rain water collection was not viewed as a serious problem, because annual rainfall in this area of southern Idaho averages only slightly more than 20cm (8"). When it was learned, however, that half of this annual rainfall can occur within a period of time as short as a few weeks, the problem assumed a different dimension.

The design called for rain water, and snow melt, to collect in a trough formed in the center of the cover with weighted pipe; as it accumulated, water would be removed from the cover's surface with a sump pump. When the weighted pipe was put in place, however, the trough that was formed was only 15cm (6") deep, even with a dozen men adding their weight to the area. A system of turnbuckles was incorporated into the already-in-place cable system in order to pull as much slack material as possible into the trough area. This resulted in a channel of the desired depth, although a rainfall of 5cm (2") within 24 hours proved to be beyond the capacity of the single pump. Two additional pumps were put into service for this extreme situation. HDPE tensile strength at yield is 2800 psi, thus permitting foot traffic and the direct placement of pumps on the cover.

Ice buildup is not ordinarily a factor for this installation, as the minimum anticipated water temperature within the anaerobic pond is 15.6 °C (60 °F). Internal stress generated by the temperature gradient between the two sides of the cover is accommodated by the HDPE's tensile strength and modulus of elasticity.

INSTALLATION

The traditional installation procedure for a floating cover, laying the liner, seaming it and then allowing it to float as the pond fills, was not viewed as an ideal installation technique. The rigid nature of the material, particularly in the 2.5mm thickness, might have resulted in pockets of excess material if this procedure had been used.

It was decided instead to float the 27,500 kg (60,000 lb.) cover across the already filled pond. The first step was to provide an assembly area on one side of the basin where panels of the 10m (33 ft.) wide liner could be unrolled and seamed. All of the cover accessories, including pipes used in the gas transmission system, manhole access covers, vent pipes and sampling ports, were also attached in this area.

The leading edge that was to be pulled over the water was fitted with a system that kept wind from under the liner and water from washing over it. Using installation techniques that are proprietary to the installer, the cover was pulled across the pond. Hermetic sealing took place immediately after the cover was in place.

QUALITY CONTROL

In light of the potential hazard associated with seam or seal failure, and because of the practical difficulty of repair, quality control procedures were necessarily strict. Welding technicians underwent retraining. The number of required pre-weld samples was increased. All welding was performed with the liner laid over wooden planks to assure contact pressure. All seams were ultrasonically tested. Seam samples were tested on site and sent to the laboratory for corroboration; typical laboratory results for the welded seams are shown below in Table III.

TABLE III

WELD SEAM QUALITY CONTROL

Tensile Testing

Evaluation: Excellent
Action Required: None

SAMPLE NO.	LOCATION OF SAMPLE SEAM # N/N	BOND STRENGTH 100% Minimum
4	South	149%
6	North	142%
6	South	146%
7	North	134%
8	South	118%
9	North	129%

CONCLUSION

As this paper is being written (January 1984) the anaerobic treatment system at the J.R. Simplot Company facility in Heyburn, Idaho, is close to its objective of energy production. Thirty days after the system went into service, it was producing 850m^3 (30,000 cubic ft.) of biogas daily. By late January 1984, with ambient air temperatures averaging below 0°F, the system had reached a production rate as high as 2500m^3 (88,000 cubic ft.) per day. (It appears that the cover may be assisting in rapid gas production because of its unanticipated insulating property. The black liner may be functioning as a black body in this area of high sun loads. The effect of this situation has not, however, been fully determined.) When a production rate of 2850m^3 (100,000 cubic ft.) per day is reached, probably in early spring 1984, the biogas will be scrubbed for hydrogen sulfide and fed to boilers, in order to produce heat and hot water for the potato processing plant. J.R. Simplot Company anticipates that energy savings for the plant could be more than $300,000 a year, and that the payback period for the entire conversion program will be about two years.

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Upstream Geomembrane Liner for a Dam on Compressible Foundations

As part of an expansion of mill capacity at a gold mine in northern Canada, a new dam had to be built between bedrock abutments, across an eroded fault zone that had been infilled with peat, soft silts, and clays Factors affecting the design of the structure included: potential for large total and differential settlements of the foundation base; insufficient quantities of suitable clay borrow to construct an impervious core; very short construction period. A design was developed with the following features: a tailings core protected by geotextile and an upstream impervious geomembrane blanket, a two-layer shell, and a decant structure located on one of the rock abutments. Details of design, construction, and subsequent performance will be discussed.

INTRODUCTION

A new dam with a decant structure was built for a tailings water retention system of a gold mine in Yellowknife using an upstream impervious geomembrane blanket. Yellowknife is located in the Northwest Territories of Canada on the northshore of Upper Slave Lake at approximately 62° northern latitude. A series of natural depressions near the mine has been used for tailings disposal and seasonal water retention. In order to increase the capacity of the tailings disposal area, two small dams were constructed at locations as indicated on Drawing No. 1. The resulting tailings pond is called Upper and Middle Pud Lake.

In June, 1981 when the water surface of Pud Lake was as its maximum level, a collapse occurred within the more recently built dam (Dam No. 2 on Fig. No. 1). In addition, a number of serious leaks were observed, particularly at the interface between the fill and the concrete walls of a decant structure in the centre of the dam.

Considering both the poor state of the old dam and the requirement for increased storage capacity in the pond, it was decided to abandon the existing dam and to replace it by a new higher structure.

The new dam was located approximately 30 meters downstream of the existing dam as shown on the site plans on Figure No. 1 and No. 2. The new dam consists of a primary dam section built across the main-valley and a secondary dam section built on the lower parts of the east abutment.

BOUNDARY CONDITIONS FOR THE DESIGN OF THE NEW DAM

The design of the new dam had to take into account the following conditions:

Highly Compressible Foundations

The main dam section is located between rock abutments and based on a highly compressible foundation. A geological section drawn a few metres downstream of the centre line of the new dam is shown by Section BB on Figure No. 3 and indicates the following sequence of materials overlying the bedrock surface:

- silt (tailings), approximately 1.0 m to 2.0 m thick
- peat, approximately 1.0 m thick
- soft to medium stiff clay, approximately 4.0 to 5.0 m thick
- silt, more than 1.0 m thick.

Type and Amount of Construction Material Available for Fill Section

The area of Yellowknife is underlain by Precambrian rock which, during the most recent glaciation, had been exposed in most areas. Till was left only in small pockets of the bedrock surface. During deglaciation, outwash sands and gravels were layed down as well as silty clayey lacustrine deposits. These deposits, however, are often quite soft or frozen and with their high moisture contents, are not well suited for most fill construction purposes. Thus, in the area of the dam site, only a limited selection of suitable fill materials was available, such as: mine tailings; mine waste rock; crushed rock; sands and gravels; and limited amounts of suitable clay (clay till).

Continuous Operation of Mine

The mine had to be kept in operation during the construction of the dam, thus requiring a minimum pond water level behind the existing dam at all times.

Limited Construction Time

Winter discharges of tailings water were effectively prohibited by environmental quality criteria. Therefore, the dam had to be completed before fall of the same year, in order to provide sufficient storage capacity for winter flows.

A total period of 90 days was available for field investigations, design, tendering, and completion of construction.

Small Size of the Dam

The relatively small dimensions of the dam did not justify measures which would otherwise be practical for a larger structure, such as drying of clay borrow, excavation of peat deposits, or dewatering the foundation soil.

DESCRIPTION OF DAM

Various design alternatives were studied. The design concept adopted is characterized by the following features for the primary dam section, as shown in Figure No. 4.

- A tailings core, protected against piping by a woven geotextile.
- An upstream impervious blanket consisting of a synthetic membrane liner (geomembrane liner).
- An upstream sheet pile cut-off through the peat bed into the underlying impervious clay deposits and connected to the upstream blanket.
- A downstream toe-drain consisting of free-draining gravel protected by geotextile.
- A two-layer shell of crushed rock and waste rock.
- A decant structure based on bedrock at the east abutment.

The elastic membrane liner was connected laterally to both abutments by waterproof clamp connections. In order to provide a smooth contact surface, concrete curbs were built on the rock surface of the abutments along the line of contact as shown on Figure No. 6.

The maximum height of the main dam is 7.6 m, the crest length is 63.0 m, and the maximum width at the base is 40.0 m. The area of the upstream impervious membrane liner is approximately 1200m^2. Construction of the dam was started in early August, 1981 and was essentially completed by the end of September, 1981.

The secondary dam section on the east abutment is based on bedrock and consists of a clay core and a two-layer shell of crushed rock and waste rock. It will not be further discussed in the context of this paper.

GEOTECHNICAL PROPERTIES OF FILL MATERIALS

The fill materials used for the main dam section are represented by their grainsize curves on a composite plot on Figure No. 5, and will be discussed with respect to their geotechnical properties in the following:

Old mine tailings, designated as material No. 1 on the drawings was the fill material most readily available on the mine site, and was selected for the main dam core. The mine tailings are basically silts with some fine sand and negligible clay sizes. The angle of internal friction for the uncompacted tailings varies between 37^o and 42^o, reflecting the angular nature of the grains. The mine tailings are highly frost susceptible and are easily erodible by water. When exposed to the critical hydraulic gradient, the tailings are prone to piping. Thus, it appeared essential to protect the tailings core by additional measures, such as an upstream impervious blanket, filter zones along the contact to other materials, and a downstream toe-drain. The in-situ moisture content of the old tailings deposits depended very much on the weather conditions, varying between 14% in August and 20% in September. Thus, at the end of construction, it became more and more difficult to find material with a moisture content close to its optimum.

Material No. 2 is the rock crush which could be obtained from the mine crusher. It constitutes a sandy fine gravel with almost no silt-sizes. This material was used in combination with geotextile as a transition zone between the tailings core and the rock fill cover in the primary dam section. The same gravel was also placed between tailings and toe filter which is made up of material No. 3. Material No. 3 originates from the rock crush (material No. 2), after sieving out the finer than sand portions. Thus, it is uniform fine gravel which is highly pervious, and therefore, is well suited for collector drains.

Clay suitable for dam construction could be obtained from a pocket of clay-till. Unfortunately, the volumes available were too small to be applicable for the main dam section and were barely sufficient for the core of the secondary dam.

CRITERIA FOR THE SELECTION OF GEOMEMBRANE

The criteria which had to be applied for the selection of the geomembrane for the upstream impervious blanket were the following:

- The membrane has to be flexible to withstand the anticipated large differential movements within the earth fill dam (more than 600 mm over a length of approximately 3 metres).
- The membrane has to be chemically resistant to the tailings materials retained by the dam.
- The membrane has to perform satisfactorily at low temperatures (approaching -40^{o}C at the surface).
- The membrane must be sturdy enough to resist tough treatment during construction.
- Seams must be jointable at relatively low temperatures, ranging between 0^{o}C and +10^{o}C during the seaming process.

Heavy gauge HDPE was initially considered for the project, but concerns were raised on handling and joining characteristics at the anticipated low installation temperatures. A flexible geomembrane was specified with properties similar to Hypalon or CPE. Reinforcement of the membrane did not appear advantageous, as it was felt that adaptability to large deformations in the fill is more desirable than very high strength values. A minimum thickness of 30 mil (0.8 mm) was specified to provide sufficient strength against puncturing and other possibilities of mechanical damages during installation of the liner. Eventually, on the basis of cost, a 30 mil non-reinforced Hypalon liner was selected.

The liner sheets were put together to a large degree in the factory, in order to minimize the number of field seams. The field seams were arranged vertical to the slope and jointed by using a one-compound cold-setting adhesive. This method proved to be successful even at the relatively low temperatures encountered during most of the operation, ranging between 0^{o}C and +5^{o}C. The main mechanical properties of the selected 30 mil Hypalon membrane are listed below:

- Tensile strength 5.0 (KN/m) (ASTM D882)
- Elongation at Failure 250(%) (ASTM D882)
- Tear Strength 39.4 (KN/m) (ASTM D751)
- Brittle Point -43^{o}C (ASTM D746)
- Operating Temp. Range +30^{o}C to -40^{o}C (ASTM D471)

CRITERIA FOR THE SELECTION OF GEOTEXTILE

The criteria for the selection of the geotextile are summarized in the following:

- High tensile strength.
- Elongation properties less than for the geomembrane to minimize creep of the overlying fill cover and thus pulling forces acting on the geomembrane.
- High puncture strength to minimize the probability of damages during placement of rock fill.
- Filter properties suitable for the grain sizes of the tailings and the rock crush.

A heavy woven geotextile was specified. Nonwoven geotextile did not appear appropriate in this case because of its very large elongation properties, and its generally lower strength.

On the basis of cost, a heavy woven polypropylene geotextile was selected with an equivalent opening size distribution as shown on the composite plot of grain size curves on Figure No. 5. The mechanical properties are listed below:

Mass	244 (g/m^2)
Tensile Strength (25 mm grab test)	1.78 (KN)
Elongation at max. load	>30%
Burst strength	3.5 MPa
Puncture strength	0.67 (KN)

DESIGN DETAILS

For the stability of the dam, a factor of safety of 1.4 was chosen when considering an intact upstream blanket. The most critical potential slip-surface was defined by the upstream impervious blanket, since the shear strength at the interface between geomembrane and silt was governed by a friction angle of 20°. This value had been obtained in previous studies for the contact between a smooth impervious polymeric liner and fine sand.

The extreme situation with the upstream impervious blanket leaking, was also investigated in a stability analysis, assuming that seapage would occur through the tailings core. For this possibility, the most critical case was governed by rapid-draw-down for a slip surface defined by the upstream blanket, resulting in a safety factor of close to one. Similar excess pressure conditions can be expected to occur during thaw of ice lenses immediately underneath the upstream membrane.

For the extreme situation of a leaking upstream blanket, a downstream toe filter was incorporated into the dam to keep the phreatic line well behind the downstream slope surface in order to minimize the danger of piping failure as well as to increase the overall stability of the dam.

Reinforcement with geotextile along the base of the embankment did not appear essential on the basis of stability considerations, as well as with respect to excessive lateral deformations in the underlying soft ground. It was believed that cracks developing in the fill would be evenly spread along its base and would soon close in the silty material.

Differential settlements between fill portions based on the rigid rock abutments and fill portions based on soft ground appeared to be more critical particularly with respect to the performance of the impervious membrane. Thus, near the attachments to the abutments, the membrane was placed in a trench (See Figure No. 6) to protect the connection against tension forces which might develop during large settlements adjacent to the rock abutment.

PERFORMANCE OF DAM

The dam was completed in Fall, 1981 and has been in operation since then, however, not yet for the proposed maximum pond water levels. A few minor cracks (less than 5 mm wide), and slightly ondulating ground along the dam crest were observed in fall of 1982, but did not appear significant enough to indicate excessive deformations of the dam. Water seepage from below a road embankment downstream of the dam which occurred in summer of 1982 could not be related to deficiencies in the dam. Nevertheless, in case water should be noticed again, it will be monitored by using dye or radioactive tracing substances.

Results from surface surveys were not available to date and therefore, no quantitative deformation records can be discussed.

LITERATURE

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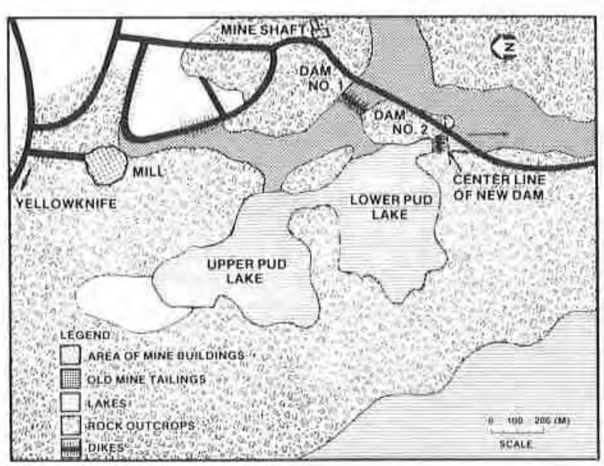


FIG. 1: SITE PLAN OF MINE

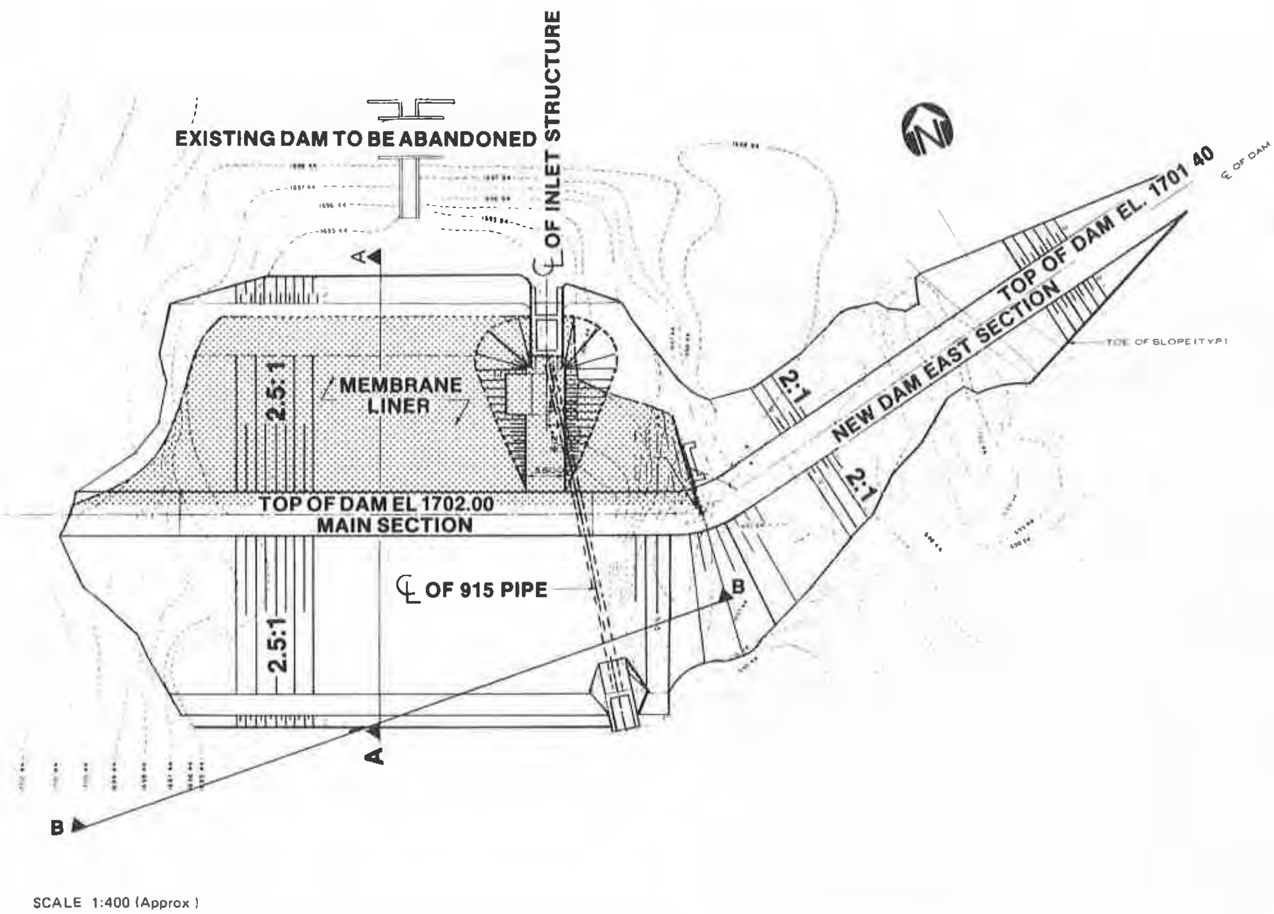


FIG. 2: PLAN VIEW OF PUD LAKE TAILING DAM.

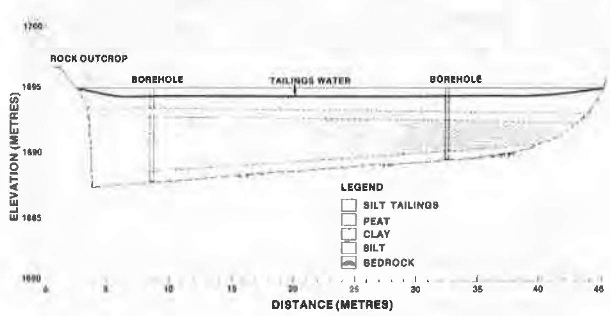


FIG. 3: GEOLOGICAL SECTION B-B

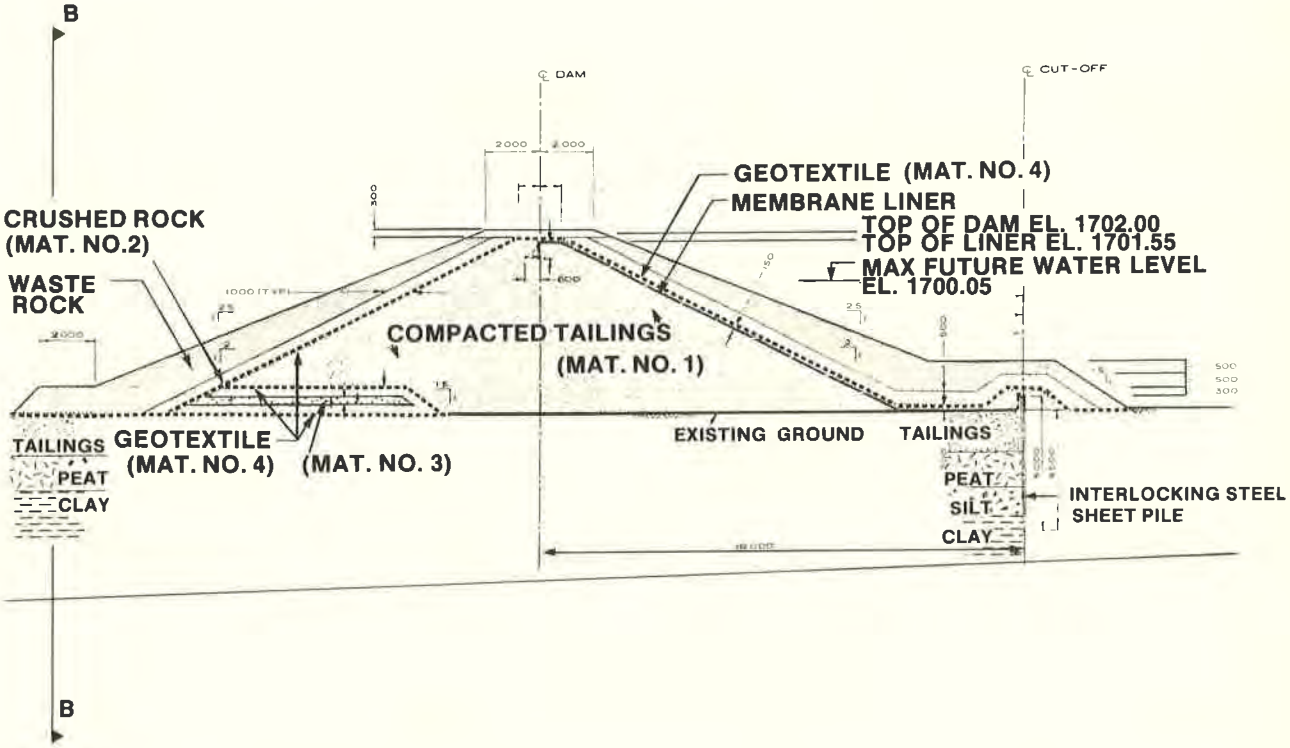
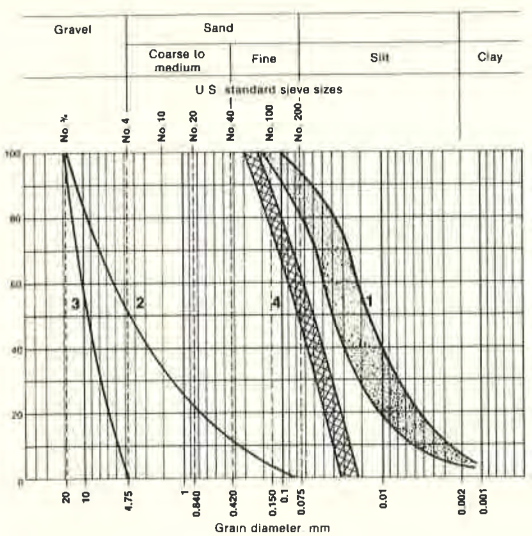


FIG. 4: TYPICAL SECTION OF MAIN DAM (SECTION A-A)



LEGEND

1. SILT TAILINGS
2. ROCK CRUSH
3. TOE DRAIN
4. GEOTEXTILE (equivalent opening size)

FIG. 5: COMPOSITE PLOT WITH GRAINSIZE DISTRIBUTION OF FILL MATERIALS.

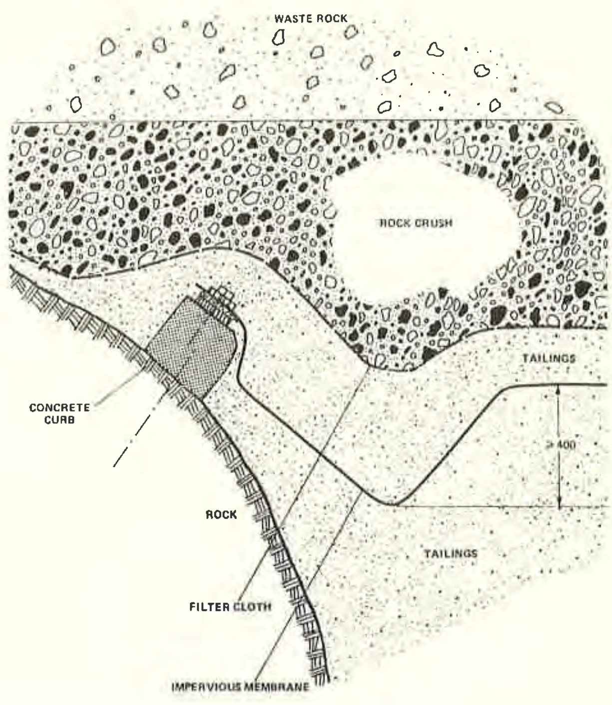


FIG. 6: DETAIL OF CONNECTION TO ROCK ABUTMENT

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Waterproof Covering for the Upstream of the "Lago Nero" Dam

The paper here presented is a report of an extraordinary maintenance intervention aiming to restore the absolute efficiency of the Lago Nero dam owned by E.N.E.L. (the Italian National Electricity Company). The dam is situated in the high valley of the Serio river in the Italian Alps, at an altitude of 2,000 m. El.

In addition to the restructuring of the main body of the dam, a project of complete waterproofing of the upstream face was engineered utilizing a totally original system that was subsequently patented. The system employed a PVC geomembrane assembled, pre-stressed and fastened to the main structure of the dam with an original device constituted by specially designed steel ribs.

1. INTRODUCTION

This dam was built during the years 1924/29 for an electric power company that was later acquired by the state. The capacity of the reservoir approximates 3,35 millions m^3.
The type of structure is that of massive gravity in concrete with a maximum height on the foundation of about 40 m, while the length along the crest measures 146 m. The surface of the upstream face measures about 3,500 m^2 and the capacity of the dam is of 35,000 m^3.

The main structure of the dam was constructed using a concrete mixture made with 250 kg. of Portland cement per one m^3 of concrete mix, while for the upstream portion of the draining system was made using a concrete mixture of 300 kg./m^3. During the concrete pour rocks were added in the ratio of 10% of the volume.

The dam was built on a bedrock (formed by) made of quartz porphyry alien from schistosity and presenting moderate diaclasis.

As a consequence of 1) lack of preliminary testing of the concrete and 2) poor construction procedures, the dam presented the problem of conspicuous leakage; via infiltration through the foundation plan and because of the porousness of the masonry.

Various interventions were taken in the past in order to reduce water loss. Concrete was injected in the bedrock and gunite was sprayed on the upstream face. These remedies proved themselves to be insufficient or just temporary.

The degradation of the structure had been increasing in time and had been further accelerated by the effect of the hardwater of the reservoir on the calcium in the concrete and by the disintegrating action due to freezing and thawing.

In the spring of 1975, E.N.E.L. engineered a project for the restoration of dam in order to improve the conditions of static safety and restore the waterproof property of the foundation and of the upstream face. Fig. 1 -

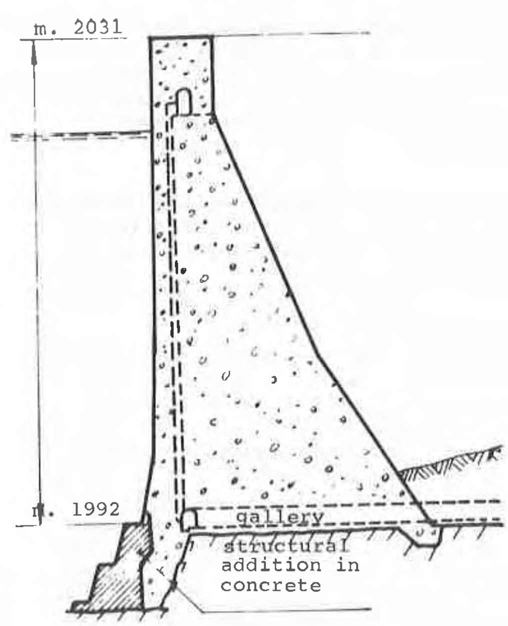

Fig. 1 Cross Section

The project, previously approved by the competent authorities, had been carried out during three working seasons in 1979 through 1981. The dam has undergone recent testing in the summer of 1983, after three experimental fillings of the reservoir.

2. The Geomembrane on the Upstream Face

The most interesting part of the intervention was the installation of the geomembrane on the upstream face. Other maintenance works were completed to prepare the body of the wall; 1) the foundations were reinforced, 2) the down stream face was restored, 3) the central portion of the wall was loaded, 4) the spillway enlarged, etc.

Proceeding to the waterproofing of the upstream face, E.N.E.L. took in consideration three alternatives:

1. Spraying gunite reinforced with a metal net.
2. The execution of an impervious membrane of steel plate.
3. The installation of a sinthetic geomembrane.

The first solution, quite traditional, did not present any particular problems of execution but entailed the complete demolition of the old protective layer and required a long time for completion.

In the specific circumstances of freezing temperatures and aggression of hard waters, a protective layer in gunite would not guarantee waterproofing but for a short time.

The second alternative, that had been proven successful in previous occasions, was discarded because of the high cost and the difficulties of execution in harsh weather conditions. Furthermore, this solution would require expensive maintenance such as frequent painting.

Because of the many drawbacks of the first two solutions, the third alternative was chosen. This solution only required some repairs to the plaster surface and offered, most advantageously, a low cost of the material and a considerably reduced time of execution.

In those years (1979) the support of analogous experiences in the field of restoration of impermeability of dams was quite limited. Previous references in Italy included the installation of a polyisobutylene geomembrane on the dam of Baitone (height = 42 m.) in 1969/71 and the installation of a PVC geomembrane on the dam of Miller (height = 10 m.) in 1976. The only known experience outside Italy had been done in West Germany in 1974 on the dam of Heimbach (height = 7.5 m.). In that case a PVC geomembrane had been used, as reported, in the ICOLD Bulletin #38.[2]

The two most relevant features considered in the making of the decision were; 1) the simplicity of installation, especially in the covering of the structural joints, 2) the rapidity and low cost of installation even in sight of eventual future repairs or total replacement of the geomembrane.

The company owner finally came to the conclusion that such an intervention would have given the way for the experimenting of new technologies being developed in the last ten years.

Furthermore, being the project of relatively small dimensions but presenting all the complexities of a larger one, it would have been an ideal, reliable model for future references.

The problems arising at the planning stage of the intervention were the following:

1. The choice of the synthetic material of the geomembrane.
2. How to proceed to installation on subvertical face.
3. The drainage of water condensation behind the geomembrane.
4. The protection against ice formation.

2.1. The Material of the Geomembrane

Among the numerous types of synthetic geomembranes presently available on the market, a selection was made on the basis of the description offered by the manufacturers.

The geomembranes selected were submitted to a series of tests in E.N.E.L.'s own laboratories in order to verify their main characteristics and investigate their qualifications for installation.

It was nevertheless acknowledged that experiments conducted in a lab could not present the entire set of circumstancial factors that may occur in real conditions where the material is used.

The material that was finally chosen was the SIBELON CNT 2800, a particular brand of extruded PVC flexibilized and stabilized, manufactured by ITALFORM, Milan, Italy. The following characteristics refer to the results of lab testing: Table 1. -

General Properties	Test Method	SIBELON
Thickness (mm)	ASTM D374-79	1,93
Tensile Test (MPa) Strength at break	ASTM D638M-81	18,0
Tensile Test Elongation at break (%)	ASTM D638M-81	285
Moisture Vapor Perm. (g/m^2. 24 h.)	ASTM D1653-72	3,2
Abrasion Test mm^3. /1000 turns)	ASTM D1044-78	60

Table 1. - General Properties of the geomembrane from the tests carrier out at the ENEL-CRIS Laboratory on Special Materials.

The geomembrane had proven to have good properties in resistance, deformability and welding potential. In addition to the above characteristics the previous successful experiences on the part of E.N.E.L. and others in the installation of the same material in similar circumstances, favored the choice.

As far as the aging of the geomembrane is concerned, CRIS (the laboratory) provided two sets of evluations:

a) Study on the deterioration of the mechanical properties

Tests were performed on a sample taken from the upstream face above the maximum water level after six years of exposure to high level UV. The material showed an average loss of 22% in resistance to traction and an average loss of 20% in elongation to the breaking point.

b) Study on the possible chemical alterations of the polymer

The analysis was conducted using an infrared spectrophotometer on the material after one year of exposure on a canal connected to an hydroelectric plant. This type of analysis, absolutely original, has provided precise indications on the chemical performance of the material. The plotted lines obtained from a sample of new material and a sample of the exposed one are strikingly similar, (fig. 3).

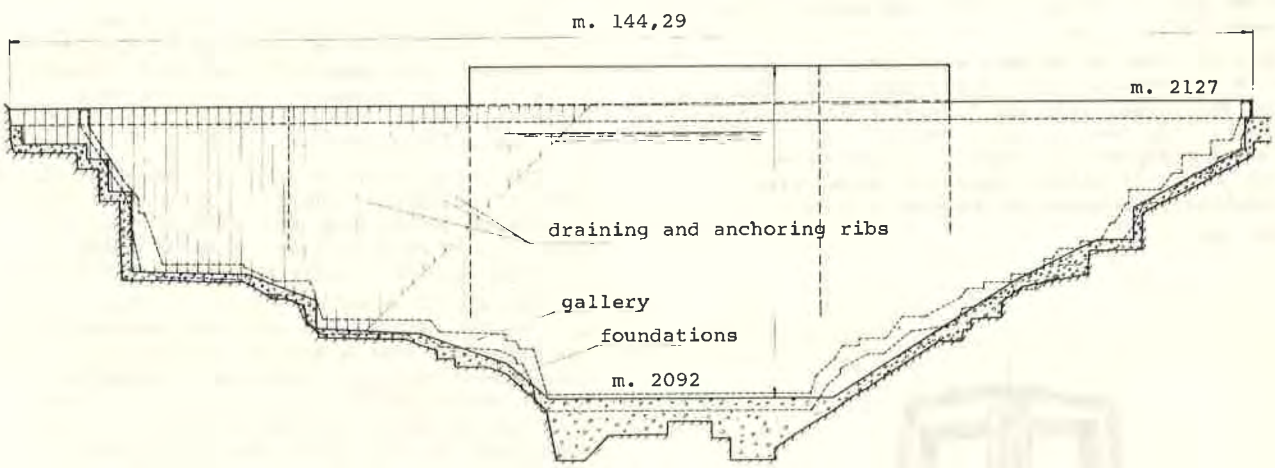

Fig. 2 View upstream face Lago Nero Dam

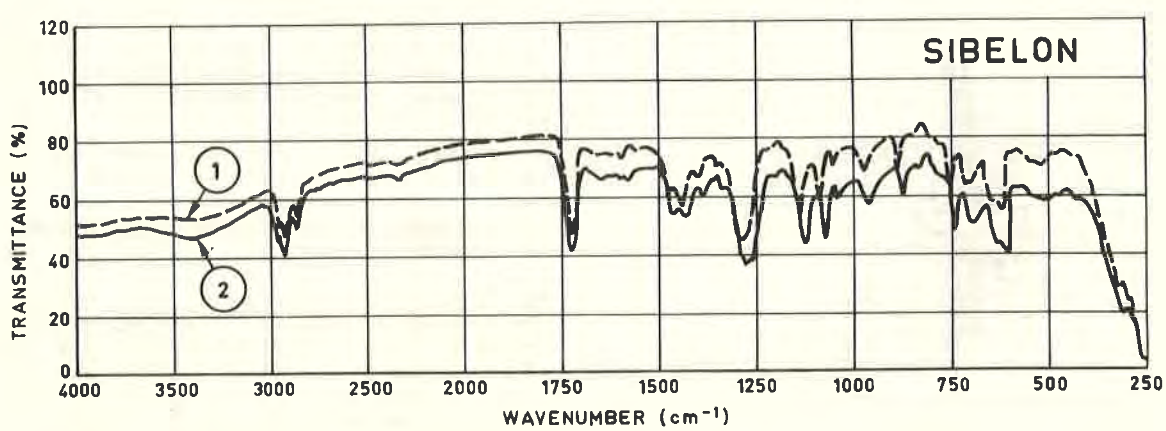

Fig. 3 Transmittance vs wavenumber by infrared spectrophotometer
(1) pvc membrane unexposed (2) pvc geomembrane exposed for one year

Therefore, no evidence of rupture of bonding nor any structural alteration of the chemical properties were found.

Dimensions and appearance of the geomembrane are as follows:

- Width - 2,05 m.
- Length; variable depending on the structure to be covered so that to avoid horizontal seams
- Color - light grey

A plyester geotextile was heat coupled as part of the manufacturing procedure. The geotextile measures 1,95 m. in width with a weight of 350 g/m^2.

The company finally came up with an original system, now patented, that allowed a continuous fastening along the vertical lines and a horizontal pre-stressing of the geomembrane. This system provided, among other advantages, to eliminate the sagging due to the weight of the geomembrane.

Both inconveniences were eliminated at once with the solution of a continuous overlapping and fastening along the lateral ends of the sheet. The fastening was devised with a system of clamping the sheet's borders between two specially molded (designed) steel ribs previously fastened to the upstream face. Fig. 4,5.

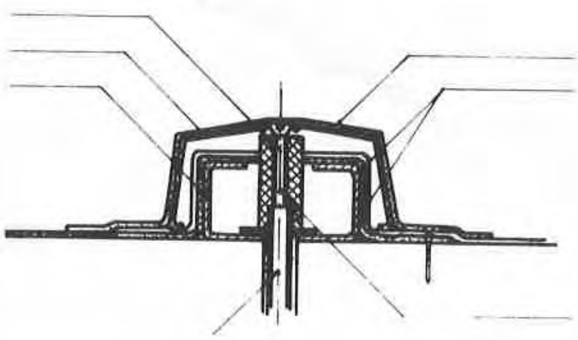

Fig. 4 - Detail A - Draining & anchoring ribs

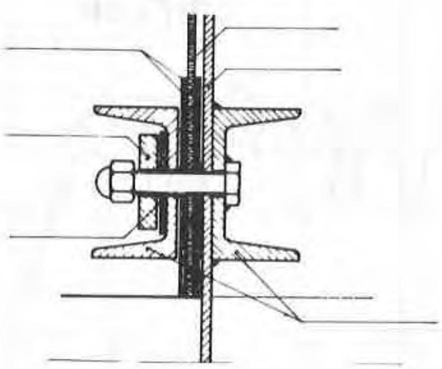

Fig. 5 - Detail B - Anchoring system at the base of dam

The system was preventively tested on a model and proved to resolve all of the problems of installation that other systems were presenting.

Particular attention was devoted to the waterproofing of the base of the upstream face. A structural addition in concrete was poured (all) along the inferior borders of the dam body where a system of metallic plates and anchors were pre-installed in order to provide a mechanical fastening of the geomembrane to the wall.

The above fastening system reproduced in scale and tested at 2M Pa pressure confirmed the absolute water tightness of the device.

The installation was performed employing self-hoisting platforms secured to the dam's crest and took 90 days for completion.

The installment procedure followed precise steps:

1. The steel ribs (60 x 30 x 2 mm.) were lined up vertically and anchored to the upstream face using specially designed threaded bolts of 10 mm. diameter. The bolts were secured in holes in the wall containing a resin that hardens upon contact.

2. The geomembrane was unrolled from the top down to the base of the dam. Each sheet was overlapped to the next for about 0,25 m. and subsequently welded along the strip which is not coupled with geotextile (about 0,10 m.).

3. The second steel rib, with a section of an open "C", was placed over and fastened to the first one with a system composed of a cone-shaped "Brugola" screw and a threaded bolt screwed into a tall threaded nut. Such a system performed the double function of a continuous vertical anchorage and of prestressing the geomembrane horizontally. (Fig. 4, det. A)

4. Strips of PVC were applied over the entire length of the vertical ribs and welded on both sides. The strips measured 0,40 m. width and 2 mm. thickness.

5. The lower ends of the sheets were fastened to the base of the dam using the procedure as described above, which is, employing metal plates plus a special trimming of semi-soft synthetic rubber.

6. Finally, all of the seals were carefully checked with a steel pricker.

2,3 Drainage and protection of the geomembrane

An efficient draining system is essential to the good functioning of the waterproofing system in order to eliminate the accumulation of water from condensation and from filtration. In this case, the system of the metallic ribs also served as the main drainage system (fig. 4, det. A). In fact, through a series of small holes along the side of the ribs this structure was engineered to also serve the function of regular draining pipes.

The graphs in figures 6,7 illustrate the patterns of permeability on the surface (Kp) and of the transmissivity θ obtained with the geotextile under a pressure varying from 0.05 to 0.5 Pa. The tests were performed with the equipment devised by CRIS and described in the paper by Cazzuffi and Puccio.

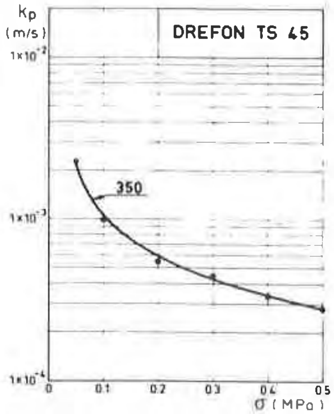

Fig. 6 Log. permeability constant Kp for the geotextile Vs. normal pressure 0

The geotextile also provided a protection against eventual punctures possibly caused by the coarse surface of the gunite. The polyester geotextile employed underwent special resistance tests performed at the laboratories of E.N.E.L.

2,4 The anti-ice system

Even though laboratory tests excluded the possibility of ice adherence to the PVC geomembrane, an anti-ice device was predisposed in order to prevent the formation of large ice blocks that could have produced lacerations. The anti-ice system would create a water movement in the proximity of the upstream face by the means of a continuous erogation of compressed air.

The system in this particular project was constituted by 13 distribution pipes complete with erogation valves adjustable from the perimetral tunnel inside the base of the dam.

3) Results and Conclusions

Upon the completion of all of the restorative works, several experimental fillings were performed under strict control. A systematic monitoring of all of the physical data having relevance to its static behaviour and performance was also carried out.

Starting in September 1981 two cycles of partial fillings and drawdowns of the reservoir were performed and at the end of June 1983 the reservoir was filled to its entire (full) capacity.

As regards the overall water tightness, daily readings of the losses were taken. Losses from the draining pipes behind the geomembrane were evaluated along with those found in the draining collector.

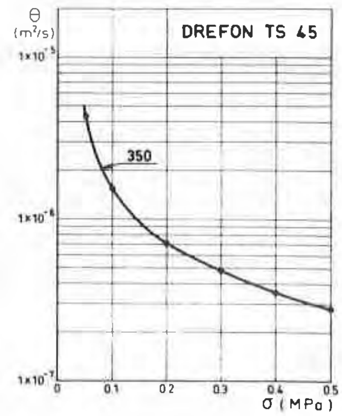

Fig. 7 Log. transmissivity 0 for the geo textile Vs. normal pressure 0

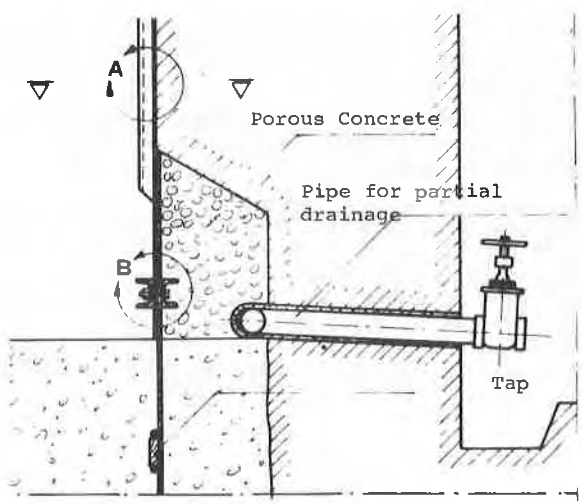

Fig. 8 Detail of draining system at the base of the dam

Maximum losses of 2,7 l/s were recorded in relation to periods before and after waterproofing of the dam and its foundation, one can conclude that the intervention was fully successful.

Particularly, a specific evaluation of losses recorded in the draining pipes shows an excellent performance of the PVC geomembrane. In fact, losses attributable to these drains which amount to 0,39 l/s equal only 14% of the total losses. These losses, on the other hand may be the results of infiltration through the perimetral base of the dam.

Whenever the reservoir was drained systematic site inspections were carried out to insure the absence of yieldings and loosening of the geomembrane and/or any other possible alterations.

All weldings were found to be in perfect efficiency; the entire lining was also found to have a complete adherence to the concrete surface even in areas under high hydrostatic load.

Winter inspections have shown the functioning of the anti-ice system which kept the ice surface at a minimum distance of 15-20 m.

On the basis of these results it can be concluded that the application of this innovative system can be very successful, provided that proper attention is given to all details.

This new technology represents the solution to the problem of waterproofing concrete dams with subvertical face, employing PVC geomembranes.

ACKNOWLEDGEMENTS

Our thanks to the "Centro di Recerca Idraulica e Strutturale dell'ENEL di Milano" (and in particular to Messrs. P. Bertacchi, P. Eng. and D. Cazzuffi, P. Eng.) for allowing mention of their tests.

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Geotextiles and Geomembranes for Rockfill Dams: An Application in Saudi Arabia

Precipitation delivers about 120,000 Million Cubic Meters per year of fresh, clean water to the Kingdom of Saudi Arabia; about 99% of this is lost as runoff to desert or sea, or as evaporation. Water demand for 1985 has been estimated at 3600 Million Cubic Meters, most of which will be met by depleting nonrenewable groundwater or fossil-fuel resources. The need for accelerated programs to increase artificial recharge in the streams of Saudi Arabia has, therefore, been recognized. Within the Saudi Arabian context the feasibility is being studied of constructing a large number of simple rockfill embankments, reinforced with geotextiles against failure due to thro ughflow or overtopping; the added feature of installing an impermeable geomembrane permits flexibility in shifting the infiltration pattern from the downstream channel to the upstream reservoir. The paper reports on model- and prototype-experiences and evaluates the potential hydrologic impact of a large-scale recharge program.

1. INTRODUCTION

Water is not one of the natural resources of which Saudi Arabia has an abundant supply. Although several resource surveys have confirmed that large amo unts of water are in storage in the Kingdom's deep aquifers, the salinity of those waters frequently exceeds 3000 ppm, and expensive treatment is required to render them useable. Therefore, the basic choice for local water managers is between tapping to the fullest extent the very limited renewable supplies, represented by sporadic and highly variable flood flows, or developing fossil oil and water resources. The latter approach has the advantage of both simplicity and technical feasibility, but government policy is now beginning to lean more towards costeffectiveness and self-sufficiency in basic commodities,including water. Hence, the concept of full utilization of renewable and rechargeable flood waters has achieved new relevance, and the related question of how to minimize costs suggests that new construction techniques should be given full consideration. This paper briefly describes the operation of a reinforced rockfill dam as a recharge structure and it reports on the preliminary findings of a related research project now in progress at the University of Petroleum and Minerals, Dhahran, Saudi Arabia. The purpose of the project is to utilize, as much as possible, geotextiles and geomembranes and other locally available materials and manpower in an effort to conserve relatively rare surface waters.

2. HYDROLOGIC SETTING

Average annual rainfall in Saudi Arabia is about 50 mm, with some regions receiving virtually no precipitation at all for periods of years. Even in areas where the supply is relatively plentiful (300 to 500 mm/year) most of the runoff has a flash-flood character and significant volumes of precious surface water reach the desert sands, or the salty waters of the Red Sea or the Gulf, and thereby they are removed from the realm of "beneficial use".

Saudi Arabia derives most of its water supplies from groundwater, or from seawater through a variety of desalination processes. Desalination requires substantial fossil fuel inputs, which means that such non-renewable energy resources need to be utilized directly rather than sold to other users. Much of the groundwater is derived from deep aquifers which were recharged thousands of years ago; under present climatological conditions only shallow alluvial aquifers with hydraulic connections to intermittent streams are being re-plenished. Hence, the deep groundwater is also "fossil" in nature.

Since the Government's Development Plan (1) stresses the need to conserve non-renewable or fossil resources, the hydrologic consequences of this policy inevitably point to increased utilization of recharge schemes to capture un-utilized flood waters.

Although paucity of streamflow records does not allow for an exact analysis of surface water availability, a conservative estimate of 24,000 million cubic meters per year is based on the following assumptions :

a) Average precipitation of 5 cm.
b) 80% losses due to evaporation.

Such a resource would still amount to 10 times the 1980 country-wide needs, as estimated in the Third Development Plan (1).

The uncontrolled flow of a flood along an otherwise dry stream channel will yield a certain infiltration benefit during its relatively short exposure time (See Fig. 1,a). Construction of a retaining structure will enhance the infiltration rate upstream, while still allowing for some recharge below the dam during periods of spills. (See Fig.1,b). A retarding scheme is likely to cause a more pronounced infiltration pattern, since it combines increased upstream infiltration with the

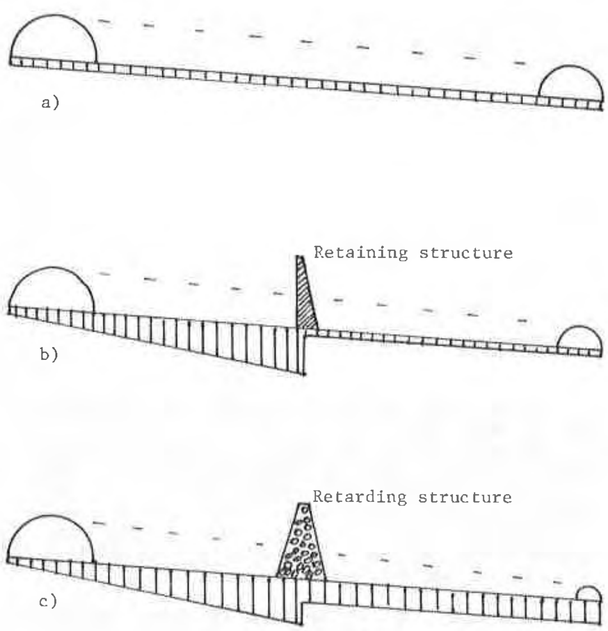

Figure 1. Recharge patterns.

opportunity of enhanced downstream recharge of the waters percolating through the permeable retarding structure (See Fig. 1,c).

Since providing a good sealing effect sometimes makes up a substantial portion of the construction cost of a dam, it is clear that the adoption of permeable recharge structures has not only hydrologic, but also economic attractions.

3. REINFORCED ROCKFILL DAMS

Although the concept of reinforcing rockfill embankments has not won widespread acceptance, its history dates back almost 5 decades. Early design and construction measures, introduced in Mexico for the purpose of protecting rockfill dams under construction between 1936 and 1943, were described and illustrated by Weiss (2).

This pioneering work was followed by several laboratory investigations, most prominently those of Escande (3), Wilkins (4),(5), Parkin et al. (6) and Olivier (7). A comprehensive comparison of model test findings and performances of several full scale dams was given by Leps (8).

Only on rare occasions does the literature provide examples of recent reinforced rockfill designs. One such case involves an afterbay dam in California designed to smooth out power plant discharges and described by Shackelford et.al. (9). Another application was concerned with protecting a rockfill dam against failure due to overtopping during construction (10). In both cases grids of steel bars were used for reinforcing purposes.

The potential sites in Saudi Arabia for construction of reinforced rockfill are in relatively remote areas,necessitating considerable transportation costs for all non-indigenous materials since those supplies are only fabricated in, or imported through, a limited number of coastal cities. The Government is keen on developing local petrochemical industries and the list of proposed products includes basic materials for the production of geotextiles and geomembranes. Against this background it appeared reasonable to incorporate geotextiles, preferably locally produced, into the design of flow-retarding reinforced rockfill structures. Consequently, a research effort was initiated, which is still in progress, to develop a design adapted to local conditions and circumstances.

4. LABORATORY EQUIPMENT

Since geotextiles may offer greater flexibility in field installation it was deemed desirable to evaluate in the laboratory whether any placement patterns other than the horizontal layers in the toe region, and the surface grids normally used with steel reinforcement, might be more appropriate. A series of test runs was conducted in a 30cm laboratory flume, using test embankments 20 cm high with side slopes of 1:1.5. The void ratio of the clean, practically monosized rock was 0.55, and based on an expression given by Wilkins (4) the average void velocity was found to be 18 cm/sec. Flexible,permeable sheets of coarse geotextile fabrics were used to simulate reinforcement and the embankments were subjected to throughflow and overflow loads. Generally, the reinforced areas were observed to settle and be rearranged into elongated, oval,flattened rolls, which thereafter displayed considerable stability. At times the resettlement process created relatively large void spaces in the interior of the model. Figure 2 illustrates a typical model run near full discharge capacity.

Figure 2. Reinforced model dam subject to throughflow.

Initially the reinforcing was limited to the toe area, up to about one third the height of the model. At low flows this proved to be satisfactory, but as the upstream water levels approached the crest elevation, the above-mentioned settlement of the reinforced toe occured and frequently this provided sufficient disturbance of the downstream slope, combined with high exit velocities, to cause a sudden slide failure of the upper portion of the downstream slope, with most of the material being washed away rapidly. Sometimes the removal of this overburden then lead to subsequent unrolling and disintegration of the reinforced toe. Figure 3 illustrates this chain of events.

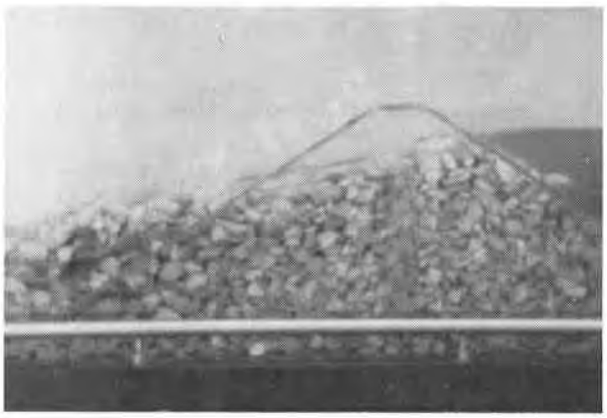

Figure 3. Sliding of upper part of the downstream slope.

In view of the contributory significance of the high exit velocities it appears logical to reduce them,by flattening the downstream slope or by installing an impervious membrane in the body of the dam, thereby increasing the overflow/throughflow ratio. The laboratory equipment is therefore now being modified to accomodate a revised testing program.

5. PROTOTYPE PHASE

The Saudi Arabian Ministry of Agriculture and Water is the governmental agency primarily responsible for safeguarding the Kingdom's water resources, and as such the construction of dams for storage and recharge purposes comes within its purview. The Ministry is now actively reviewing the feasibility of including one or two prototype structures in its dam construction program. The proposed embankments will be about 4 meters in height, with geotextile reinforcement placed as shown in Figure 4. Rockfill materials will most likely be obtained from the stream beds, without the need for quarrying. The choice of the geotextile material has not yet been made; it will largely depend on commercial availability, and on a minimum required delivery time.

Since the Ministry carries out an extensive maintenance program with its own construction crews, the possibility of in-house project execution is being considered. This would eliminate the need for the usual tender submittal and evaluation procedure, and it might bring construction forward to the 1984 summer flood-free season.

The main difference between the two prototypes will be that one will be equipped with an impervious geomembrane parallel to the upstream face, primarily to evaluate, under flood conditions, the resistance of the downstream face against raveling and surface erosion. The use of a downstream rock pavement, underlain by a porous geomembrane, and mechanically secured by fabric matting or another cell-type matrix, may be used as a means of safeguarding earthfill embankments against failure by overtopping.

Basic instrumentation envisaged for the prototypes includes a network of benchmarks to monitor differential settlement, and recording water level gages upstream and downstream to study the hydraulic operation. In addition, direct measurement of recharge efficiency by means of piezometers will be attempted.

6. CONCLUSIONS

At this stage of the investigation it appears to be technically feasible to make field installations of coarse geotextile matting or coarse woven fabrics for the purpose of protecting simple rockfill embankments against failure due to overtopping. Flexibility of the materials is an advantage where settlement is likely to occur.

Corollary installation of impervious membranes, in order to alter the throughflow water patterns may

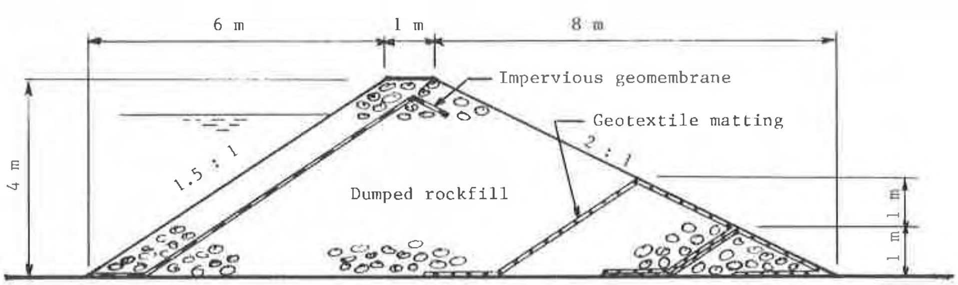

Figure 4. Typical section of prototype rockfill dam.

be advisable from a stability point of view.

Major issues not adequately settled include the economic feasibility of the technique in remote areas of Saudi Arabia, and the degree to which unskilled field crews will be able to implement proper field installation.

ACKNOWLEDGEMENT

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Design of Dam Using Geomembrane—Geotextile Sandwich, Robertson Lake Project, Lower Northshore—Quebec

Design considerations for a rockfill dam over 40 m high, located in the north-eastern part of Quebec, using a synthetic membrane sandwich as an impermeable barrier are discussed. The sandwich is composed of a central impermeable geomembrane bonded between two geotextile layers, glued in the factory.

The main body of the dam shall be composed of rockfill. The impermeable sandwich shall be placed on the upstream face of the dam, on a supporting layer consisting of crushed stones. A transition zone separates the main body of the dam and the supporting layer. The sandwich is protected against ice, waves & wind by rockfill layers. The sandwich is anchored in the bedrock at the toe of the dam; at the top, the sandwich is anchored in the crest.

INTRODUCTION

The Robertson Lake Project is situated on the Lower Northshore of the St-Lawrence River, some 1200 km north-east of Montreal. The main features of this 21 MW low head hydro-project are : a main dam about 40 m high on the Ha! Ha! River; a small dyke approximately 10 m high; an upstream cofferdam about 15 m high; an open-air powerhouse located at the downstream toe of the dam; an intake channel; an intake structure together with two penstocks, located on the right bank of the river (fig. 1)

The construction of the project is slated to start at the end of summer 1985 (mainly camp site and road preparation), and to be terminated in December, 1987.

Technical and economic considerations led to the adoption of a rockfill dam with upstream impermeable synthetic sandwich revetment, as the main water - retaining structure of the project. The impermeable sandwich is composed of a central geomembrane bonded between two geotextile layers.

The relatively important height of the dam, the prevailing climatic conditions, lack of previous experience with similar materials and a large variety of geomembranes available in the market required an extensive research program, carried out at the Ecole Polytechnique Montreal, to determine the mechanical properties of several different types of geomembranes and their behaviour under severe climatic conditions. Results of this study are presented in an accompanying paper.

SITE CHARACTERISTICS

The average temperature is 12.5° in August & -11.5° C in January. The average maximum temperature of 15.5°C is recorded in August, while the average minimum of -16.5° C is observed in January. The absolute maximum and minimum temperatures are : 30°C and -35°C respectively. The freezing index of the area is situated around 3000 degree (F) - days (3).

This area belonging to the great canadian shield is mainly rocky with rugged terrain. The overburden is almost inexistent. Different types of Precambrian Gneiss predominate the rock type. The bedrock is moderately fractured on the surface (3 to 5 m) but generally sound with a few closed joints thereafter.

At the location of the dam, the Ha! Ha! River about 20 m wide and 1 to 3 m deep cascades over a bed of boulders ranging between 0.5 to 2 m in diameter. The right abutment of the dam rises gradually above the river with a 30° slope. The left abutment is steeper with an average slope of about 45°. Both abutments are formed by sound gneiss bedrock with a thin cover (1 m$\pm$) of overburden. The thickness of the boulders in the riverbed, at the location of the dam, is estimated to vary between 2 and 5 m. The bedrock outcrops in the riverbed near the left abutment, upstream of the dam. An inclined borehole drilled at the downstream toe of the dam indicates the presence of rock at 6 m depth in the center of the river, eliminating the possibility of a deep burried valley (figs. 1 & 2). A fault located near the left abutment and possibly dipping downwards into the left abutment, follows the

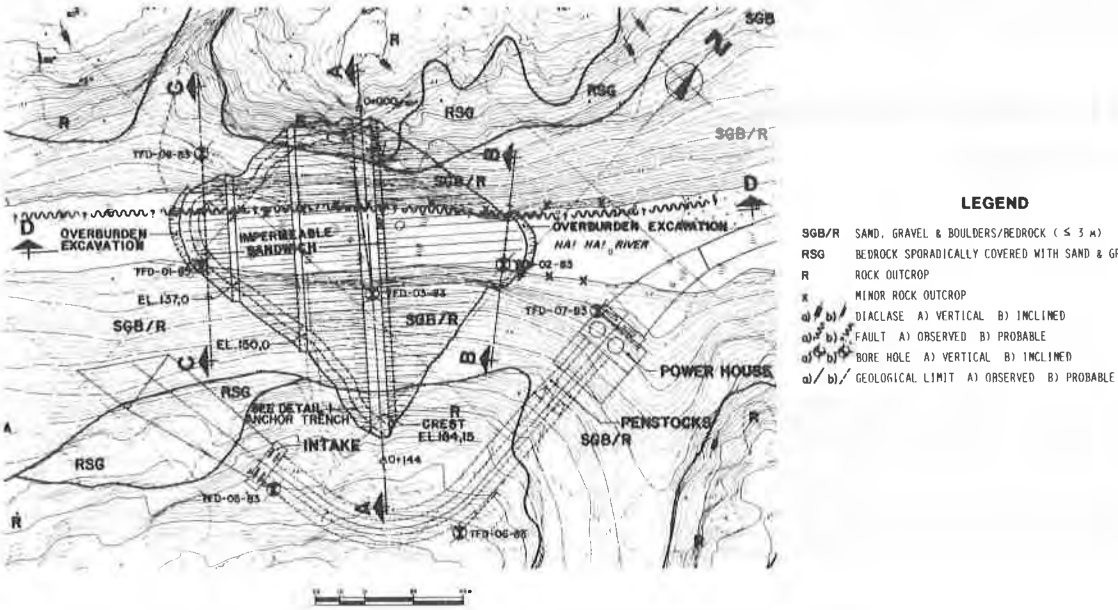

FIG. 1 PLAN VIEW OF MAIN DAM, INTAKE, PENSTOCKS AND POWER HOUSE

river over its entire length and then continues onward over a very long distance. The shear zone (fig. 2) consists of extremely fractured rock with the presence of silt seams. Water absorption determined by Packer Test in the shear zone was rather small: 0.56 l/min/m, probably due to the presence of silt which fills the fissures in the rock. The first 3 to 4 m of the bedrock is slightly fractured and thereafter it is generally sound.

CONSTRUCTION MATERIALS

None of the construction material is readily available either for the construction of the dam or the manufacturing of concrete, at the site. Rockfill for the water-retaining structures (main dam, dyke and cofferdam) will be available partly from the rock excavation (powerhouse, penstocks, intake channel, diversion channel, etc.) and partly from rock quarry. Coarse and fine agregates for concrete, sand and other fine rockfill will have to be manufactured at the site.

CHOICE OF DAM TYPE

The choice of the dam type was based on the following factors :

1) Total absence of impervious fill;
2) Foundation of the dam;
3) Climatic conditions.

Several types of dam, as enumerated below, were studied :

1) Rockfill dam with a central bituminous concrete core;
2) Rockfill dam with an upstream concrete facing;
3) Concrete arch dam;
4) Rockfill dam with an impermeable synthetic sandwich, composed of a central geomembrane bonded between two geotextile layers placed on the upstream slope of the dam (fig. 3)

The last solution, rockfill dam with a synthetic impermeable sandwich, was retained after the cost estimates indicated this to be the most economical. Furthermore, the simple construction techniques required for this type of dam provide an additional advantage when one considers the remoteness of the site.

Although several dams of this type have been constructed around the world (France, Czechoslovakia, USSR, etc.) (1, 2, 4, 5, 6 & 8) but none has so far been constructed in North America. The highest dam of this type measures about 27 m. The ICOLD report on the Use of Thin Membrane on Fill Dams (6) recommends to limit the height of this type of dam to 30 m. The sound rock foundation, the unavailability of construction materials and the technical considerations led to the adoption of this solution for the dam which measures slightly over 40 m in height. It is to be noted that the two other water-retaining structures (cofferdam and the dyke) will also employ synthetic sandwich as the impermeable barrier.

DESIGN CRITERIA

A) Embankment

The design and construction of a dam incorporating an impermeable sandwich, composed of a central geomemgrane bonded on both sides by geotextiles, as its impervious element require the consideration of some particular design criteria and adoption of some special construction procedures. Thus for the Robertson Lake Project the following aspects were taken into consideration :

a) The stability of the dam should be maintained in the event the sandwich (geomembrane) is punctured and a substantial leakage occurs through the sandwich. Consequently, the main body of the dam should be built with a well compacted free-draining rockfill which would remain stable even under large flow discharge.
b) To further increase the dam's stability, the impervious element (sandwich) should be placed on or near

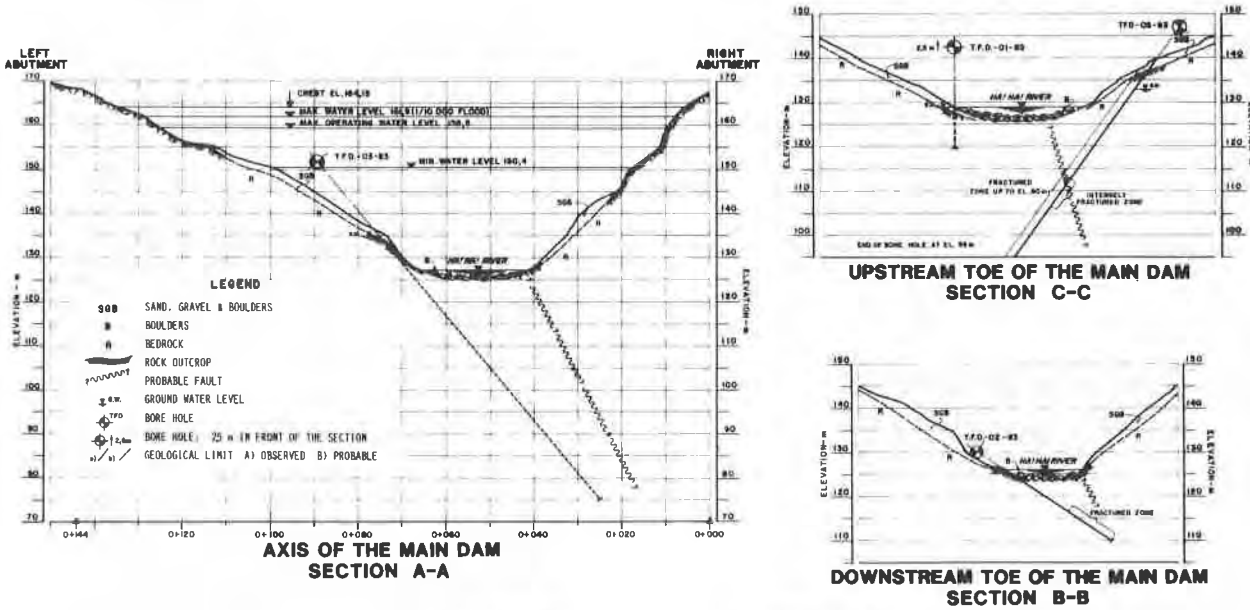

FIG. 2 PROFILES ALONG AXIS, UPSTREAM & DOWNSTREAM TOES OF THE MAIN DAM

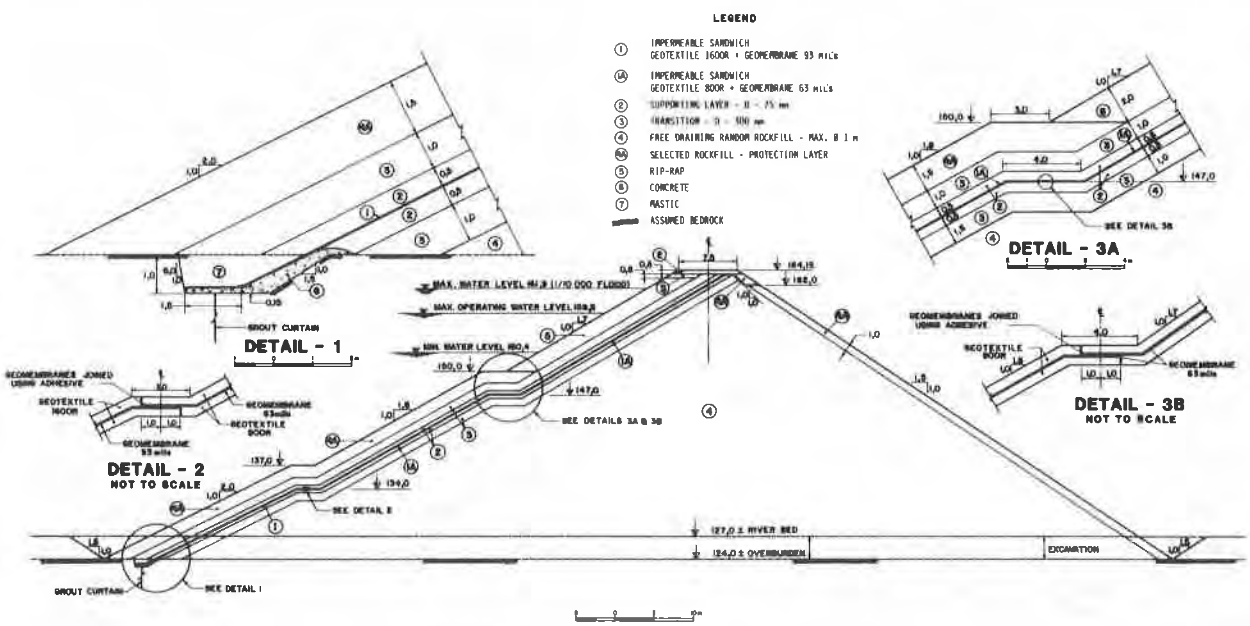

FIG.3 TYPICAL MAIN DAM CROSS SECTION (SECTION D–D)

the upstream face of the dam. Finite element analyses have shown that for dams with impervious membrane placed on the upstream slope of the dam the major principal stresses in the body of the dam are almost vertical, and the deformations which follow as a result of the hydrostatic thrust are small and in an almost horizontal direction. With the membrane placed in the center the filling of the reservoir has two effects : the upstream part of the dam up to the impermeable sandwich is submerged and thus intergranular stresses are reduced. The hydrostatic load is applied to the center of the dam and, therefore, is resisted only by half of its volume. This produces a relieving of the pressure in the upstream slope where the major stress is reduced, and an increase in the intensity and the obliquity of this stress in the downstream slope. The upstream part tends to rise up, while strains in the downstream part gradually become horizontal. A markedly disuniform state of stress and deformation thus result.

The explanation of what happens with a central sandwich is quite simple. The sandwich under the hydrostatic thrust tends to move in a downstream direction. In analogy with the earth pressure theory on a retaining structure, the sandwich induces a passive thrust in the downstream shell and an active thrust in the upstream shell, where failure may occur as has been verified in practice.

The consequences are, generally, small carcks with slipping of the material towards the bottom, in the highest zone of the upstream slope.

Thus the necessity of flattening the upstream slope to limit the influence of this decompressed zone.

c) The upstream slope of the rockfill under the sandwich should be limited to a maximum of 1V : 1.7H, even if from stability view point it seems to be more than sufficient, to facilitate the installation of the sandwich; this is about the steepest slope on which a man can work without support.

d) The membrane should be installed on the slope, once the rockfill and the supporting layer have been completed, to avoid undesirable stresses in the membrane due to differential settlements of the embankment.

e) Due to lack of experience with this type of dam over 30 m high, further safety measures should be introduced by dividing the upstream slope into three different slopes each 13 m high, with two intermediate berms (fig. 3). The reduction in the length of the slope has the advantage to :

- reduce shear stresses between the geotextiles and the geomembrane;
- reduce shear stresses between the sandwich and the overlying protective layer;
- reduce creep
- keep the size of the roll small enough to facilitate its transportation and manipulation, keeping in view the remoteness of the site.

f) The impermeable sandwich should be placed on a supporting layer, constituted of a well graded sand and gravel, not susceptible to internal erosion and compacted along the slope. This layer, 50 cm thick, should have a permeability of at least 10^{-2} - 10^{-3} cm/s. The object of this layer is to avoid pore pressure build-up under the sandwich and to minimize the danger of puncturing by large angular or subangular stones.

B) Geomembrane

The impermeable element to be used for water-proofing the main dam at the Robertson Lake Project shall be composed of a geomembrane-geotextile "sandwich", i.e. the impermeable geomembrane shall be bonded between two geotextile layers. The main reasons for forming a geomembrane-geotextile sandwich are to :

- provide additional protection against puncturing of the geomembrane by angular and subangular particles of the fill on which it shall be placed;
- minimize the danger of tearing or puncturing of the geomembrane during transportation and placement; (5)
- eliminate the pore pressure build-up immediately under the geomembrane, since the geotextile acts as a drainage layer;
- act as insulator, reducing the damaging action of freezing-thawing cycles;
- transfer part of the load induced by sliding of the overlying protective layer to the geotextile, taking advantage of its relatively high tensile strength
- facilitate the stress distribution underneath the geomembrane. The presence of the geotextile between the geomembrane and the supporting fill acts as an equilizing element. Equalizing elements have three beneficial effects. First, they tend to soften and enlarge the outline of the sharp points of early contact between geomembrane and support, reducing the danger of punching failure. Second, they tend to fill up the most depressed areas of the granular support, allowing the geomembrane to find a continuous and complete support at a lower average deformation than that which would correspond to the bare geomembrane. The geomembrane therefore does not reach the level of permanent deformation. Third, the equalizing element provides a deformable medium between the membrane and the support, which tends to eliminate points of fixity of the membrane and renders much easier and more rapid the transfer of force from the most stressed areas towards those under less stress, and thus an acceptable equalization of stresses takes place.

The geomembrane, the main element of the impermeable geotextile-membrane sandwich, and the sandwich as a whole must satisfy the following requirements :

a) Imperviousness : While large water losses cannot be allowed, seepage discharge around 0.00125 l/s/m^2 can be tolerated. Consequently, for a maximum hydraulic head of 40 m, the geomembrane should have a maximum coefficient of permeability of 10^{-10} cm/s. This requirement applies equally to the joints.

b) Thickness : While no precise rules exist to establish the minimum thickness of the geomembrane, current practice and experience recommend a minimum thickness of 1.5 - 2.00 mm (60 - 80 mils) for dams higher than 20 m. The selected thickness should be verified by burst test under a hydrostatic pressure of at least 300 - 400 kPa.

c) Tensile strength : Currently manufactured geomembranes have a tensile strength in the range of 10 - 20 KN/m, which is insufficient to resist tensile stresses induced by the sliding of the protective layer. Thus the design criteria should ascertain that no sliding takes place at the sandwich-soil interface. This requires that the slope of the fill on which the sandwich shall rest is less than its friction angle (minimum safety factor : 1.5). As a supplementary safety measure, the installation of a berm at the toe of the slope should also be considered.

d) Resistance to puncturing : The geomembrane should be protected on both sides against puncturing due to the presence of angular particles. This criterium requires the installation of geotextile layers on both sides of the geomembrane and an appropriate granular composition of the supporting and protective layers. The geomembrane alone should have sufficient strength against puncturing under the imposed loads.

e) Resistance to freezing and thawing: The relatively severe climatic conditions at the project site (min. temperature : -35°C) could subject the membrane to frequent and large freezing and thawing cycles. However, since the sandwich shall be burried under at least 3.0 m of protective layer and the major part of the membrane shall be permenantly submerged under the reservoir, the frost action is limited only to the membrane above the minimum water level.

The geomembrane above the minimum water level should fulfill the following requirements :
- maintain an adequate flexibility without cracking for temperatures ranging from $0^{o}C$ to $-35^{o}C$;
- maintain adequate tensile and burst strengths, for this temperature range;
- strain at rupture should not be more than 5% at $-35^{o}C$ and not more than 10% at $-20^{o}C$.

f) Geomembrane - geotextile sandwich : To facilitate the installation and to ensure a high quality of bonding, the geomembrane should be bonded to the geotextile layers in the factory and delivered to the site in rolls of 8 to 11 m widths. The bonding should be sufficiently strong to :
- avoid unbonding during handling and placement;
- ensure that the shear strength between the geomembrane and the geotextiles is at least equal to the shear stress between the sandwich and the adjacent soil layers;
- resist damage due to the freeze-thaw cycles, over the life of the project;
- resist high pore pressure without unbonding;
- have a minimum life of 50 years (no chemical deterioration) under the protective fill.

To minimize the number of joints to be executed at the site, specially under difficult climatic conditions, the sandwich should be delivered to the site in widths of 8 to 11 m, and minimum length of 30 m.

g) Joints : Horizontal joints should be allowed only on the intermediate berms. The tensile strength of the joints should be equal to that of the geomembrane, and should resist hydraulic pressures of at least 400 kPa. The bonding process should not damage the two geomembranes and should not deteriote with time.

h) Durability : The impermeable sandwich should be stable under ultra-violet rays (the membrane being buried under the fill, this requirement does not play an important role), various chemicals (the remoteness of the site does not impose this requirement), biological action (micro-organismes). The storage conditions (humidity, frost, heat, U.V., etc.) prior to installation should also be taken into consideration for the geomembrane - geotextile sandwich selection.

CONSTRUCTION PROCEDURES

Foundation Treatment

After the temporary diversion of the river is completed, the excavation of the overburden material in the river bed shall be carried out. The present design envisages, due to lack of strength parameter of the overburden materials, the excavation of the overburden over the entire width of the dam. This decision will be finalized once the river bed is dried and an assessment has been made regarding the overburden properties.

If the overburden is judged to have sufficient strength to carry the dam without any major deformation ($\leq$ 1%), the excavation of the overburden shall be limited to an area upstream of the main rockfill (zone 4) (fig.3)

Grouting of the bedrock and specially the shear zone shall be carried out from the bottom of the anchor trench (fig. 3). The grout curtain shall be spaced 3 m c/c where the height of the dam exceeds 23 m; the depth shall equal half the head, minimum being 12 m. Elsewhere, the spacing shall be 6 m c/c with a minimum depth of 10 m.

Embankment construction

Taking into account the embankment design criteria, the construction of the embankment fill shall be carried out as follow :

The main body of the dam shall be composed of rockfill up to 1 m in diameter. The impermeable sandwich shall be placed on the upstream face of the dam, on a 1 m wide supporting layer, consisting of crushed stones (0 - 75 mm) and compacted parallel to the slope. A 3 m wide transition (0 - 300 mm, max. fine 5%) separates the main body of the dam from the supporting layer. Once the placement of the transition zone is completed, it shall be reprofiled by compacting along the slope using first fine gravel (15 - 30 mm) and then crushed stones (1 - 6 mm). In order to increase the surface cohesion between the transition and the supporting layers, the face of the transition shall be sprayed with a shallow impregnation of a cationic bituminous emulsion at a rate of 1.5 to 2 kg/m^2.

The impermeable sandwich shall be protected against ice, waves and wind by rockfill layers (rip-rap between elevation 149 m and 164,15 m) (fig. 3).

Sandwich placement

The installation of the sandwich for the main dam will involve an area of about 8000 m^2. The sandwich shall be rolled down from the crest of the berm to the bottom of either the toe of the dam or the next berm , in rolls 8 to 11 m wide. Temporary weighting with sand sacks or concrete blocks shall be provided against the uplifting of the membrane due to high winds. The adjacent sandwiches shall be joined together, as shown in fig. 3, by bonding their geomembranes. The type of joint to be executed, either by the application of a chemical adhesive or by soldering, shall depend on the selection of the geomembrane to be incorporated into the sandwich. In case of joints formed by the application of chemical adhesive, the geotextile shall be glued only on one side of the geomembrane over the entire width of the roll; while on the other side, the geomembrane shall remain unprotected over a 0.5 m wide strip, at each end of the roll. To avoid more than two sandwiches at the horizontal joints, rolls of varying width shall be used. The placement of the sandwich shall start from the first berm at el. 137 m to the toe of the dam and then proceed from the second to the first berm and so on.

The bonding of the impermeable sandwich with the upstream toe of the dam and the bedrock shall be achieved by excavating an anchor trench all along the perimeter of the upstream toe of the dam as shown in figs. 1 and 3 (7). The bottom of this trench shall be covered with a minimum of 15 cm of concrete. To avoid stress concentration in the permeable sandwich at its junction with the anchor trench, a concrete plinth with rounded edges shall be built. The anchor trench shall be filled with mastic compatible with the geomembrane - geotextile sandwich, up to the initial bedrock level (fig. 3).

At the top, the sandwich shall be anchored into the crest of the dyke as shown in fig. 3.

INSTRUMENTATION

Post construction behaviour of the dam shall be monitored by inclinometers located in the rockfill, settlement plates established at various levels in the rockfill, settlement gauges placed on the crest of the dams and reference points provided on the downstream slope of the dam.

CONSTRUCTION PROBLEMS

Although no major construction problems are foreseen a few minor problems will be encountered in the placement of the sandwich ;

1) Uplifting of the sandwich due to high winds;
2) Smoothing out the air pockets, once the sandwich has been rolled out;
3) Bonding the adjacent geomembranes either by glue or soldering;
4) Bonding of the sandwich to the foundation and the abutments.

CONCLUSION

The main dam at the Robertson Lake Project over 40 m high will be built using a synthetic membrane sandwich as an impermeable barrier, placed on the upstream slope of the dam. The sandwich shall be composed of a central geomembrane bonded between two geotextile layers. The geomembrane and the geotextiles shall be glued together in the factory and delivered to the site in rolls between 8 to 11 m wide.

Results of the research study show that it is possible to construct dam using synthetic membrane in northern climate.

ACKNOWLEDGEMENT

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Use of Thin PVC-Foils on Embankment and Cofferdams for Hydro-Electric Power Plants on the Austrian Danube

It is described the use of thin PVC-foils as sealing on cofferdams for run-of-river stations. If the natural subsoil offers the possibility, mainly diaphragm walls are used for sealing on the Danube.

Details are given about the sizes and the mechanical properties. The combination of diaphragm wall with a foil sealing is very economic especially for cofferdams because there are no problems for its construction and its demolition.

Since 1954 the Österreichische Donaukraftwerke AG has been constructing run-of-river stations on the Danube. Whilst the first two stages were constructed directly in the narrow valley of the river, all the others that have been constructed since then, are situated in the flatland, which is partly used for agriculture but mainly covered by meadows and woods (see Table 1 and Fig.1).

The method basically uses one large construction pit outside the riverbed, if possible at the inner side of a river meander. There the whole barrage consisting of powerhouse, weir and lock, can be constructed at one time (see Fig. 2).

As the whole site is located in an area where inundations are likely to occur, it is necessary to protect it during the construction period from the Danube's floods by enclosing cofferdams. Even if the medium flow only amounts to some 2000 m3/s, floods up to 14000 m3/s may occur. While parts of the barrage area are being concreted inside the construction pit, a new riverbed is excavated inside and outside the pit. Upon completion of the above mentioned parts the river is diverted into the new bed and at the same time the cofferdams are removed.

These cofferdams are erected only a few metres in height on the original ground surface, e.g. 1 to 2 m above the water level of the 100-year-flood. As filling material gravel is taken directly from the river. The sealing is made by means of PVC-membranes of only 0.5 mm in thickness. The cofferdam sections that run in parallel with the riverside, can later on be integrated into embankment dams parting from the barrage. In these dams foils of 0.6 mm in thickness are being used (see Table 2).

The foils are produced in 1.3 m wide stripes on calanders in the factory. Their tensile strength and resistance to perforation is rather high (see Table 3). Finally they are welded together to 15 m wide webs, folded to large piles and delivered to the building site. After having been placed on the dam crest, they are spread out by hand and rolled down the slope (see Fig. 3 and 4). The embankment itself consists of sand and gravel with a maximum grain size of up to 150 mm. On the slope (inclination 1V:2H) the foil edges are folded and stapled in order to form a dense seam.

Starting from the toe, the sealing is covered with a 1 m thick sand cover. Ballast material may be a further protection against lifting caused by water pressure from the dam inside.

Riprap protects the upstream slope from wave action. This is important because of the river's width in the dammed area in windy conditions. Besides, the Danube is navigated not only by single vessels up to a load of 1 300 t but also by units consisting of several vessels.

Between the slope stabilizing riprap and the underlying sand-gravel dam fill, geotextiles are inserted to protect from inner erosion of the fine aggregates (see Fig.8 and 9). These textiles are either non-woven materials of polypropylene fibres or fine grids made of nylon strings.

At present the costs of the foil itself amount to AS 30.-/m2, the mere application works including slope profiling amount to AS 45.-/m2.

The construction pit reaches far into the natural subsoil and is therefore exposed to groundwater penetration. Before 1970 steel sheet piles were used for sealing but since then only the so-called diaphragm walls were employed for that purpose. They reach lenghts

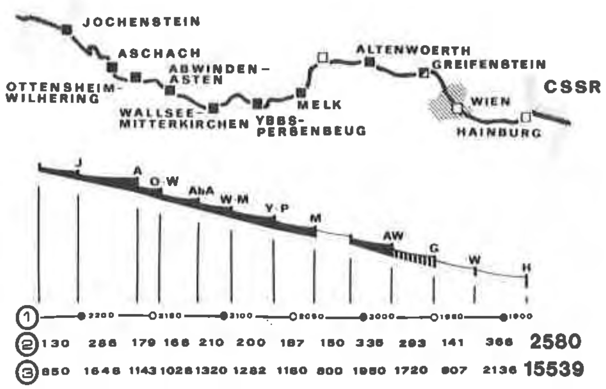


Fig.1 DoKW's power stations on Danube river

(1) River kilometre (from estuary)
(2) Installed capacity (MW)
(3) Mean annual energy (GWh)

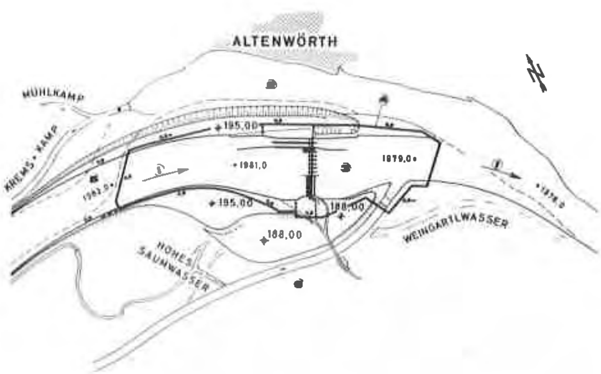


Fig.2: Danube River power station Altenwoerth
Drawing of building pit

(1) Danube River
(2) Dead stream branch (after obstruction remaining part of river)
(3) Building pit
(4) Building pit dam (coffer dam), consist of diaphragm and foil sealing (thickness 0,5 mm or 0,6 mm)
(5) Danubian lock dam
(6) Diversion cut of river Traisen

Fig.3: Placing of foils on the slope

Fig.4: Foils on the slope: along the ladder a construnction joint is being made; trimming in background

of 5 - 8 km corresponding to the large construction pit enclosures (see Table 1).

The diaphragm wall has to seal the 8 to 12 m thick zone of alluvial deposits that overlie the highly impervious tertiary subsoil. These diaphragm walls are built as follows: a thick steel pile with an injection tube is driven into the soil (see Fig. 5 and 6). Upon removal of the pile a special plastic mortar mixture is injected (see below) in order to form an impervious, though thin wall. Depending on the mixture, one can reach different plastic or stiff behaviour of the wall (see Table 4). The quality of this method could be examined by means of excavated samples (Fig. 7). The functioning of such walls can also be controlled by piezometer tubes on both sides of the wall. From permeability tests a value of 10^{-6} cm/s was derived.

In general the driving of the steel pile in the alluvial deposits does not cause any difficulties. In rare cases other sealing methods have to be used for short sections. In the case of coarse rock layers overlying the tertiary subsoil, sealing mortar has to be injected into the soil via tubes. In the presence of groundwater flow, special injection mixtures (e.g. addition of water glass) have to be used to reach a sudden setting in water before the injection material is washed away. At present diaphragm wall costs without addition of water glass amount to AS 330.-/m2 glass to AS 500.-/m2:

<u>Mixture of diaphragm wall mortar:</u>

cement	17%	of solid materials
sandstone dust	80%	- " -
bentonite	3%	- " -
water	60%	of solid material weight

1 m3 of injection mortar weighs approx. 1700 kg, a 50 l water glass addition corresponds to approx. 3%.

The combination of diaphragm wall in the subsoil and foil sealing in the dam has proved to be especially successful in the case of cofferdams for construction pit enclosures. This is due to following two reasons:

1. Simple and quick realization of the construction. The foils are absolutely impervious (see Table 5). Ageing problems of the PVC-material do not occur within the 2 or 3 years of lifetime, especially as the foil is not exposed to weather conditions or daylight.

2. Upon removal of the cofferdams the sealing made of foils and diaphragm wall does not represent any obstacle. It is simply dredged away together with the dam body consisting of compacted gravel. This is mainly true for the dam sections that run across the river and cut off the new riverbed. It was often very difficult and expensive to remove the steel sheet piles that were used before 1970. Sometimes they even had to be cut under water which did not only cause high costs but made their reuse impossible.

In order to transform the cofferdams in parallel with the river to permanent embank-

Fig.5: Manufacturing of a diaphragm wall for building pit-dam

Fig.6: Driving plant pulling the H-pile

Fig.7: Excavated diaphragm wall from test driving

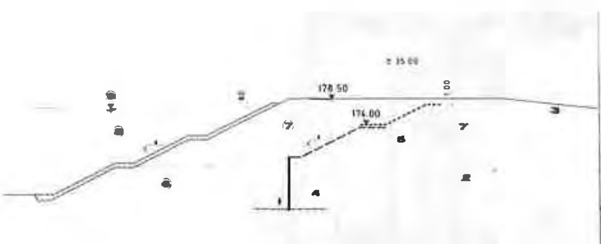


Fig.8: Danube River power station
Section of dam adjacent power station

(1) Flysch (bedrock)
(2) Gravel (Alluvial Sediments)
(3) Fine sand (recent sediments of Danube River)
(4) Diaphragm wall
(5) Sealing foils by PVC (thickness 0,6 mm)
(6) Initial ground level
(7) Compacted sand - gravel
(8) Riprap
(9) Storage level at middle water of Danube River

ment dams, as it was mentioned above, the foil sealing has to be installed in a special way (see Fig. 9): during the construction period it serves to protect the construction pit against the flood coming from outside. Upon completion of the construction, the cofferdams are considerably raised to transform them to embankment dams and the foil has to be extended to the definite dam crest (8). In these dams the water pressure is coming from the other direction, i.e. from the basin. In the first period the foil is installed opposite to the water pressure and is therefore in the right position in the permanent embankment dam. It only has to be weighted with sufficient ballast (2),(12).

Also the other upstream dams that extend for many kilometres along both sides of the Danube river, need a sealing. In the natural soil a diaphragm wall that reaches through the alluvial deposits to the impervious subsoil, takes care of the sealing (see Table 6). The dams themselves are either sealed by earthcore or asphaltic concrete sealings (see Fig. 10 - (1) and (9)).

If no more geomembranes will be installed here, then this is due to the fact that in the immediate vicinity of the sites, nature provides enough sealing material - "fine sand"- for an earth core sealing. Therefore this way of sealing is noticeably cheaper. These dams are impervious enough not to cause any problems. The permeability of earth core sealings made of adequate mixtures of gravel and fine sand (slightly clayey sand to be found on the ground surface along the riverbanks) shows values between 1 to 5 x 10^{-6} cm/s. The compactness of this mixture amounts to 98% of the simple Proctor density.

No information can be given on seepage in the area of the foil-sealed embankment dams as these dam sections can not be measured separately from other upstream dams with different sealing methods. There are no recent cases of seepage boil in the foil-sealed area that might indicate leakage in a foil.

Besides, the total amount of seepage steadily decreases in the course of the years (Fig. 11 and 12). It can be assumed that the riverbed's self-sealing in the reservoir greatly contributes to that by depositing fine and superfine floating particles. Such a self-sealing can only take place in a reservoir that right from the beginning has been well sealed in the technical sense as, due to higher flow velocities near major leakages or cracks, these fine particles would not settle there.

Self-sealing is, by nature, less important for cofferdams. Due to the short lifetime of these dams, this phenomenon can not occur in this case at all and therefore the geomembranes really have to be impervious. For the embankment dams, however, they have to exert their full function only during the first years, same as all the other sealing installations mentioned above. After each major Danube flood, the self-sealing increases through the intensified deposit of floating particles in the whole riverbed area. Thus it causes a seepage reduction to levels where the environment is not affected anymore.

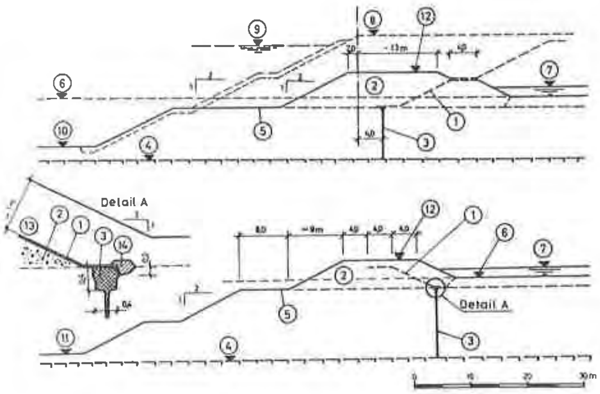


Fig.9: Power station Altenwoerth
Cross sections of cofferdams

(1) Synthetic foil (PVC)
(2) Fill (gravel compacted)
(3) Diaphragm wall (SW)
(4) Tertiary clay and sand strata
(5) Surface of alluvial gravel
(6) Original ground surface
(7) Water level at 100-year-flood
(8) Definitive typ of upstream fill
(9) Normal water level
(10) Bottom of upstream excavation
(11) Bottom of downstream excavation
(12) Top of cofferdam
(13) Protection layer (sand Ø: 0-5 mmm)
(14) Concrete layer (quality B 80)

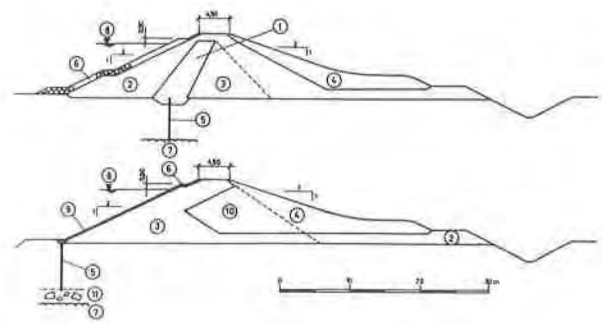


Fig 10: Power station Altenwoerth
Typical cross sections of embankments

(1) Core: gravel and sand mix
(2) Upstream shoulder (gravel)
(3) Gravel, compacted (dam body)
(4) Fine sand slope
(5) Diaphragm wall (SW)
(6) Riprap
(7) Tertiary clay and sand sediments
(8) Water level at mean flow
(9) Asphaltic concrete facing
(10) Fine sand compacted (dam body)
(11) Stratum of boulders

TABLE 1

	Surface of construction pit	Length of construction pit enclosure	Height of cofferdams
	10^6 m^2	10^3 m	m
Altenwoerth	1,35	6,09	10,0
Abwinden-A.	0,91	4,80	7,5
Melk	1,38	6,16	7,0
Greifenstein	1,57	7,75	8,5

TABLE 3

	Tensile strength in MPa	Extensibility in %	Cracking at - 20° C
New	15	200	-
after 3 years	11	140	-

TABLE 5

	Quantity of pumping water at medium flow l/s	Quantity of pumping water l/s	Dewatering costs in Mill. AS
Aw	104	17	21 (1972)
AbA	120	25	31 (1975)
Me	1000	162	39 (1978)
Gr	176	23	41 (1981)

TABLE 4

Mixture	Setting time	Compressive strength in MPa	Permeability in cm/s
without water-glass	3 days	0,5 - 0,8	5.10^{-7}
with 3 % water-glass	20 s	0,10	5.10^{-6}

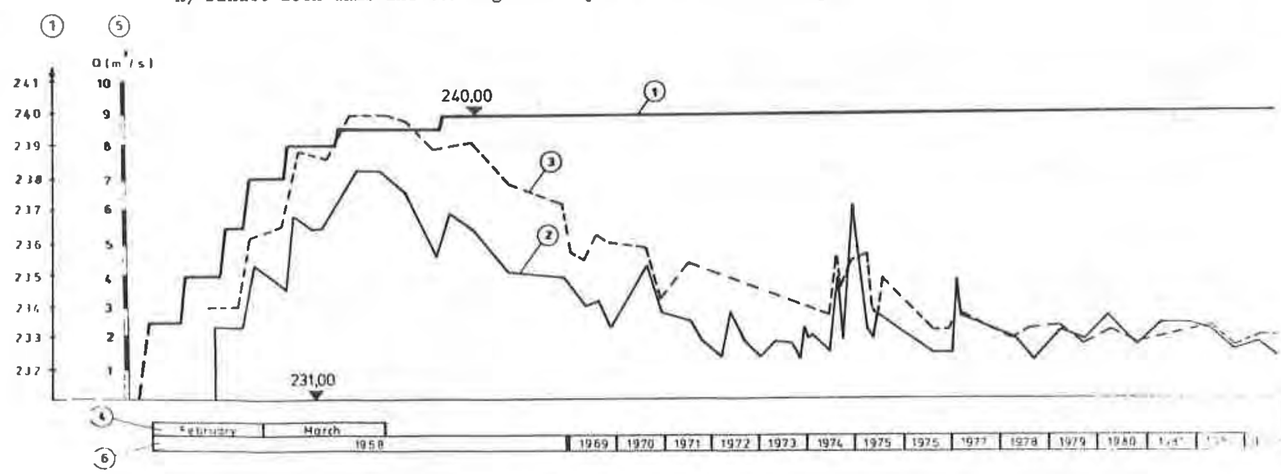
TABLE 2

	Total surface of foil sealing 10^3 m^2	in construction pit (thickness 0,5 mm) 10^3 m^2	in construction pit (thickness 0,6 mm) 10^3 m^2	embankment dams - incl. Danube lock dam.Thickn.:0,6 mm 10^3 m^2
Aw	175	50	44	81
AbA	85	50	20	15
Me	150	78	40	32
Gr	210	51	65	94

TABLE 6

	Total surface of diaphragm wall	In power plant area		In storage area
		construction pit	permanent embankment dam	
	$10^3\ m^2$	$10^3\ m^2$	$10^3\ m^2$	$10^3\ m^2$
Aw	348	54	$7^{x)}$	287
Aba	374	49	27	298
Me	303	60	51	192
Gr	440	74	26	340

x) Danube lock dams and sealing works by mean of steel sheet pilings



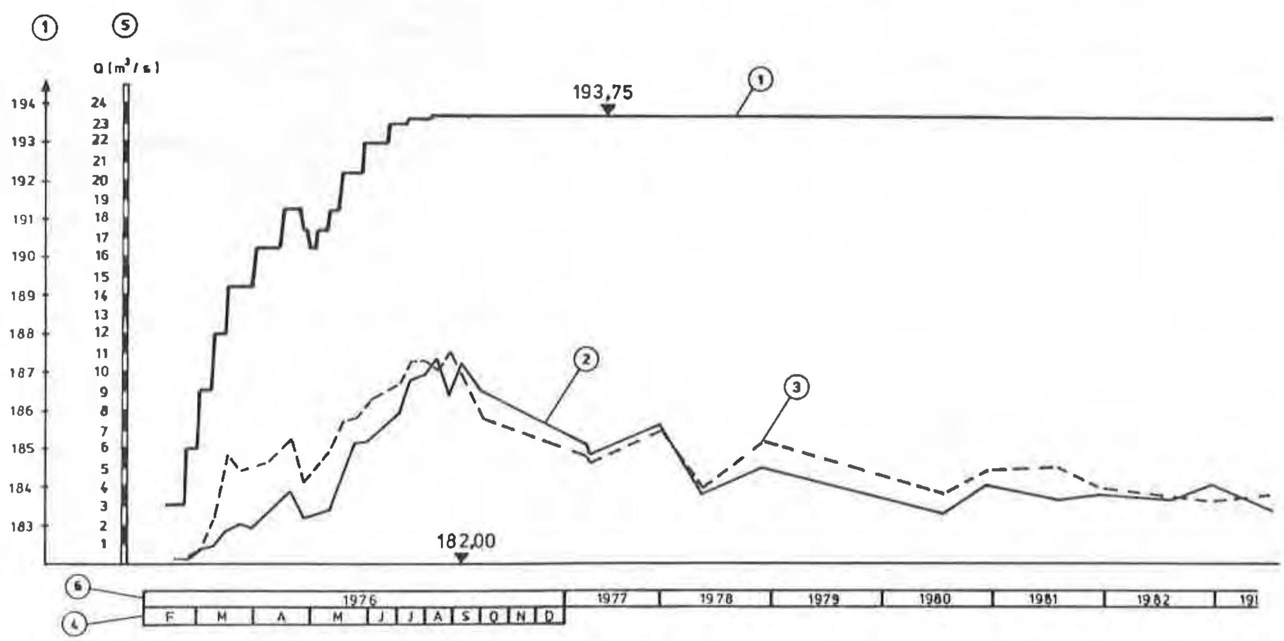


Fig.11 Power station Wallsee-Mitterkirchen:
Fig.12 Relationship between water level and rushing-out-water
(1) Water level
(2)(3) Graph of rushing-out-water from the right and the left bank
(4) Filling period (Feb. and March)
(5) Scale of flow
(6) Years of operation

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Automated Fabric Dam Aids Water Research Project

The fabric dam has adequately controlled the stream levels to meet its design purposes for a two year period. In the future other type structures may be developed to control stream water levels and pass flood waters and rafts of weeds as well as the Fabridam.

One of the features of stream water level control is the raising of the water table levels in the fields which have furnished water to increase crop yields by 36% over the two-year period. Secondly, an underground lake was formed in the surrounding area that supplied 352,840 M^3 of water which was pumped from Mitchell Creek in 1983.

The structure cost was $248,700 installed. A present value analysis based on increased yields showed the Fabridam would pay for itself on this location in 15 years and leave $22,200 ($1,480/Y) for maintenance and management. In addition, irrigation water was available for 6-center pivot systems, 3 traveling volume gun systems, and one subirrigation system.

Need for Study

Water conservation is a major concern in many irrigated areas of the world. In many agricultural water resource projects improvement in open channel flow is needed to control floods and to provide better drainage. During drier periods, however, these deepened channels may continue to lower the water table and crops may suffer from drought. Controlling stream water levels to store water in the soil is now being studies in North Carolina as a possible solution to these problems.

An agricultural research project sponsored jointly by United States Department of Agriculture, Agriculture Research Service, Soil Conservation Service and North Carolina State University was established in 1978 to study this problem. The project study area, approximately 800 hectares in Edgecombe and Pitt Counties, North Carolina, is a part of the Conetoe Creek Watershed project constructed under the Soil Conservation Service Public Law-566 Flood Prevention Program. Mitchell Creek Main serves the area as the outlet channel (2). To meet project objectives a structure was needed in this channel that would control the stream water level during the growing season but would automatically open during heavy rains to allow flood flow to pass without causing crop damage.

A heavy growth of aquatic weeds in the nutrient-rich water of Mitchell Creek further complicated the problem. Experience had shown that, during major floods, these weeds break loose from the channel bottom in mass and are carried rapidly downstream, possibly causing channel blockage and subsequent failure of the structure.

Figure 1. The Water Inflatable Fabric Dam on Mitchell Creek near Tarboro, North Carolina, U.S.A.

A fabric dam (Fabridam) (Fig. 1) was selected as the type structure that would allow the weed mass to pass and also allow desired automation. This type structure made of 2-ply neoprene coated nylon is attached to a concrete pad placed to fit the channel cross-section. By filling with water, the dam can be raised or lowered to any desired elevation. During floods the Fabridam automatically deflates flat on its pad allowing flood water and any weed accumulations to pass unrestricted. Holding water at a high level in the channel early in the growing season allows water to remain in the soil that would otherwise be lost (1). Runoff from rain can either be stored or released depending on crop needs (3). This type water management is limited to locations that have seasonally excess rainfall, are nearly level, and have soils on which crops respond well to drainage but in many years supplemental irrigation is beneficial. It is estimated, however, that as many as 5 million hectares in the southeastern part of the United States alone could benefit if water table levels were properly controlled (4).

The objective of this water research project is to learn the most practical way to manage the water table in certain soils to produce optimum yields, and determine the amount of supplemental water stored. The purpose of this paper is to acquaint others with this effort and to provide them with specific information on how an automatic structure can be installed and effectively used to help accomplish this objective.

Construction of the Fabric Dam

The dam is on an earth channel that was excavated as part of a Public Law-566 flood prevention and drainage project completed 17 years ago. The channel was approximately 3.3 meters deep with a 5.3 meter bottom width with 1:1 slide slopes. Soils in the channel bottom and sides are silty sands and poorly graded sands.

An earth by-pass channel (Fig. 2) was excavated around the right side of the work area to handle stream flow during construction. Coffer dams were constructed across the stream channel approximately 17 meters upstream and downstream from the work area.

Interlocking steel sheet piling was driven across the channel to a depth of about 4 metres below the channel bottom and extending approximately 5 meters into the edges of the bank on each side. The piling was to reduce seepage below the structure (Fig. 2).

A well field was installed around the proposed channel work and vault building to remove seepage water during construction (Fig. 2). Handling seepage water was a problem during the early stages of conconstruction without the well field.

Figure 2. Early construction phase of the Fabridam showing the by-pass channel in background, the interlocking sheet piling, the well point system for dewatering and the third pour conditions for the vault building.

The vault building contains the pumps, valves, and controls necessary to operate the inflatable dam. The building is located in the right side of the channel looking downstream. It is approximately 10 meters in height and 2.9 meters by 2.4 meters of the base. The building is a reinforced concrete structure poured in lifts. The bottom elevation of the structure is approximately 0.7 meters below the elevation of the channel bottom. The exterior of the building was treated for water tightness up to ground level. A sump pump was installed in the base of the building to pick up seepage. After the first two pours of concrete on this building, the work on the channel structure was started (Fig. 2).

The channel bottom and sides were excavated and shaped to accept the placement of the reinforced concrete slabs. The top of the steel sheet piling was cut off parallel and approximately 30 centimeters below the finished surface of the concrete slabs. The top of the steel piling is embedded in the upstream edge of the concrete slab. The surface of the slab was poured flush with channel bottom and sides. The surface of the slab were trowled to a hard smooth finish.

The upper part of the vault building was completed. The top 1.8 meters of the building is a tank for holding make up water for the dam. A 5-HP pump keeps the tank full. There are five pipes that extend from the lower portion of the vault building. Two pipes extend upstream from the building. One is a sensing line. The other is water intake line for the tank and bag.

Two pipes exiting into the inflated area extend under the concrete floor of the dam to the control vault. One pipe is a sensing line. The other pipe is used to inflate and deflate the dam. This pipe extends through the control vault downstream and exits near the channel bottom to deflate the dam.

Stainless steel anchor bolts are located on 30 centimeters center in the concrete slab bottom and sides. (Fig. 3). The bolts attach the bag material at its upstream and downstream edges. The bag is made of nylon reinforced synthetic rubbererized material, approximately 0.32 cm thick. An epoxy coating was applied to the concrete slab under the dam and to the area that will be in contact with the bag when deflated. This coating insures a watertight connection with the bag and reduces abrasion.

Figure 3. Side wall and bottom of the concrete pad on which the Fabridam rests showing bolts to hold the fabric and make the water tight seal to the pad.

A single-ply nylon, neoprene coated seal sheet was laid on the surface of the concrete slabs covering both the upstream anchoring bolts and the downstream anchoring bolts. This sheet ensures watertightness.

The bag material was cut to fit the anchor bolts. Anchoring plates cover the bag material and bolts. They are held in place with stainless steel nuts. This forms a watertight connection with the concrete surface (Fig. 4). A lot of effort was necessary in fitting, cleaning and attaching this material. The bag was inflated with air to check for leaks.

Figure 4. The excess fabric of the Fabridam was folded to form a smooth surface for the dam to rest on when deflated.

A reverse filter system was installed downstream from the concrete slab in the bottom and sides of the channel (Fig. 5). This is a graded filter that is composed of small riprap on the surface underlain with layers of gravel and sand for a combined thickness of 0.9 meters. The riprap layer was grouted leaving many openings for water movement. The top surfaces of the filter are flush with the surfaces of the concrete slabs. This filter is to reduce hydrostatic pressure under the concrete slab and to prevent scouring below.

With the bag satisfactorily installed and controls in the vault building in place, the two coffer dams were removed. The by-pass channel was filled in and the area graded so that the earth adjacent to the vault building was approximately 0.5 meters higher than the surrounding area. The entire disturbed work area was vegetated with a perennial grass.

Figure 5. Reverse filter downstream of the Fabridam.

Operation and Performance of Fabridam

The 2.7 meter high Fabridam structure (Fig. 1) was installed across Mitchell Creek about midway in the 3.5-km study area and put into operation on April 2, 1982. The water inflatable fabric dam is about 6 m wide at the bottom of the creek bed and 13 m wide at bank height. The dam is used to automatically control the water level in the creek. The Fabridam collapses during flooding which allows the channel to carry full design flow. It can automatically control the water level in the channel to any depth up to 2.45 m. For example, if the control level is set at 11.45 m above Mean Sea Level and a flood raises the upstream level to 11.60 (0.15 m rise), the Fabridam begins to deflate, but will remain controlled between 11.45 and 11.60 m. If the flood level continues to rise to 11.62 m (.17 m rise), another valve opens and the Fabridam deflates faster, but automatic controls keep it between 11.45 and 11.60 m. If the flooding continues and the upstream water level reaches 11.65 m (0.2 m rise), a 20.3-cm syphon will deflate the Fabridam at a rate of about 0.06 m/min until there is no restriction to flow in the channel. As soon as the flood passes and the syphon breaks, the Fabridam inflates to the original setting of 11.45 m.

Water Level Controls

Controlling the stream water levels affected the water table levels in the soil of adjacent lands for a distance of 900 m from the creek. Near the creek the water table was about 1.5 m closer to the ground surface after the Fabridam was installed and from about 400 m to 900 m from the creek the water table was about 0.5 m closer to the surface. The 3-dimensional water surface from 305 m below the Fabridam to 1135 m above the structure and from Mitchell Creek to 621 m to the right looking upstream at the Fabridam is presented in Figure 6. Below the Fabridam, the water table surface near Mitchell Creek was at about 3 meters below the soil surface and increased to about 1.5 meters from the surface at 621 m from the stream. Gradients in the water table surface in the channel direction near the Fabridam indicated soil water flow around the dam to the low stream elevation below the dam. The water table surface was essentially controlled in a flat condition above the Fabridam over the 65-ha area and 1135 meters upstream from the Fabridam (Figure 6). Most of the variations in water table depth was due to soil surface variations. The water surface in the soil adjacent to Mitchell Creek was successfully regulated by stream water level control in the creek. These data show that future design of water resource projects should consider both drainage and water table control.

Irrigation Water Available

Controlling the stream water level provided an underground lake from which irrigation of crops was made possible. For example, before the Fabridam was installed to control the stream water level, only one center pivot system and one traveling volume gun were able to pump enough water from Mitchell Creek and then they could pump less than 12 hours each day. In 1983, with the stream water level controlled, water was pumped from Mitchell Creek and the underground lake (Fig. 6) to supply 6-center pivot systems, 3-traveling volume guns and one subirrigation system. A total of 352,840 m^3 (35.3 hectare-m) was pumped from above the Fabridam in 1983. Figure 7 shows the accumulated water pumped for irrigation in relation to the stream water level and the water table surface in relation to distance from the creek. The top of the Fabridam was set to control the stream water level at 11.45 m above Mean Sea Level during the corn growing season. Anytime the stream water level

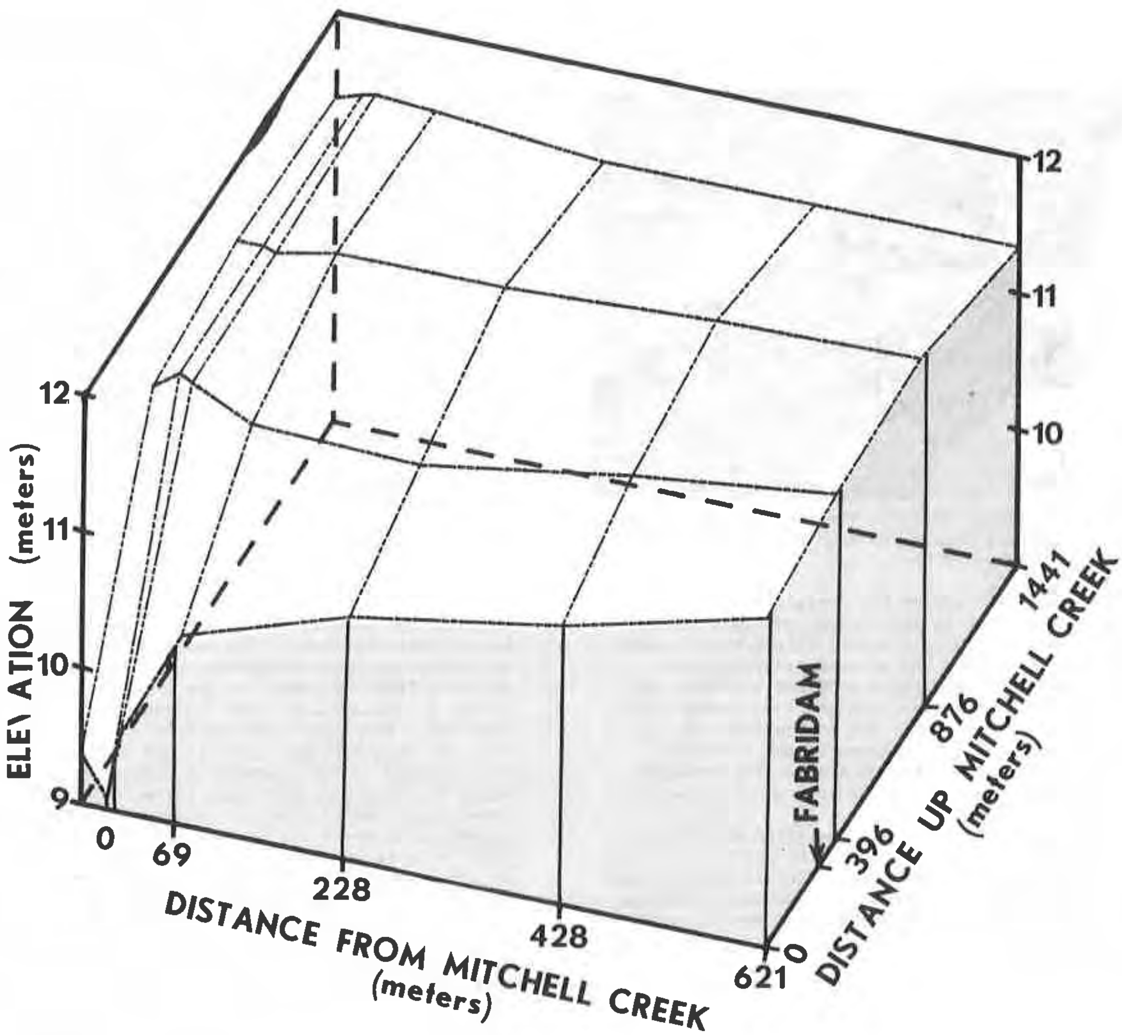

Figure 6. Three dimensional relationship of water table surface from 305 meters below the Fabridam to 1135 meters above the structure and to 621 meters away from Mitchell Creek on July 10, 1982.

receded below 11.45 m the stream was not flowing over the Fabridam and downstream flow was supplied by seepage through the soil. The pumping of 31.2 ha-m of irrigation water lowered the stream water level 0.82 m. This lowered the ground water level 0.69 m at 9 m from the creek, 0.58 m at 85 m from the creek and 0.33 at 610 m from the creek. As the water was pumped from the creek lowering the stream water level, water flowed into the creek from the underground lake allowing 312,000 m^3 of water to be pumped for irrigation purposes.

Corn Yields

Corn yields were sampled by hand from 3 x 2 m plots over the area above the Fabridam (control) and below the Fabridam (no control). Some fields were irrigated and some were not. Stream water level control increased corn yields by 36% over the no control area in nonirrigated fields and by 26% in irrigated field for the 1982 and 1983 cropping seasons (Table 1).

Table 1. Corn yields as affected by controlling stream water levels with a Fabridam.

Stream Water Level	1982		1983		Mean	
	Surface Watering					
	Nonirr.	Irr.	Nonirr.	Irr.	Nonirr.	Irr.
	t/ha					
Control	8.32b[1/]	10.33a	5.42c	9.96a	6.87	10.15
No Control	6.93c	8.29b	3.19d	7.77b	5.06	8.03

1/ Yields followed by the same letter within the same year are not significantly different at the 5% level DMRT.

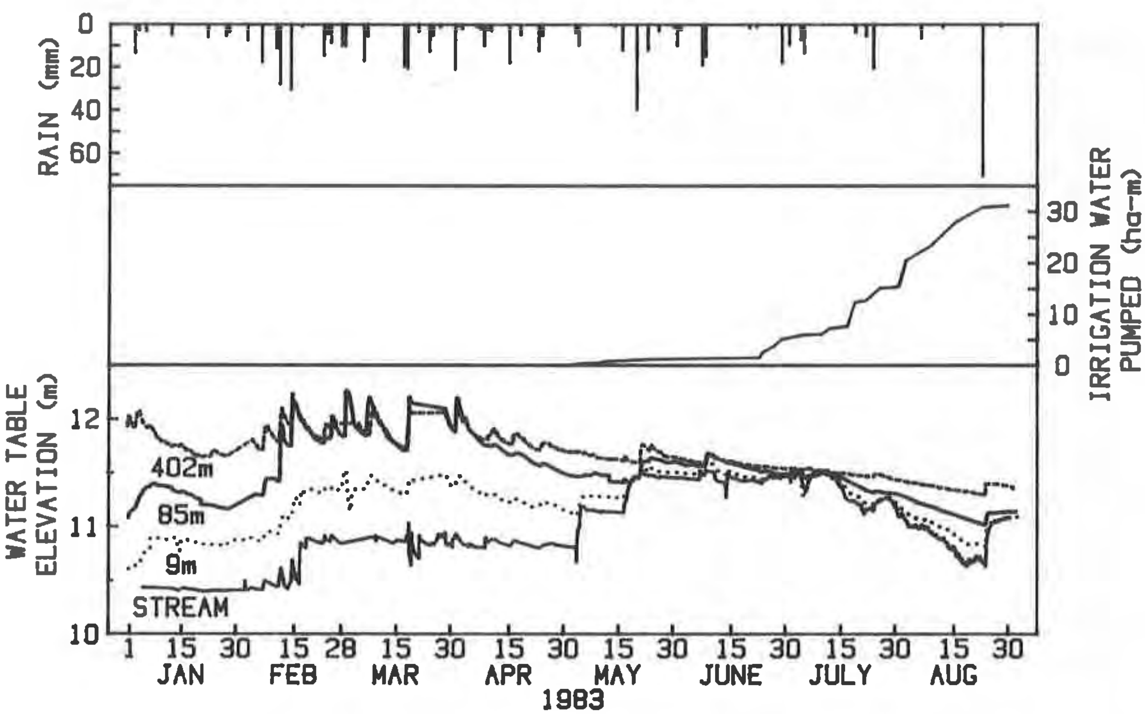

Figure 7. Relationship of rainfall, irrigation water pumped and water table elevations at various distances from Mitchell Creek in 1983. The control level of the Fabridam was set at 10.4 m until February 16, 10.8 m until May 4, 11.1 m until May 17, and 11.5 m from May 17 through the rest of the cropping season.

Table 2. Present value of a corn crop on the Mitchell Creek area with and without stream water level control at \$98.40/ton (\$2.50/bu) projected for 15 years at 10 percent interest (PWF = 7.606). The estimated life of the Fabridam is 50 years.

Stream Water Level Mgmt.	Average Yield[1/]	Area	Present crop value[3/]	Present Mgmt. cost	Returns for maint. & mgmt.
	(t/ha)	(ha)[2/]	----------dollars---------		
Control	6.87	200	1,028,300	24 ,700	779,600
No Control	5.06	200	757,400	-	757,400
Increase due to control	1.81	-	270,900	248,700	22,200

1/ Average yield for 1982 and 1983.
2/ Water table affected at least 2200 m upstream and 450 m on either side of creek.
3/ Present crop value = 757,400 = 5.06 t/ha x 200 ha x \$98.40/t x (PWF = 7.606) - rounded to \$100

The Fabridam cost \$248,700 installed. This high cost was in part due to it being a prototype in the area. However, based on the average corn yield data for 1982 and 1983, structures as expensive as the Fabridam will pay for themselves in these sandy soils over a 15 year period and leave a balance of \$22,200 (\$1,480/year) for maintenance and management (Table 2). This does not include the fact that 352,840 m^3 additional water was stored and pumped for irrigation in 1983.

Acknowledgement

The Fabridam is a patented structure by N. M. Imbertson & Assoc., Inc., Burbank, CA, designed specifically for this project from ARS specifications. Mention of this trademark does not constitute a guarantee or warranty of the product by the U.S. Dept. of Agric. and does not imply its approval to the exclusion of other products or structures that may also be suitable.

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Flexible Membrane Lining for Emergency Spillways

Growing concern over inadequate emergency spillway capacity for existing embankment dams, a low-cost alternative may be the use of flexible membrane-lined emergency spillway. The membrane industry offers a wide range of materials for consideration. To date, only limited work has been done by others. The Bureau of Reclamation has begun an investigation on the use of membrane emergency spillways for low-head structures. With increased knowledge, consideration can be given to higher head structures. A field study is expected to be completed in 1985. It will involve installation of an 80-meter-long, hypalon lining on a spillway at a dam located near Grand Junction, Colorado. A study program will follow the installation. A 2-year study of the material to be used show minimal change.

PROBLEM

On September 30, 1981, the Corps of Engineers discovered, in their inspection of non-Federal dams program, that nearly 3,000 dams were unsafe, requiring some remedial action. Of these, 81 percent were deficient because their spillways were too small to pass the estimated maximum floods. This reflects the difference between present-day design flood criteria, contained in the "Recommended Guidelines for the Safety Inspection of Dams," and the criteria in vogue at the time the dams were constructed (1, 2, 3).

Embankment dams are particularly sensitive to failure caused by overtopping, both during construction and while in service. However, inadequate spillway capacity is not the only cause of overtopping failure. There have also been many cases where dams were overtopped because of gate failure (4).

These potential hazards can be avoided by adding an emergency spillway with the required discharge capacity. However, in many situations the cost for a conventional concrete-lined spillway, or even a rockfaced compacted-earth spillway, would be prohibitive.

PROPOSED SOLUTION

An attractive approach is to provide a membrane-lined emergency spillway. The flexible membrane would be covered with soil until it is needed. At the beginning of emergency spillway operation, the soil would be washed away, and the membrane lining would carry the flow, protecting the embankment from erosion (5).

Significant advances have been made in recent years in the manufacture of flexible membrane materials that are suitable for a wide range of water resources engineering work. Therefore, many excellent materials are now available. Much work has been done to identify the important properties of these materials. Laboratory testing, field studies, and observations of these materials in place have provided guidance for the selection of durable materials.

A search of the literature did not turn up reports on the use of flexible membranes for emergency or normal spillways at embankment dams. However, one report suggested that some encouraging work was being done in France, and another study, from the U.S.S.R., concluded that flexible, or soft, spillways should be studied further (6) (7).

INVESTIGATION

Because the use of a flexible membrane in a low-cost auxiliary spillway appeared to have merit, a study of modest scope was begun by the USBR. The investigation started with an evaluation of the feasibility of various applications for low-head structures. Locations where the consequences of failure would not be serious were given primary consideration. Some potential applications include:

- The many low-head earth dams of the Bureau of Indian Affairs, National Park Service, and other Department of the Interior agencies for which the USBR has some responsibility
- The low-head structures among the 50 to 60 percent of USBR dams, inspected to date that have inadequate spillways
- New low-head earth dams
- Low-head dikes on large reservoirs
- Saddles suitable for emergency overflow where erosion could be a problem
- Side channels for low-head embankments
- Canal wasteways

- Diversion structures
- Drop structures
- Improvements to existing emergency/auxiliary spillways

The primary objective of the research is to develop design procedures, materials specifications, construction procedures, and cost data to assist in the selection, design, and construction of flexible membrane or geotextile emergency/auxiliary spillways for low-head structures.

The study is to include investigation of the following:

- The durability of materials with respect to aging.
- The durability of the material during passage of floodflows
- Structural design
- Hydraulic design
- The effects of permeability/impermeability with respect to uplift
- Cost data including cost comparisons
- Identification of limits of usefulness: heights, maximum water velocity, effects of drag on membranes, and the effects of membrane wrinkles on flow
- The consequences of failure
- Soil and vegetation (root) effects
- The ability to pass debris during floodflow
- Means of handling vehicular traffic
- Materials requirements

The study is currently focusing on the general design and related material requirements for low-head application. From this, some specific design and construction concepts will be developed. At the same time, limited field studies will help identify the advantages and specific areas needing more study. One field study has been started at Cottonwood Dam No. 5, located in western Colorado.

FIELD STUDY

The construction of a flexible membrane emergency spillway at Cottonwood Dam No. 5 is an important part of the present study. This small earth dam, which is being rehabilitated, offered an excellent opportunity for the field study. Some of the questions that should be answered from this study are:

1. When membranes are used on slopes, what effect does drag caused by the water velocity component have on the membrane? How does drag affect the membrane's tensile properties at its support, and does drag cause the uplift of the sheet from its foundation?

2. What are the effects of abrasive sands and materials on the membrane?

3. What are efficient methods of anchoring the membrane along the sides and in the transverse direction?

4. What would be the minimum depth of cover material required to protect the membrane from the elements and the best type of cover material to use?

5. What effect does high water velocity, 4 to 5 m/s, have on the membrane when the sheet is wrinkled after placement?

6. Are special foundation treatments needed before the membrane is placed?

7. What are the effects of aging on the durability and permeability of membranes?

8. How is the membrane affected by different soil types and vegetation (root) growth?

Cottonwood Dam No. 5 is one of 17 small private reservoirs of the Collbran Project that were constructed on Grand Mesa, near Grand Junction, Colorado. These reservoirs, which are filled during the spring runoff, were to regulate the runoff from small streams. This stored water is released on demand for hydroelectric power and irrigation.

A Bureau of Reclamation SEED (Safety Examination of Existing Dams) report recommended that Cottonwood Dam No. 5 be breached and reconstructed. This recommendation provided the opportunity for the implementation of the flexible membrane emergency spillway study. The earth dam is 137 m wide and 5.8 m high at elevation 3050 m.

DESIGN CONSIDERATIONS

A summary of the spillway design considerations is listed below.

1. The spillway is alined to pass through the more plastic materials on the right abutment to provide additional erosion protection if needed.

2. Two grade sills are to be provided: one at the upstream end of the membrane liner to provide flow control and prevent piping, the other at the downstream end to prevent head-cutting, back into the spillway.

At the grade sills the membrane is to be attached to concrete with redwood furring strips and nails to distribute the load evenly across the sheets and to prevent separation from the grade sill.

3. The edges of the liner along the sides and transverse joints are to be placed in trenches, which are subsequently backfilled with compacted soil.

4. No longitudinal joints in the membrane are allowed. This prevents stress buildup in one sheet from being transferred to another.

5. Each sheet is to overlap its adjacent downstream sheet by approximately 1.5 m to provide adequate protection of the compacted back-

fill and anchor the downstream sheet. A minimum overlap of 0.61 m is to be provided on the horizontal curve.

6. A protective cover of 150 mm of noncohesive material is to be provided over the flexible membrane to protect it from foot, animal, and vehicle traffic. Materials will be dredged from Cottonwood Lake No. 1, which are noncohesive and are expected to wash away during spillway operation.

7. The alinement was chosen so that there are no discharges along the toe of the dam.

8. Inflow design flood is the 100-year flood. This results in a design discharge of 1.13 m^3/s.

9. Flow passes through the critical depth at the upstream grade sill; therefore, the flow is supercritical over all areas protected by the flexible membrane liner.

10. The channel bottom width is 3.66 m, with 2:1 side slopes and a depth of 0.91 m (to provide freeboard). This is based on buildup action in the horizontal curve.

11. The assumed Manning's number is "n" = 0.025 for the protective cover in place, and "n" = 0.015 for the flexible membrane.

12. Maximum channel velocity is 4.42 m/s.

13. Energy dissipation is to be provided by a natural hydraulic jump, which should form over the downstream riprap protection.

14. Riprap is sized to resist movement caused by velocities associated with the design discharge

The desired physical properties of flexible membranes for applications in the emergency spillways include the following:

- High tensile strength and flexibility
- High puncture and abrasion resistance
- Good impact tear resistance
- Good weatherability
- Immunity to bacterial and fungus attack
- Ease of welding seams in the field

At the present time there are two types of lining materials that may be suitable for this application. These are the fabric-reinforced materials, such as hypalon and CPE (chlorinated polyethylene); and HPDE (high density polyethylene) materials. A few of the more important physical properties of these lining materials and their approximate costs are listed below.

	Reinforced materials	HDPE materials (includes HDPE/alloy materials)
Available thickness, (mm)	0.76, 0.91, 1.1, 1.52	0.51, 0.76, 0.91, 1.52, 2.54
Tensile strength, (kN)	0.9 to 1.3	0.3 to 0.7
Tear strength, (kN)	0.15 to 0.45	0.45 to 2.25
Approximate cost, ($\$/m^2$)	5 to 8	5 to 10

Since different test methods and sample sizes are used to determine tensile and tear strengths, and puncture resistances, the values above are not directly comparable between the two types of lining materials.

A question has been raised about the cavitation damage to the membrane during high velocity flow. Cavitation damage occurs downstream from an abrupt change in surface condition. A seam or wrinkle might constitute such a condition. The most severe condition is a right angle offset into the direction of flow. The criteria for incipient cavitation to begin is for the cavitation number

$$\sigma = \frac{P_o - P_v}{\rho_w v^2/2}$$

where P_o = absolute pressure at the surface
P_v = vapor pressure of water
ρ_w = mass density of water
v = water velocity

For a right angle offset into the direction of flow $\sigma = 1.9$. A safe criteria for no cavitation to occur, then, is for $\sigma > 2$. This results in requiring a requirement for a velocity greater than 8.2 m/s for cavitation to occur. The design velocity of 4.42 m/s is much lower than this value. Hence cavitation damage is not expected (8) (9).

For a test at Cottonwood No. 5, field seaming would be impractical because of the remote location of the dam. Therefore, only reinforced sheets will be used at Cottonwood Dam No. 5, since they can be prefabricated for transport to the jobsite in lieu of nonreinforced maerials which have to be field seamed.

The material selected for the field study is 0.9 mm reinforced hypalon sheet, fabricated to 11.6 by 12.2 m and 11.6 by 7.01 m. This material met the specifications shown in table 1.

During all periods of shipment and storage, the membrane lining is being protected from direct sunlight, temperatures greater than 140 °F, mud, dirt, dust, and debris. To the extent possible, the membrane is maintained wrapped in a heavy-duty protective covering, or covered at the jobsite until it is installed. Before installation, the membrane lining will be inspected for defects, such as rips, tears,and holes; and it will be repaired accordingly.

Table 1. - Material properties (10)

Property	Test method	Value
Gage (nominal)		36 (mils) 0.91 (mm)
Piles reinforcing		1
Thickness (mm, minimum)	ASTM D 751	
1. Overall		0.86
2. Over scrim	Optical method	0.28
Breaking strength-fabric (kN, minimum)	ASTM D 751 method A	0.89
Tear strength (kN, minimum)	ASTM D 751 (modified)	
1. Initial		10.5
2. After aging		4.4
Low temperature (°C)	ASTM D 2136 3.2-mm mandrel, 4 h, pass	-40
Dimensional stability (each direction percent change maximum)	ASTM D 1204 100 °C, 1 h	2
Volatile loss (percent loss maximum)	ASTM D 1203 method A, 0.36-mm sheet	0.5
Resistance to soil burial (percent change maximum in original values)	ASTM D 3083 0.36-mm sheet (modified)	
a. Unsupported sheet		
1. Breaking strength		5
2. Elongation at break		20
3. Modulus at 100% elongation		20
b Membrane fabric breaking strength	ASTM D 751 method A, procedure 1	0.11
Hydrostatic resistance (MPa, minimum)	ASTM D 751 method A, procedure 1	1.72
Ply adhesion (each direction kN/mm width, minimum)	ASTM D 413 machine method type A	1.2
Factory Seam Requirements		
Bonded seam strength - shear (factory seam, breaking factor, kN width)	ASTM D 751 (modified)	0.712
Peel adhesion (kN/inch, minimum)	ASTM D 413 (modified)	Ply separation in plane of scrim or 2.6 kN/m
Resistance to soil burial (percent change maximum in original value)	ASTM D 3083 (modified)	
Peel adhesion		20
Bonded seam strength		-25

PROPOSED STUDY PROGRAM AT COTTONWOOD DAM NO. 5

To obtain the maximum benefit from the field study, a study program will be followed with both a short-term and a long-term phase. The key items of the program are:

Short-Term Phase

1. During installation

   a. Obtain information for the selection of membrane cover materials used.

   b. Note any possible improvements in the design concepts.

   c. Determine changes in construction guidelines for future construction.

   d. Establish two or three survey control points for elevation and alinement for future study.

2. Study the flow with the cover material in place. The emergency spillway will be operated using flashboards in the gate chamber of outlet works.

   a. Observe the movement of the cover material with flow.

   b. Determine the need for a fuse plug.

   c. Observe the density of the cover material before and after flow.

   d. Establish the depth of cover needed for flow requirements.

3. Study the flow without the cover protection

   a. Observe membrane behavior, movement, uplift, and stress.

Long-Term Phase

1. Long-term weathering and compaction of cover material - how does this effect the flow?

2. Membrane samples will be removed periodically from test sites near the spillway to study aging.

3. Damage to membrane lining from:

   a. Root growth.

   b. Animal hoof damage.

4. Study of the removal of the cover material after a period of weathering and the effect on the membrane after a curing period.

5. An annual field inspection of damage to the membrane.

6. The effects of water velocity on the membrane during any normal operations.

7. Anchoring methods for membrane protection.

8. Crest sill construction and end sill construction.

9. The best methods of patching the membrane, if necessary.

As of October 1983, the construction of the dam has progressed to the point where little additional work is required before construction of the spillway could begin. Because the dam is located at an elevation of 3050 m and in a remote location, the construction site is sensitive to the weather. It had been hoped that the spillway would be installed and first tested in 1982; however, heavy winter snows and summer rains have resulted in a 2-year postponement of the spillway construction.

During the 2-year postponement water emersion and outdoor exposure tests have been conducted on samples of the membrane to be used in the field study. The test conditions and results are summarized in the following table. since the data did not show any significant change with time, the effects were apparently the initial adjustment to the test environment; the average results are given.

Table 2. - Two-year materials evaluation

Test interval in weeks		Water immersion 4, 13, 26, 52, 78, 105	Outdoor exposure 26, 52, 78, 104
	Original value	Average percent change from original value	
Tests on membrane			
Thickness	0.950 mm	1.5 ± 2	-6.0 ± 0.9
Breaking strength	1.13 kN	-12 ± 17	-
Tear strength	0.721 kN	-20 ± 9	-
Hydrostatic resistance	2.89 MPa	-2.3 ± 3	-0.65 ± 0.6
Ply adhesion	1.26 kN/m	-6.7 ± 6	-16 ± 6
Water absorption by wt.	-	1.5 ± 0.2	-
Tests on factory seams			
Bonded seam strength - shear	0.62 kN	10 ± 26	-
Peel adhesion	4.03 Kn/m	-7.3 ± 17	9.3 ± 9

Although some properties had some changes, these changes are considered minimal. This is because there is no indication of progressive deterioration with time and the changes are consistent with those that occur with the care of this material that takes place in the first few weeks of exposure.

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The Use of Geomembranes in Foundations on Collapsible Soils

A methodology is presented for solving liquid control problems on collapsible soils by using a geomembrane-geotextile system (GGS) placed at the base of a large mineral embankment (heap leaching). A previous selection of the GGS materials was performed by means of uniaxial tensile and cone puncturing tests. The selected GGS were tested on a large scale oedometer to reproduce working vertical stresses acting upon the system, soil stratigraphy, creep effects and construction irregularities during foundation grading. It is conclude that tensile strength and cone puncturing index give a crude idea of the GGS behavior against soil puncture effects. For specific working conditions like those existing at the heap base, large scale tests should be used; otherwise soil particle effects on the GGS could not be properly accounted for.

INTRODUCTION

To achieve an impervious condition at the base of important earthworks the use of a geomembrane-geotextile system (GGS) turns into an attractive solution, especially when natural soils for impervious blankets are not available at the site. When collapsible soils are detected below the foundation level, a perfect impervious condition is required to avoid any liquid percolation into the supporting soil strata.

One of the best solutions to obtain this condition is by means of a GGS. However, the use of GGS is associated with a very complex interaction problem of interfases between an artifitial material and the surrounding geologic media. GGS inside a soil mass can be highly stressed at particle contacts or inflated due to high pore pressures, making geomembranes to be punctured (5). That is why size distribution, maximum size and shape of soil particles are very important factors on the GGS behavior.

Under these circumstances the GGS has to withstand the action of both hydrostatic pressures and local high level stresses between particles, which means a very hard working condition. Because large settlements occur when collapsible soils are percolated by liquids, geomembrane puncture must be avoided. Otherwise local failures can be progressively extended ending up into a collapse of the whole structure.

In order to reproduce GGS working conditions for a particular problem, large scale tests can be implemented. Even this is the best way to approach the field problem, these type of tests are very expensive mainly when handling with different geomembrane - geotextile combinations. To minimize the number of large scale tests, simple uniaxial tensile tests (ASTM D 638) and cone puncturing tests (3) can be used at first. Once a limited amount of combinations are selected by means of these simple tests, then a program for large tests can be established.

THEORETICAL MODELS

At the present time theoretical approaches are being developed to solve GGS - soil interaction problems (1, 2, 4). Theoretical methods assume that GGS can be modelled as an interface with well defined elastic properties and thickness (macroscale analysis). In order to analyze the two - dimensional behaviour of the system both stresses and strains induced on the GGS can be computed by means of finite element techniques. However some limitations arise from this approach, because puncture effects at soil particle - geomembrane contacts are not considered into the model (microscale analysis). More sophisticated approaches have been developed for a better modelling of the interface behavior. This has been done by introducing frictional sliding as well as unbonding and rebonding of the interface contacts. In spite of these additional improvements, a lot of work has to be done to end up into a microscale analysis. At the present time it seems that the only way for predicting GGS behavior is through empirical methods.

EMPIRICAL MODELS

Microscale effects can be taken into account by using methods based on inflated geomembrane tests, which give design parameters for predicting the GGS behavior. (5, 6). Alternative approaches can be implemented by means of large scale tests in order to reproduce those conditions acting in the field. All of these empirical methods are focused to see if the GGS is enough strong to withstand the design working stresses induced by a certain type of soil, without geomembrane puncturing.

Figure 1 shows a large scale oedometer which was used to simulate vertical loads on a GGS placed at the base of a heap leaching. For this specific case, it was decided to put a rock drainage layer over the GGS with and without a compacted gravel mattress acting as a protective layer. Heaps were located over a collapsible soil where liquid percolation has to be avoided. Working stresses at the heap base were of the order of 0.98 MPa, but the test program was conducted using stresses of 0.98 to 1.47 MPa in order to have a test load with a factor of safety ranging from 1,0 to 1,5.

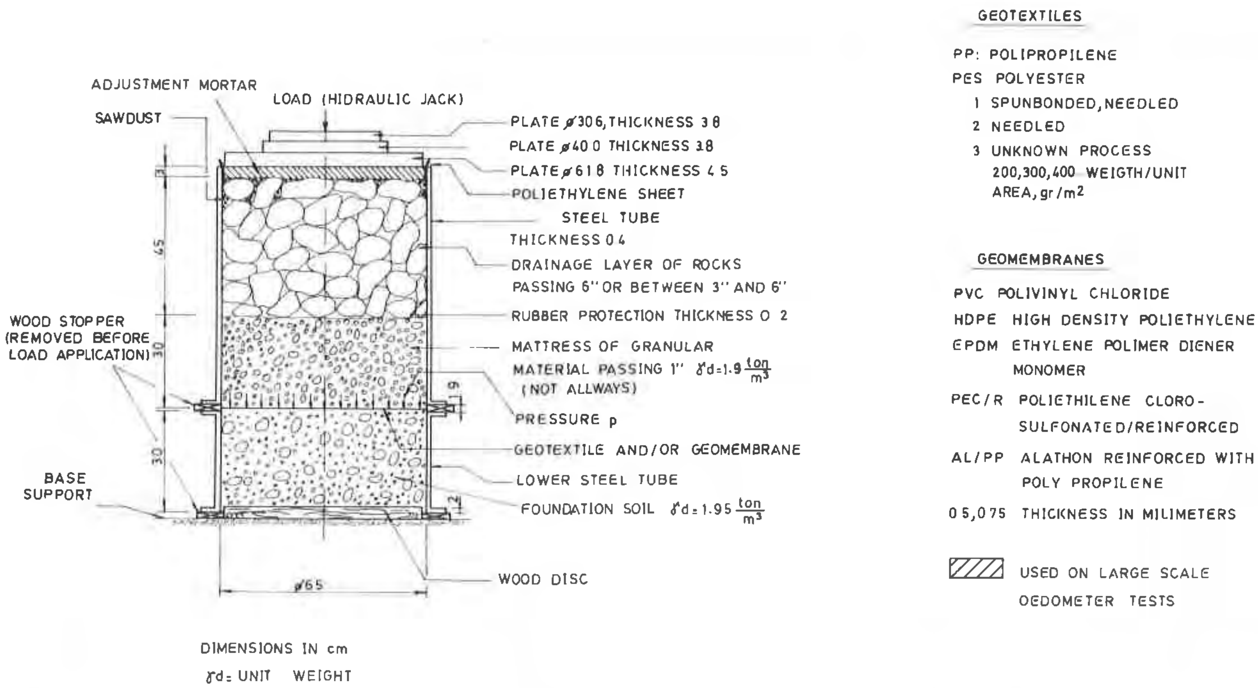

FIG 1 ROCK CONFINED PUNCTURING EQUIPMENT
(LARGE SCALE OEDOMETER TEST)

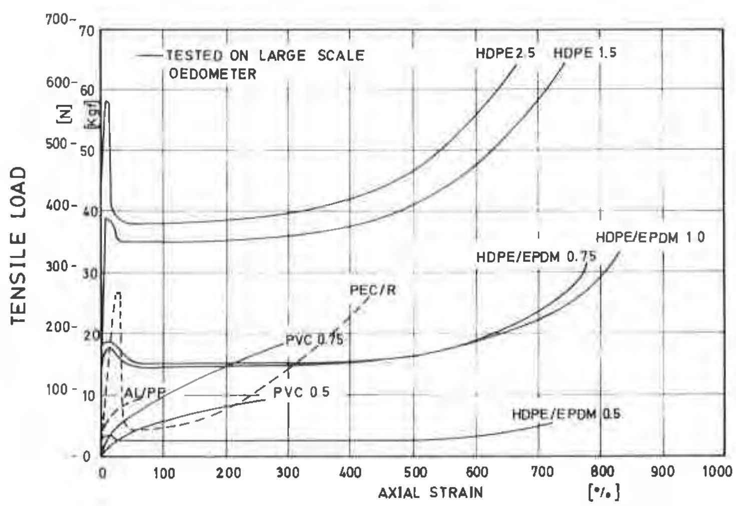

FIG 2 RESULTS FROM UNIAXIAL TENSILE TESTS (ASTM D 638)
ON GEOMEMBRANES

A previous material selection was performed by means of uniaxial tensile tests on different geomembranes (Fig. 2). As a complementary support for this selection, cone puncturing tests were carried out on geomembranes with and without geotextile protection (Fig. 3). When selecting the GGS to be tested on the large scale oedometer, economic considerations were taken into account to have a reliable system from a technical and economical point ot view.

Table I shows a summary with the testing conditions which include cases without granular mattress. Mattress was eliminated in some cases to explore the possibility of using a GGS directly below the rock drainage layer in order to reduce the cost of the whole system. To take into account construction defects during level and compaction operations on the foundation surface, it was decided to test all the GGS by putting isolated gravel particles on the foundation soil surface

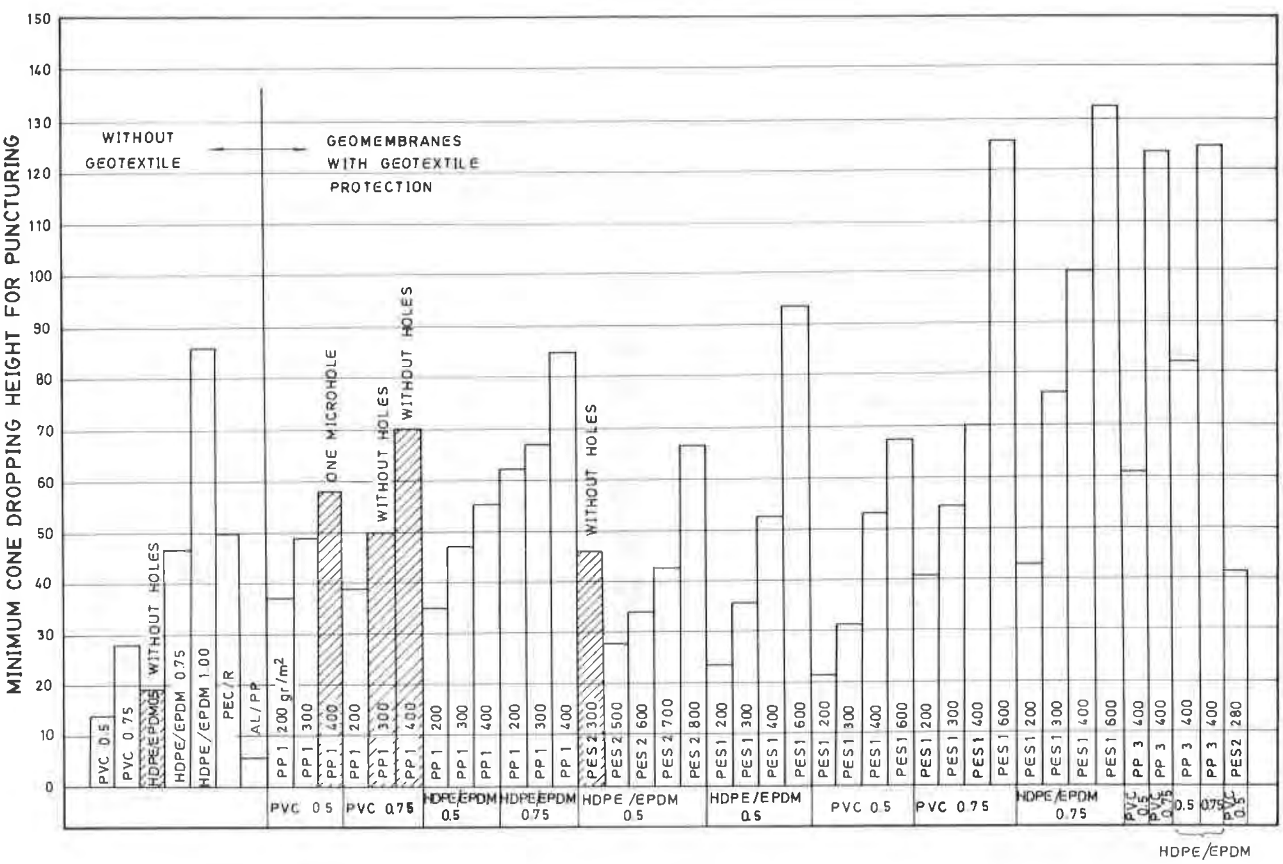

FIG. 3 RESULTS FROM CONE PUNCTURING TESTS

inside the oedometer. Particles with subangular shape and size ranging from 3/8 to 1/2 inches were used (maximum size of the foundation soil was 2 inches).

Due to the geomembrane material composition it was considered to analyze the long term behaviour (creep). As a first approach, tests were conducted by loading steel bearings placed directly over the geomembrane. Figure 4 shows typical results obtained with these type of tests for bearing diameters, D, ranging from 2 to 10 mm (selection of these diameters was in accordance with D_{50} values of the granular mattress and foundation soil, where D_{50} means the ASTM mesh size where 50% by weight of soil is retained). The vertical load, V, applied throught the bearings was equal to 0.25 FS πD^2 p, where p corresponds to the working vertical stress acting on the GGS and FS= 2 is a factor of safety to take into account the subangular shape of the soil particles against bearing roundness. Elastic and creep deformations were defined as shown in Fig. 4. Most of the test results demonstrated that geomembrane deformations due to creep were an important percentage of the elastic deformations. Assuming no plastic failure on the geomembrane, at the end of the heap life period creep deformations could be at least 50% of the elastic deformations. Accordingly, all the oedometer samples were loaded during 2,5 hrs in order to induce creep deformations on the GGS. Moreover, during test N° 11 a loading period of 30 days was used. It is necessary to point out that test N° 11 was performed with those GGS finally selected and with a soil stratigraphy including the compacted granular mattress (according with the oedometer test results geomembranes were punctured when mattress was excluded).

At the end of the oedometer tests all geomembranes were submitted to hole and cracks analysis. Results from this analysis are included in the last column of Table I showing that geomembrane puncturing was mostly controlled by both the soil particle size acting directly on the GGS and the geotextile protection, instead of the geomembrane uniaxial tensile strength. For example, membrane HDPE 2.5 exhibits the higher tensile strength (see Fig. 2), but was punctured becouse no granular mattress was provided. Also, cone puncturing tests does not show good correlation with large scale oedometer tests. For example, membrane HDPE/EPDM 0.5 without geotextile protection exhibits the lower cone index, but it was not punctured at all during the oedometer test.

CONCLUSIONS

1. Uniaxial tensile strength and cone puncturing tests give a crude idea of the GGS behaviour against soil puncture effects. These type of tests are recommended to have a preliminary criteria for selecting geomembrane - geotextile systems.

2. For specific working conditions like those shown for the heap problem, large scale tests have to be used, especially when high stress levels are present. Otherwise, soil particle effects on the GGS could not be properly accounted for.

TABLE I : RESULTS FROM LARGE SCALE OEDOMETER TESTS

LOAD PERIOD	TEST	P (MPa)	GEOMEM-BRANE	THICK-NESS (mm)	GEOTEXTILE	GEOTEXTILE POSITION	GRANULAR MATTRES	ROCK LAYER	GEOMEMBRANE DAMAGE
2,5 hrs	1	0	HDPE/EPDM	0,5	PES2 300		YES	-	WITHOUT HOLES
	2	0,98	HDPE/EPDM	0,5	PES2 300		YES	< 6"	WITHOUT HOLES
2,5 hrs	3	0,98	HDPE/EPDM	1,0	PES2 300		NO	3"-6"	1 HOLE AND 1CRACK 2,6mm
2,5 hrs	4	0,98	HDPE/EPDM	0,75	PES2 300		YES	< 6"	WITHOUT HOLES
2,5 hrs	5	0,98	PVC	0,75	PES1 400		YES	< 6"	WITHOUT HOLES
2,5 hrs	6	0,98	PVC	0,5	PP1 400		YES	< 6"	1 MICRO HOLE
2,5 hrs	7	1,47	PVC	0,75	PP1 300		YES	< 6"	MICRO HOLES IN PROTECTED AREA
2,5 hrs	8	1,47	HDPE/EPDM	0,75	PES2 300		YES	< 6"	WITHOUT HOLES
2,5 hrs	9	1,47	HDPE	1,5	PP3 400		NO	3"-6"	1 HOLE IN PROTECTED AREA 1 HOLE IN UNPROTECTED AREA
2,5 hrs	10	1,47	HDPE	2,5	PP3 400		NO	3"-6"	1 HOLE IN PROTECTED AREA
30 days	11	1,18	HDPE/EPDM	0,75	PES2 300		YES	<6"	WITHOUT HOLES
			PVC	0,75	PP1 400		YES	<6"	WITHOUT HOLES

WITH GEOTEXTILE — WITHOUT

TEST 1 PERFORMED TO SEE EFFECTS DUE TO MATTRESS COMPACTION

TEST 11 TO SEE CREEP EFFECTS

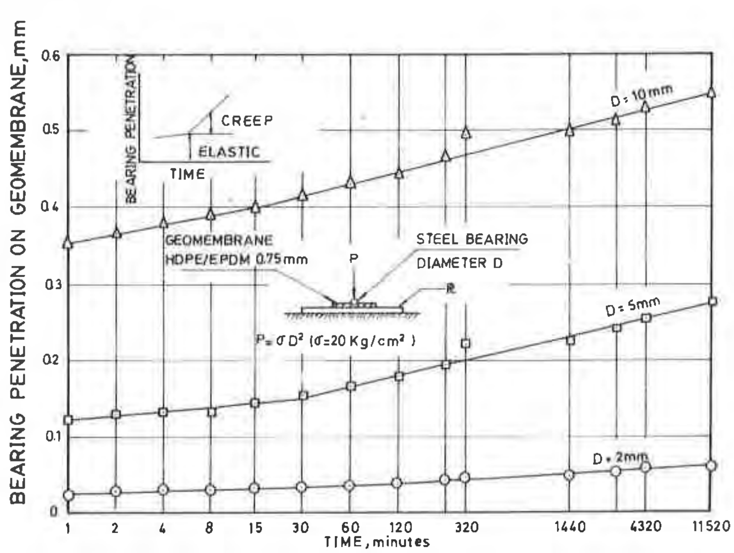

FIG4 RESULTS FROM CREEP TEST ON HDPE/EPDM 0,75 GEOMEMBRANES BY USING STEEL BEARINGS

3. The large scale oedometer used during the test was designed to reproduce field compression stresses acting on the GGS. However, some modifications can be introduced to test samples under both compression and shear stresses, which is a very important condition during seismic events.

4. When GGS are highly stressed by soil particles it is recommended to include long term tests to ensure no puncturing will take place due to creep or plastic failure of the geomembrane materials.

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The French Experience of Geomembranes in Fill Dams

The authors aim at showing both the wide variety of problems met and the solutions found - particularly as regards the dam construction - through the description of seven dams built from 1968 to 1983. The dams dealt with are 12 to 18 m high, the fill being made either of earth or rockfill.

Geomembranes are composed of reinforced bitumen, butyl or P.V.C., as a general rule, only these materials seem to be used so far in France as a waterproof material for fill dams.

As far as higher dams are concerned, the supporting structures were developed according to sophisticated techniques (bituminous concrete, emulsion gravel ...).

In most cases, geotextiles were used as an immediate protective layer of geomembranes. The variety of protective laying techniques implemented as well as the high cost involved seem to suggest that it is a fundamental feature of the watertightness through geomembranes.

This paper is a follow-up to the report presented by J. Labre and D. Loudière (1) concerning the design of dams lined with geomembranes, and it illustrates and completes the latter.

Using data related to seven dams, the authors intend to show how a wide variety of problems were solved, notably during construction. Of course, this method was first used during the 1960s on very low structures, but it was confirmed by use on dams more than 10 metres high ; in this report we shall, in fact, only be dealing with dams more than 10 m high.

It should be noted that this report does not only deal with the technical aspect, but also gives some indication of costs, since this is always one of the deciding factors when choosing a particular technique. It should also be noted that some of the structures were originally designed using other lining systems, in particular upstream linings in bituminous concrete ; during consultation with firms, it was the alternative use of a membrane which, at a given level of security, proved to be less expensive.

Apart from a table summarizing the characteristics of the seven dams, the report contains the following :

- details of two, already long standing dams, Miel and Néris
- description of two large dams, Ospédale and Codole
- particulars of current developments in technique
- some costing factors

I. Two early structures : Miel and Neris.

The Miel dam in Corrèze (France) was completed in 1968. The fill is made up of a quarry run of granitic origin, comprising a minimum of boulders. No precautions were taken to ensure that these boulders were not flush with the upstream slope of the fill. The lining provided is 1 mm thick butyl and the supporting layer is made up of 5/30 rolled filter gravel, 15 cm thick.

Given the relatively weak slope of the fill (1/2.5) no problem of surface stability presented itself. The protection is comprised of a rockfill 70 cm thick overlying a transition layer 20 cm thick. Anchorage is ensured at the toe of the slope by a cut off trench filled with concrete.

In order to obtain a better guarantee of watertightness at the sheet connections, it was arranged for the seaming process to take place in the workshop and the whole lining (2740 m^2 in all) to be transported in three sheets, which meant that only two lines of seaming would be required on site. To make the laying and cutting of the geomembrane easier, the earthworks were carefully executed in simple geometric shape.

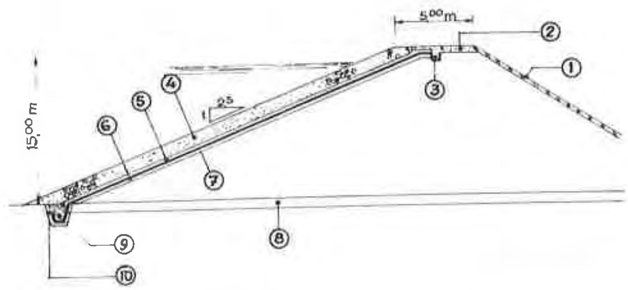

Fig.1 Miel dam - Typical section.

1. Earth protection
2. Sand crest protection
3. Anchorage trench
4. Rip - rap
5. Geomembrane
6. Filter
7. Earthfill
8. Drain
9. Cut-off trench
10. Concrete

The important facts to note about the Miel dam are the absence of geotextiles and the importance of the transition layers ; with respect to the geomembrane, one should note the reduced length of in situ seaming and the perfect behaviour of the structure after 15 years.

The Néris dam is a rockfill dam. It is higher than the Miel dam and, in particular, has much steeper slopes (1/1.6 for the upstream slope). The upstream face was shaped with a layer of 40/60 materials then faced with a

CHARACTERISTICS OF THE DAMS

Name of dam	Year built (end)	Designer	Locality (department)	Height (m)	Volume stored ($10^6 m^3$)	Purpose	Slope of upstream face	Geomembrane (thickness in mm)	Area of lining (m^2)	Support	Protection
MIEL	1968	Ministère de l'Agriculture	CORREZE	15		Leisure activities	1/2.5	Butyl (t = 1)	2740	Rolled graved	Rockfill on sand
NERIS	1972	Ministère de l'Équipement	ALLIER	18		Drinking water supply	1/1.6	Butyl (t = 1.5)		Open graded bituminous concrete	Prefabricated slabs on geotextile
OSPEDALE	1978	SOMIVAC	CORSE	26	3	Drinking water supply	1/1.7	Reinforced bitumen (t = 4)	5000	Geotextile on open graded bituminous concrete	Paving blocks on geotextile
CODOLE	1983	SOMIVAC	CORSE	28	6.5	Drinking water and irrigation	1/1.7	P.V.C. (t = 2)		Geotextile on open graded bituminous concrete	Slabs poured in-situ
LANOUAILLE	1983	Ministère de l'Agriculture	DORDOGNE	12	0.8	Leisure activities	1/2.5	Reinforced bitumen (t = 4)	4700	Geotextile	Rockfill
LA LANDE CHITELET	1983	Ministère de l'Équipement	VOSGES	17	0.6		1/1.7	Reinforced bitumen (t = 4)	1700	Geotextile on stabilized gravel	Paving blocks on geotextile
MAS D'ARMAND	1981	SOMIVAL	LOZERE	21	0.84	Leisure activities		Reinforced bitumen (t = 4)	9800	Geotextile	Partial paving blocks stuck onto geotextile

cold open graded bituminous concrete 3 to 4 cm thick. The lining itself is comprised of butyl 1.5 mm thick.

The geomembrane is protected by a geotextile on which 4 cm thick prefabricated concrete slabs with imbricated points are set, each one measuring 0.80 x 1 m and pierced with 6 holes designed to improve drainage beneath the slab.

The geomembrane has been anchored just below the crest and at the toe it has been embedded in a cut off trench filled with concrete without formwork to an average height of two metres. In 1983 the Neris town council confirmed the good behaviour of the dam and the absence of any problem since it was constructed in 1971 and 1972.

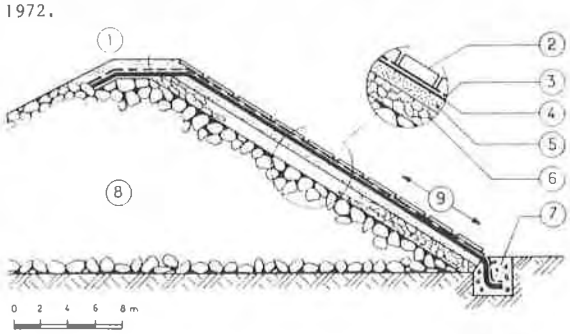

Fig.2 Neris dam - Typical section.

1. Earth
2. Prefabricated slabs
3. Geotextile
4. Butyl geomembrane
5. Bituminous concrete
6. Run - of quarry stone
7. Concrete
8. Rockfill
9. Slope 1/1.6

II. Two large dams : Ospedale and Codole.

The general characteristics of the two dams are shown at the beginning of this report and their design principles were given in another report (1). Therefore only details concerning their construction are given below.

2.1. The Ospédale dam (2).

The supporting layer is successively comprised of :

- a compacted 25/50 ballast, impregnated with a bituminous emulsion.
- filtration by means of cold open graded bituminous concrete, at a mean proportioning of 140 kg/m^2. This material is laid using a hopper driven by winches set on the crest. It is compacted by the hopper itself, which has two smooth-wheeled rollers.
- a layer of non-woven polyester geotextile 0.270 kg/m^2 (BIDIM U-34), delivered in 4 m wide strips and rolled down by hand. The geotextile is stuck on to the support at different points to ensure that it will not be displaced by wind during the laying.

The geomembrane used is of the reinforced bitumen type (COLETANCHE NTP 3) ; it is 0.005 m thick and weighs 5.5. kg/m^2. It is a heavy geomembrane. It is delivered in strips 4 m wide. It was positioned :

- over short distances (less than 30 m) by placing the roll of geomembrane at the top and pulling it down by hand towards the bottom.
- over long distances (more than 30 m) by means of a trolley driven on the face with winches on the crest (without sliding of the geomembrane on the support).

Fig. 3 Ospedale dam. View of upstream slope.

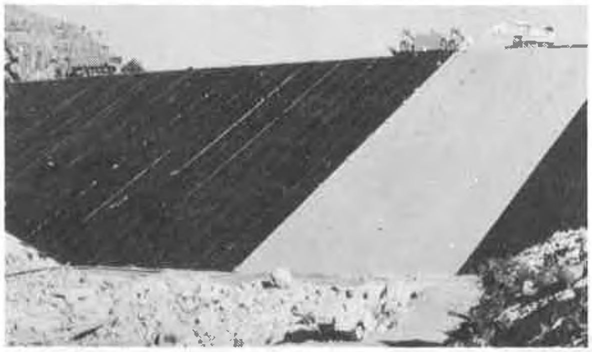

Fig.4 Ospedale dam. Laying the upstream facing.

The joints are seamed over a width of 0.30 m, using a heavy wheel-mounted welding machine, activated by a winch positioned on the crest. Seaming is carried out from the bottom upwards so that the liquefied bitumen does not run and thus effectively contributes to the seaming process.

Control of the seaming is performed by means of an ultrasonic device which makes it possible to detect any eventual discontinuity in the seaming which is relatively frequent when carried out on such a large scale.

The top connection is carried out in the conventional way by burying the geomembrane in a trench.

The bottom connection is only carried out on concrete structures already fitted with a fixed metal plate. This plate has bolts which make it possible to clamp down the geomembrane by means of a counterplate in galvanized steel.

For reasons of security, there is a double membrane in this zone over a width of 2 m. The geomembrane is stuck to the concrete beyond the metal plate over a width of 0.30 m. Finally, the whole area is covered by a 0.50 m strip of bituminous geomembrane, which protects the counterplate.

The protective layer was executed in two stages :

- first of all the non-women geotextile identical to that of the support (BIDIM U-34) was stuck onto the geomembrane at various points ;
- then a layer of sinusoidal self-locking concrete blocks 0.08 m thick was laid by hand.

The only problems encountered during construction were those due to wind (see paragraph relative to the Lande Chitelet dam) and those concerning the fitting of the self-locking concrete blocks.

The blocks were laid from the bottom upwards and each successive layer gradually loses its regular alignment because of the irregularities of the support. It then becomes increasingly more difficult to fit the blocks together and on the upper part of the structure there are, locally, important installation defects, in particular large gaps.

OSPEDALE is the oldest of the large dams lined with a thin geomembrane and a follow-up provides very valuable information.

Generally speaking, this structure has an excellent behaviour ; its settlements are extremely slight and leakages restricted (barely 1 l/s dam full).

Only the self-locking concrete blocks have presented problems : at the time of heavy storms several of the blocks were torn away in places (350 m^2 at the maximum). This damage was easily repaired and the new disposition of the blocks seems an improvement on their former arrangement.

2.2. The Codole dam (3).

The construction principle for the supporting layer is the same as the one used in the building of the OSPEDALE dam. Here, however, the polyester geotextile layer is already stuck to the membrane, and is therefore laid at the same time as the latter.

At CODOLE, the geomembrane is made of flexible P.V.C. 2 mm thick (TERSOM V), and weighing 2.8. kg/m^2. A layer of polyester geotextile (400 g/m^2) is glued to the underside of this membrane at the works and this brings the total weight to 3.2. kg/m^2. This lining is delivered in strips 2 m wide, long enough to cover the whole face without horizontal seaming (maximum 50 metres). The lining was positioned by placing the roll on the crest and pulling it down by hand : just two men can roll down 60 m or more of membrane in spite of the friction of the geotextile on the bituminous concrete of the support.

The strips are seamed at the joints over a width of 50 mm (the felt is obviously not present in the seaming band). This seaming is carried out entirely by hand, using a hot air welding torch : a good welder can execute 500 m seaming per day. The electronic control devices did not give very goo results, so a manual and visual control on the seams was also carried out : the percentage of defective seams (therefore redone) was in the order of 0 to 5 % depending on the skill of the welder. It should be noted that this operation requires certain precautions if one is to avoid some slight slipping which can happen if two assembled strips do not have the same temperature, the same prestressing when rolled up, or have undergone a slightly different manufacturing process, in particular in the caseof a PVC-geotextile compound. Generally speaking it is always advisable to leave the strips lying in-situ for two or three hours before beginning the seaming process.

Compared with the OSPEDALE dam the connections have been greatly perfected and merit further explanation.

- The top connection was made on a longitudinal concrete beam, using a metal plate fixed by means of chemical anchor bolts (chemical anchor bolts are screws fixed to the concrete by means of quick-setting epoxy resin inserted in the form of cartridges).

- The toe connection was carried out on concrete structures by means of a high density polyethylene (H.D.P.E.) plate, fixed by chemical anchor bolts. Seepage between the membrane and the concrete is controlled, under the plate, by a triple joint (a band of polyurethane mastic between two bands of chlorinated butyl). In addition, the geomembrane is folded back on itself across the plate and welded down (photo).

The protective layers comprises :

- a layer of polyester geotextile 400 g/m^2
- a layer of concrete 0.14 m thick, poured directly onto the felt layer, in regular 4.5 m x 5 m slabs.

The vertical joints between the slabs are open in order to allow underpressure drainage. The horizontal joints are simply scored on the surface to allow for cracking. The slabs were cast by means of a light hopper (1.5 m^3) driven on the face by a winch positioned on the crest. The formwork was executed in units of 2.50 m so

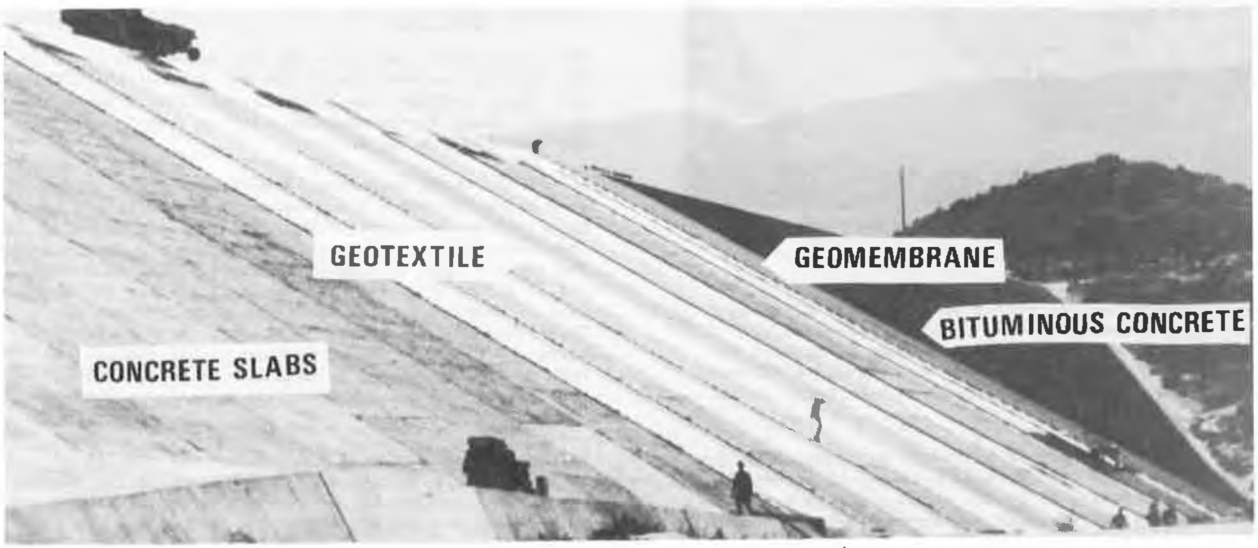

Fig. 5 Codole dam - The end of the construction

that they could be shifted manually, and to allow for the irregularities of the support.

In December 1983, the dam was in the process of being filled and seemed entirely satisfactory (total seepage flow 2 l/s).

III. Development of the technique

Three recent examples will be used to show the variety of problems met with and the solutions brought to bear on them.

3.1. The Lanouaille dam

The Lanouaille dam was built during the summers of 1982 and 1983 under the superintendence of the Direction Départementale de l'Agriculture, by two local firms (earthworks) and a specialist firm (laying of the membrane).

At the dam location, the substratum is comprised of micaschists overlaid with 3.50 m of peaty materials which had to be removed before construction of the dam. The latter is made up of a sand-gravel mixture with silt resulting from the weathering of the micaschists ; watertightness is ensured by a geomembrane placed on the upstream face, laid in the following way :

- the surface of the compacted fill was prepared using a hydraulic loading shovel and finished off by hand (removal of some pebbles).

- next there was successive laying of :

. drains

. a non-woven textile (380 g/m^2) (ensuring protection of the under-surface of the geomembrane and drainage beneath the latter)

. the geomembrane (TERANAP 431 TP : elastomer - bitumen + reinforcement in non-woven polyester, 4 mm thick)

. a non-woven geotextile (380 g/m^2)

. a layer of 0/30 mm rolled sand-gravel mixture, 0.15 m thick

. and 0/250 mm rock fill (on the upper part only, set on a small berm 0.50 m wide, 0,35 m thick).

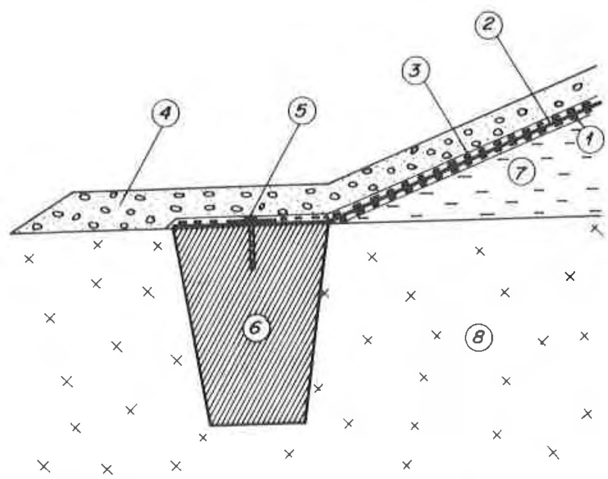

Fig.6 Lanouaille dam - Detail of toe anchorage

1. Geotextile
2. Geomembrane
3. Geotextile
4. 0-30 sand gravel mixture
5. Steel plate
6. Concrete cut off trench
7. Earth fill
8. Bedroch

The géomembrane was laid in rolls 4.00 m wide, which were rolled down from the crest of the dam ; the seaming was carried out by heating the two bands with a hot air welding torch and manual pressure. The membrane is anchored on the crest in a trench 0.40 x 0.40 m. At the toe, connection between the geomembrane and the substratum is ensured by a cut off trench hollowed out by a rock breaker in the unweathered substratum (0.80 m deep and at least 0.50 m wide) and filled with concrete. The geomembrane was stuck when hot to the concrete structure, which was previously impregnated with a bituminous product over a width of 0.30 m ; this connection was consolidated by mechanical means by the positioning of a metallic strip 20 x 2 mm nailed with a cartridge gun (spacing between the nails = 15 cm).

Fig.7 Lanouaille dam - Laying the membrane.

The dam was in the process of being filled in 1983.

3.2. The Lande-Chitelet dam

This is a rockfill dam whose upstream face was covered with a 0.15 m layer of 20/40 grain size materials before installation of the lining.

This semi-crushed 20/40 material can damage the geomembrane because of the sharp edges it presents and the hydraulic head to be withstood. Moreover it is insufficiently stable to enable work to be satisfactorily carried out on a 1/1.7 slope. It was stabilized with 10 kg/m^2 of gritted bituminous emulsion with roughly 15 l/m^2 of 6/10 stone chips.

This operation was carried out using a special trolley fitted with a lance for spreading the emulsion and a hopper provided with trap doors at the base for chipping. This trolley moved about on two smooth-wheeled rollers which also ensured roller compaction of the surface. The movement of the whole equipment was controlled by a cable operated by means of a winch positioned on the crest ot the structure.

Then a 0.270 kg/m^2 geotextile (BIDIM U-34) was unrolled over the whole surface. The 5.30 m wide roll was held by an hydraulic loading shovel at the top of the structure and the sheet pulled down manually by workmen moving from the top to the bottom of the facing. Before installing the subsequent layers, the geotextile was ballasted with sand bags to prevent any movement due to wind. The different bands of BIDIM were sewn together.

The lining, comprised of a bituminous geomembrane COLETANCHE NTP 3 (thickness 0.005 m, weight per unit area 5.5. kg/m^2, roll 4 m wide and 65 m long) was laid in the same way as the geotextile. The 0.20 m overlap between the bands was seamed when hot by means of handheld gas welding torches. By measure of security all the joints were covered with a 0.30 m-wide strip of geomembrane, stuck on when hot.

During the site work, when the geomembrane had been connected along the whole of its periphery (anchorage in a trench at its upper part, and connection by sticking and clamping down with a fixed metal plate along the cut off wall) a violent wind lifted up the geomembrane locally to a height of 0.20 to 0.30 m. Folds were formed diagonally in the direction of the wind on a part of the dam lining.

Theses folds were lined up at the foot of the structure, cut open with a sharp tool and flattened out, and then all covered with a 1 metre wide strip of geomembrane, stuck on when hot.

The lining is protected by sinusoidal, 0.08 m thick, concrete blocks which are self-locking in all directions and laid by hand after installation of an intercalated geotextile (BIDIM U-34) on top of the geomembrane.

In order to fully interlock with each other, the sinusoidal blocks must be very regular and of excellent quality, with no concrete burrs or cement grout. On the site, a great many poor quality blocks had to be rejected.

Initially, impervious bituminous concrete had been planned for the lining ; the connection of this layer with the cut off wall was to be carried out by giving the bituminous concrete a relatively pronounced curve. The preliminary design of the support not having been modified for the solution using a geomembrane, there was no question of laying self-locking blocks on a curved surface. The connection of the layer of blocks with the cut off wall was therefore carried out by means of a longitudinal concrete beam, varying in width from 1 m to 1.50 m.

3.3. The Mas d'Armand dam (4)

The MAS D'ARMAND dam is used to form a lake of constant level with an area of 14 hectares, inside a reservoir created by the construction of the NAUSSAC dam, which regulates the flow of the Haute Allier.

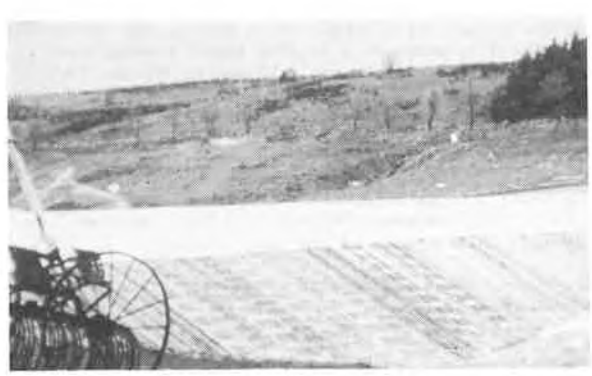

Fig. 8 Mas d'Armand dam - General view.

One of the characteristics of this structure, therefore, is that it retains water at a constant level upstream and has its downstream part immersed, but with a variable water level.

The upstream lining of the dam is on a support impregnated with gritted bituminous emulsion and comprised of a 0.270 kg/m² geotextile (BIDIM U-34), overlaid with a COLETANCHE NTP 3 bituminous geomembrane (thickness 0.005m, weight per unit area 5.5. kg/m², roll 4 m wide and 65 m long). There is no lining on the downstream part of the structure.

Taking into account the fact that upstream the height of the water is constant, the lining is only protected at water level against wave action. This protection is ensured by self-locking concrete blocks, 0.08 m thick, stuck onto a sheet of woven geotextile. This sheet is fixed at its upper part by burial in the trench which is used to anchor the bituminous geomembrane. The sheet is not stuck onto the lining and is not fixed at its lower part ; it is only held in place by the anchor trench and friction on the geomembrane.

Fig. 9. Mas d'Armand dam - Upstream slope facing.

Because of the work schedule and the time of year the earthworks were executed, the MAS D'ARMAND dam was built in two stages. Thus the lining of the dam, 21 metres in total height, was carried out in the course of two distinct site operations, the first for a height of 7 metres, and the second for a height of 14 metres, the geomembrane being laid by means of a loading shovel moving about on the crest. The geomembranes laid during the two working stages are joined by an overlap 0.40 m wide seamed when hot, on a berm two metres wide.

IV. Some costing factors :

The table set out below makes it possible to compare the prices of "bituminous concrete" and "thin membrane" as linings, on the basis of the same tender.

The prices shown are updated to January 1984, and do not include V.A.T. :

. September 1977 for OSPEDALE

. April 1980 for CODOLE

The surface area of the lining is 4.000 m² at OSPEDALE and 16.000 m² at CODOLE.

TABLE OF COSTING FACTORS

	OSPEDALE		CODOLE	
BITUMINOUS CONCRETE	**F.F.**	**US $**	**F.F.**	**US $**
– Total cost	4.600.000	575.000	11.049.000	1.381.125
– Cost per m²	1.150	144	690	86
THIN MEMBRANE				
– supporting layer	551.000	68.875	1.452.000	181.500
– lining	457.000	57.125	1.935.000	241.875
– protective layer	1.637.000	204.625	4.687.000	585.875
– connections and other work	772.000	96.500	832.000	104.000
– total cost	3.417.000	427.130	8.906.000	1.113.250
– Cost per m²	854	107	557	70

For the Lanouaille dam, two approximate average prices can be given, (excluding V.A.T. January 1984) :

80 F/m² Construction of the cut off trench filled with concrete, connections to the structures, and upper and lower geotextiles.

and

118 F/m² adding the rockfill and gravel protection.

These factors are designed to show the large variation in costs according to the precautions taken and the small amount which the cost of supplying and laying the geomembrane represents. It is certain that geographical factors on the island of Corsica greatly influence costs.

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Seven dams have been described in this report. The authors have attempted to emphasize particular aspects of each site where membranes were laid.

On these sites it has been shown that the technical skill of the firms is necessary for a successful conclusion of the work ; in effect, certain important details are worked out with firm's engineers (connections, support finish,..). The laying of the membrane and the execution of the joints are all the more successful if the work is carefully carried out, controlled and checked and numerous precautions taken (wind, rain, differential deformations, etc.).

Progressive evolution in techniques, the existence of long-standing points of reference, and the good behaviour of the structures described constitute the best proof that the most highly qualified and experienced designers and firms in the field of dam construction are certainly in a position to use geomembranes in structures of up to fifty metres in height.

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Permeability of Polymeric Membrane Lining Materials

Permeabilities to three gases (carbon dioxide, methane, and nitrogen), water vapor, and five solvents (methanol, acetone, cyclohexane, xylene, and chloroform) are reported for a broad range of commercial polymeric membranes. Gas and water vapor transmission (WVT) data were determined by ASTM methods and solvent vapor transmission data were obtained in a modified WVT procedure. Permeability characteristics of thermoplastic and partially crystalline membranes were also assessed in pouch-type tests with salt solutions, actual wastes, acetone, xylene, and water-soluble and organic-soluble dyes. All membranes were permeable to some extent. Magnitude and direction of fluid transport vary with the membrane composition and its thickness, temperature, the permeant fluid, and the driving force which depends upon the concentration or the vapor pressure gradient across the membrane. Permeability depends upon the solubility and diffusion characteristics of the permeant in the particular membrane.

INTRODUCTION

Because of their low permeability to gases, liquids, and vapors, flexible polymeric membranes offer a feasible approach to meeting the requirements of the U.S. Environmental Protection Agency's regulations for the lining of waste storage, treatment, and disposal facilities to prevent migration into the groundwater of waste liquids and their constituents. In the initial use of liners for waste disposal facilities, the primary purpose was to control the transmission of water and trace metals to the groundwater. To these species polymeric membranes have shown extremely low permeation. However, polymeric compositions are well known to be permeable to organic species (7, 8), and differ greatly in magnitude, depending on the polymeric material and the permeating fluid or molecular species. The migration of organics, which are of great concern in groundwater protection, should also be controlled by membrane liners. From the standpoint of designing lined hazardous waste facilities, it is important to know the magnitude of the permeability of the polymeric membranes to different constituents and combinations and to know the driving forces that are involved so that transmission rates can be estimated.

This paper discusses the permeability of polymeric membrane liners and presents experimental results for a range of commercial polymeric membranes to gases, water vapor, and solvent vapors. Also, presented are current results of studies of the permeability characteristics of membranes as observed in pouch experiments.

TRANSPORT THROUGH POLYMERIC MEMBRANES

In contrast to soils, polymeric membrane liners are nonporous. In soils the flow of liquids containing dissolved constituents is through the pores with hydraulic gradient the principal driving force. For nonporous membranes, the transport of a permeating fluid, be it gas, liquid, or vapor, depends on the solubility of the constituent in the membrane and its diffusibility in the membrane. This transport proceeds essentially in three steps:

1. Dissolution of the fluid or constituent into the membrane.
2. Diffusion of the fluid through the membrane.
3. Evaporation or dissolution of the fluid on the downstream side of the membrane.

Step 1 depends upon the solubility of the fluid in the membrane and relative "activity" of the fluid on both sides of the interface. The diffusion through the membrane in Step 2 involves a variety of parameters which include the molecular size of the permeating fluid and various characteristics of the membrane compound, e.g. presence of fillers, crystalline zones, plasticizers, crosslinks, etc. Step 3 is similar to the first step and depends upon the relative "activity" of individual constituents on both sides of the interface at the downstream liner surface.

The major driving force for the movement of a given constituent through a membrane is its concentration gradient across that membrane. Each constituent in a gaseous or liquid mixture tends to move independently from a higher to a lower concentration.

GAS PERMEABILITY

Permeability to gases, particularly methane, is an important factor in the use of polymeric lining materials for the control of gas migration in a variety of engineering applications. These include such uses as covers and curtain walls to control movement of methane from landfills and as barriers to prevent entrance of methane into buildings and other structures near municipal solid waste (MSW) landfills.

The permeability of membrane liner materials to three gases of interest in land disposal facilities was determined by ASTM D1434, Procedure V (1). In this procedure a specimen of the liner is clamped into a stainless steel cell so as to form a barrier to gas flow through the cell. All air is flushed from the system with the test gas and then one side of the cell maintained at a positive pressure while the other remains at atmospheric pressure. A capillary mounted on the atmospheric pressure side of the cell is used to monitor the volume of gas slowly diffusing through the liner. The test apparatus is shown in Figure 1.

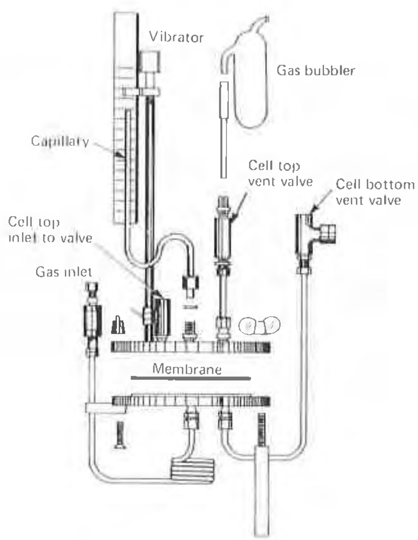

Figure 1. Gas permeability apparatus in ASTM D1434, Procedure V (1).

In determining permeability, both the system pressure and cell temperature must be maintained at constant values throughout the test. Reproducible data cannot be obtained until the test gas, cell, and membrane are at thermal equilibrium and the membrane has reached equilibrium in its absorption of the test gas. When these conditions are met, the passage of test gas through the membrane will reach a steady state rate and the permeability determination is made.

Data collection consists of tabulating the linear displacement of the liquid meniscus within the capillary versus time. These data can be reduced to a displacement rate using a linear regression routine and further reduced to the gas transmission rate (GTR) in terms of gas volume/area x time x Δ pressure if the capillary diameter, specimen surface area, and applied pressure differential are recorded. Determination of the gas transmission rate at two or more temperatures allows the calculation of an activation energy (E_d) and of a preexponential factor (log GTR_0) for the transmission process. Knowledge of these two parameters for a given liner-gas combination makes possible the calculation of the gas transmission rate at any temperature.

Figure 2 illustrates the relation between temperature and transmission rate of carbon dioxide, nitrogen, and methane through an elasticized polyolefin (ELPO) membrane. This relationship is roughly linear over a moderate temperature range.

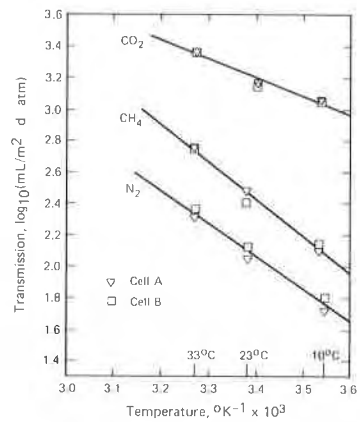

Figure 2. Permeability of ELPO to CO_2, CH_4, and N_2 as a function of temperature. Data are reported as GTR for a membrane of 0.158 mm under pressure difference of one atmosphere.

The data for CO_2 are tabulated in Table 1.

TABLE 1. TEMPERATURE DEPENDENCE OF LINER PERMEABILITY TO CARBON DIOXIDE

Cell	Temperature, °C	$(T°K)^{-1}$ X values	GTR, mL(STP)/ $m^2 \cdot d \cdot atm$	Log_{10} GTR Y values
A	10	3.531×10^{-3}	1120	3.0492
A	23.1	3.395×10^{-3}	1140	3.1492
A	33.0	3.266×10^{-3}	2290	3.3598
B	10	3.531×10^{-3}	1150	3.0607
B	23.1	3.395×10^{-3}	1490	3.1732
B	33.0	3.266×10^{-3}	2330	3.3674

Data for the permeability of a series of polymeric membrane liners to carbon dioxide, methane, and nitrogen at 23°C are presented in Table 2.

These data presented on gas permeability show:

1. Major differences in permeability among the lining materials.
2. Permeability of polymeric materials differs for a given generic polymer type due to compound variation, (e.g. filler and plasticizer contents).
3. Rates of transmission varied with the gas.
4. Gas transmission decreased with increased thickness of the membrane.

5. Higher polymer crystallinity yields lower permeability, as shown when comparing LDPE, LLDPE (linear low-density polyethylene), and HDPE.

6. Permeability of polymeric membranes to gases increases with temperature.

TABLE 2. PERMEABILITY OF POLYMERIC MEMBRANE LINERS TO GASES AT 23°C DETERMINED BY ASTM D1434, PROCEDURE V

Polymer	Liner serial number[a]	Thickness, mm	Type[b]	Gas transmission rate, mL (STP[c])/m²·d·atm		
				CO_2	CH_4	N_2
Butyl	44	1.60	XL	512	120	19.7
CPE	77	0.72	TP	106[d]	6.31[d]	≤1.45[d]
CSPE	6R	0.82	TP	122	21.6	26.2
CSPE	55	0.86	TP	418	124	27.1
ELPO	36	0.58	CX	1450	280	125
EPDM	83R	0.89	TP	2720[e]	...	...
EPDM	91	0.90	XL	5260	1400	314
EPDM	8	1.50	XL	...	470[f]	...
Neoprene	90	0.90	XL	716	80.9	31.1
PB	221	0.71	CX	818	248	62.3
HDPE	265	0.61	CX	729	138	...
HDPE	269	0.86	CX	467	104	...
LDPE	21	0.25	CX	6180[e]	1340[e]	...
LLDPE	281	0.46	CX	1370	322	...
PVC	93	0.25	TP	7730[e]	1150[e]	
PVC	88	0.49	TP	3010	446	108
PVC	59	0.81	TP	2840[e]	285[e]	...

[a]Matrecon liner serial number; R = Fabric-reinforced.
[b]XL = Crosslinked; TP = Thermoplastic; CX = Crystalline.
[c]STP = Standard temperature and pressure.
[d]Measured at a pressure gradient of 40 psi; all others measured at 20 psi.
[e]Measured at 30°C.
[f]Measured at 20°C.

WATER VAPOR TRANSMISSION (WVT)

The permeability of polymeric membranes to water vapor is important to a variety of applications such as the covers for reservoirs and other impoundments, the lining of canals and tunnels, and as a moisture barrier in various buildings and structures.

The WVT rates of polymeric membrane liners were determined by ASTM E96, Procedure BW (2). In this procedure, a circular specimen of the liner is sealed with molten wax into the mouth of an aluminum cup partially filled with deionized water. The test cell is illustrated in Figure 3. The entire assembly is kept in an inverted position so that water is in contact with the membrane surface and stored in a cabinet maintained at 23°C and a relative humidity (RH) of 50 ±5% and equipped with a small fan to ensure uniform air velocity over the surfaces of the specimens as required by the procedure. The WVT occurs across a water vapor pressure gradient of 100% RH (inside the cup) to 50% RH. Transmission of one g/m²·d of water is equal in practical units to 1.07 gallon per acre per day.

The test cells are periodically weighed and the resulting data are reduced using a linear regression program to yield a loss rate which can be converted to a WVT value in units of g/m²·d. WVT data for a series of polymeric membrane liners are presented in Table 3 by polymer and increasing thickness.

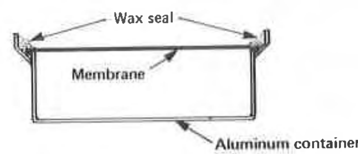

Figure 3. Cross section of water vapor transmission cup used in ASTM E96 (2).

TABLE 3. WATER VAPOR TRANSMISSION (WVT) RATES OF POLYMERIC MEMBRANE LINERS

Polymer	Liner serial number[a]	Thickness, mm	Type	WVT, g/m²·d
Butyl rubber	163R	0.85	XL	0.384
	57	0.85	XL	0.020
	22	1.85	XL	0.097
CPE	86	0.53	TP	0.643
	135	0.79	TP	1.400
	77	0.79	TP	0.320
	12	0.85	TP	0.264
	147R	0.94	TP	0.305
	165	0.97	TP	0.643
CSPE	169R	0.74	TP	0.333
	148	0.76	TP	0.663
	55	0.89	TP	0.438
	151R	0.91	TP	0.748
	6R	0.94	TP	0.422
	170R	1.07	TP	0.252
ELPO	36	0.72	CX	0.142
ECO[b]	178	1.160	XL	20.18
	178	1.650	XL	14.30
EPDM	41	0.51	XL	0.270
	83	0.94	XL	0.190
	8	1.70	XL	0.172
Neoprene	42	0.51	XL	0.304
	168	0.91	XL	0.473
	167R	1.27	XL	0.429
	9	1.59	XL	0.237
Nitrile rubber	171R	0.76	TP	5.51
PB[c]	221	0.69	CX	0.084
PEEL[d]	75	0.20	CX	10.50
LDPE	108	0.76	CX	0.0573
HDPE	184	0.80	CX	0.0172
	179	2.44	CX	0.0062
HDPE-A	181	0.86	CX	0.0472
PVC	89	0.28	TP	4.42
	17	0.51	TP	2.97
	146	0.76	TP	1.94
	143	0.79	TP	1.85
PVC-E	176R	0.91	TP	2.78
PVC-OR	40	0.83	TP	4.17
Saran film (0.5 mil)	222	0.013	TP	0.563

[a]Matrecon serial number; R = fabric-reinforced.
[b]ECO = epichlorohydrin rubber.
[c]PB = polybutylene.
[d]PEEL = polyester elastomer.

As with the gas transmission data, WVT varies considerably among the polymer types; for example, the rates are much lower through hydrocarbon types (e.g. butyl rubber, EPDM, ELPO), than through polar types (e.g. ECO, nitrile rubber). Also, within a polymer type there is considerable variation due to differences in composition. Increased thickness and increased crystallinity, in the case of crystalline materials, reduce transmission rates.

SOLVENT VAPOR TRANSMISSION (SVT)

Considerable data (7, 8) exist with respect to the transport of organics through polymeric films, but almost no data exist with respect to polymeric membrane liners. Preliminary experiments were performed on neat solvents to assess their transmission rates under controlled conditions through different membranes. Solvent vapor transmission rates were determined by a modification of ASTM E96 in which a circular specimen of the liner is mechanically clamped onto the mouth of an aluminum cup partially filled with the test solvent (Figure 4). The method differs from E96, Procedure BW, in that the cups are stored in an upright position so that solvent vapor contacts the membrane. SVT occurs as a result of the concentration gradient across the membrane by the presence of a saturated atmosphere within the cup and the essentially zero level outside the cup. The rate of SVT is determier as described above for WVT.

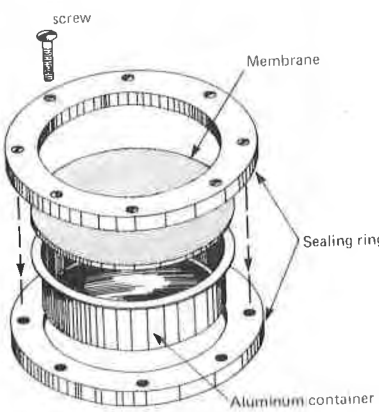

Figure 4. SVT cup with aluminum sealing rings.

SVT data for a series of membranes selected for good solvent resistance are given in Table 4. These data are for five organic solvents, i.e. methanol, acetone, cyclohexane, xylene, and chloroform. Although limited, the data show substantially different transmission rates among the membranes and among the different solvents. Increased crystallinity among the polyethylene membranes reduces transmission, as does increased thickness. The alloying of HDPE results in significantly higher transmission.

PERMEABILITY OF MEMBRANES IN POUCH TESTS

The permeability characteristics of thermoplastic and partially crystalline polymeric membrane have been studied in a pouch procedure which simulates some in-service conditions of liners in waste disposal facilities (3, 4, 5, 6,). In this procedure, small containers are fabricated of the membrane to be tested and filled with a test liquid, sealed, and immersed in a liquid of known composition. When the outer liquid initially is DI water, permeabilities of the membrane to water and to the dissolved constituents can be determined by observing with time the change in the weight of the pouch and changes in the pH, electrical conductivity, and the composition of the outer water. Because of differences in the concentrations of different molecular species across the membrane wall, osmosis will cause the outer water to enter the pouch and cause ions and dissolved constituents in the pouch liquid to tend to leave the pouch and enter the outer water. An increase in weight of the pouch may only reflect absorption of the outer water by the wall and not permeation of water into the pouch. When the test is completed the pouch and its contents are weighed separately to determine the actual amounts that have been absorbed by the wall and the amounts that have permeated the pouch wall.

TABLE 4. SOLVENT VAPOR TRANSMISSION (SVT) RATES OF POLYMERIC MEMBRANE LINERS BY ASTM E96, MODIFIED

Polymer	Liner serial number	Thickness, mm	Type	SVT, $g/m^2 \cdot d$ Methyl alcohol	Acetone	Cyclohexane	Xylene	Chloroform
CSPE	170R	1.07-1.12	TP	...	221	...	...	...
ELPO	172	0.53-0.61	CX	2.10	8.62	760	359	3230
HDPE	184	0.77-0.83	CX	0.16	0.56	11.7	21.6	54.8
	179	2.42-2.81	CX	...	...	...	6.86	15.8
HDPE-A	180	0.53	CX	...	...	...	295	...
	181	0.85-0.88	CX	0.50	2.19	151	212	506
	182	0.97	CX	...	...	...	220	...
LDPE	108	0.74-0.76	CX	0.74	2.83	161	116	570
PB	221	0.64-0.74	CX	0.35	1.23	616	178	2120
Teflon	234	0.10	CX	0.34	1.27	0.026	0.16	20.6

Experimental Results

The studies of the permeability of polymeric membranes using the pouch procedure included membranes of CPE, CSPE, HDPE, HDPE-A, LDPE, PB, PVC, and PVC-OR (4). The test liquids included aqueous salt solutions (of sodium chloride and of lithium chloride), organic solvents (acetone and xylene), mixtures of acetone and water, water-soluble and organic-soluble dyes, and several actual waste liquids.

Permeability to Water

Of nine pouches filled with the 5% NaCl aqueous solution, eight showed the transmission of water into the pouch, as judged by the increased weight of the pouch contents (Table 5). The ninth pouch, CSPE 55, gained in weight but showed a weight loss for its contents indicating absorption of the pouch liquid by the wall. The pouches with a highly acidic waste showed the transmission of water from outside to inside the pouches. The weight changes during exposure are shown in Figure 6. Table 6 presents the weight data on the pouches when they were dismantled. The results indicate a low permeability for ELPO. The behavior of pouches containing a highly alkaline waste was similar except for the ELPO pouch which gained weight after 300 days of exposure, indicating a slowly developing effect of the waste on this membrane.

The effect of the concentration gradient across the pouch wall is clearly demonstrated in the PVC pouches with LiCl (Figure 7). The pouches containing 5% LiCl show a rate of transmission that is approximately half that of the pouches with 10% LiCl.

TABLE 5. POUCH TEST OF MEMBRANES WITH 5% NaCl AS THE TEST LIQUID

		Weight change, g			Water trans-mission, g/m2d	EC outer water[b], µmho
Polymer[a]	Exposure time, d	Empty pouch	Filled pouch	Pouch contents		
CPE 86	316	3.60	4.02	0.42	0.291	86
CSPE 6R	1151	6.97	10.23	3.26	0.069	585
CSPE 55	315	5.35	4.70	-0.65	-0.002	66
ELPO 36	1151	-0.01	2.32	2.31	0.049	34
LDPE 21	315	-0.04	0.85	0.81	0.073	31
PB 98	316	0.14	0.65	0.51	0.043	27
PVC 17	310	-0.40	6.45	6.85	0.619	371
PVC 19	315	-0.60	1.77	2.37	0.214	16
PVC 59	1115	0.19	11.00	10.81	0.238	17

[a]Number = Matrecon serial number; R = fabric-reinforced.
[b]Electrical conductivity of outer water before it was replaced at 1169 days.

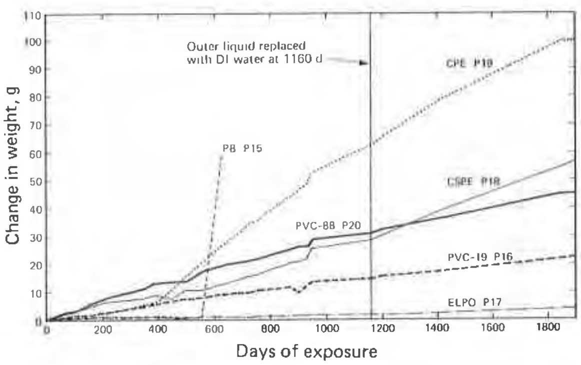

Figure 5. Weight changes of pouches containing a highly acidic waste.

TABLE 6. POUCH TEST OF MEMBRANES WITH A HIGHLY ACIDIC WASTE AS TEST LIQUID

		Weight change, g			Water trans-mission, g/m^2d	EC outer water[b], µmho
Polymer[a]	Exposure time, d	Empty pouch	Filled pouch	Pouch contents		
CPE 86	1887	14.28	100.00	98.70	1.213	14,600
CSPE 85	1887	28.26	58.87	46.57	0.510	2,500
ELPO 36	1887	1.03	3.58	1.40	0.017	92
PVC 19	1887	1.94	22.50	21.03	0.252	4,800
PVC 88	1887	11.46	45.43	35.84	0.363	9,000

[a]Number = Matrecon serial number; R = fabric-reinforced.
[b]Electrical conductivity of outer water before it was replaced at 1169 days.

The pouches filled with acetone or xylene and placed in DI water showed the movement of each solvent out of the pouches and no water movement into the pouches (Figures 7 and 8). The pouches with 50:50 acetone:DI water showed outward movement of acetone when immersed in DI water and inward movement of acetone when immersed in acetone (Figure 9).

Permeability to Ions

In the experiments performed with salt solutions, the ions showed little tendency to permeate polymeric sheetings. Even at high concentration gradients (10% LiCl solution), the electrical conductivity of the outer liquids remained low. In the tests with the highly acidic waste, the hydrogen ions in the pouches appeared to have migrated through the walls; the pH of the outer solution dropped and the electrical conductivity increased (Table 6).

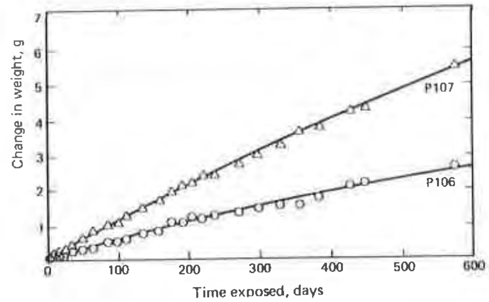

Figure 6. Weight changes of PVC pouches containing 5 (P106) and 10% (P107) aqueous solutions of LiCl during immersion in DI water.

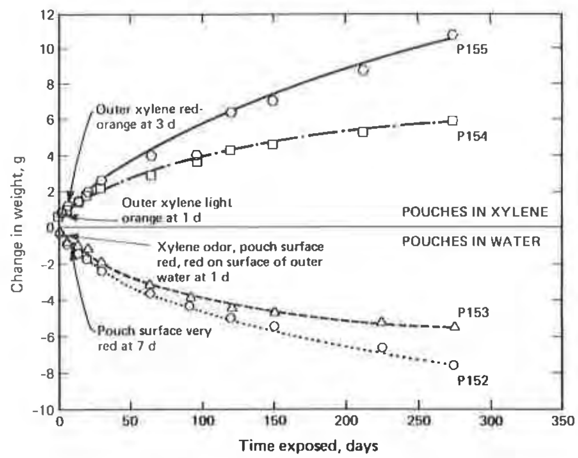

Figure 7. Weight changes in HDPE-A pouches filled with xylene and 1% Automate Red.

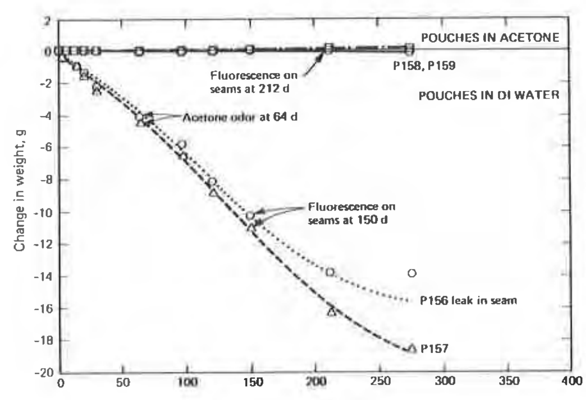

Figure 8. Weight changes in HDPE-A pouches filled with acetone and 1% Fluorescent Yellow.

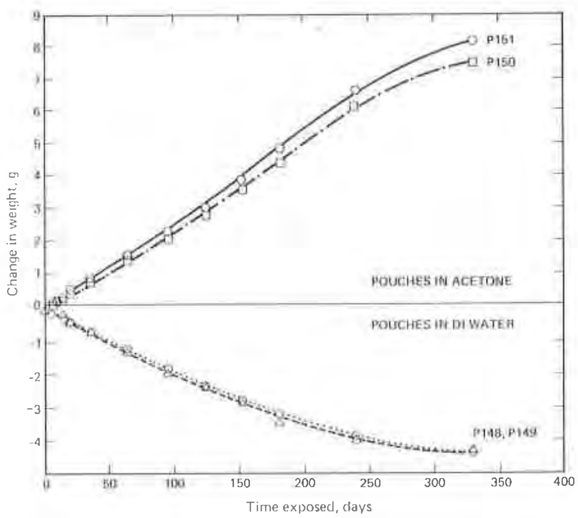

Figure 9. Weight changes in HDPE-A pouches with 50:50 acetone:DI water and 1% methyl violet.

Permeability to Organics

Both acetone and xylene permeated the walls of the pouches (Figures 8 and 9) when the pouches were placed in DI water. The acetone permeated the walls and dissolved in the outer water; the xylene permeated but, because it is not soluble in water, rose to the surface. When the pouches with the acetone and xylene were placed in the same solvents, the movement was into the pouch where the solvent contained dissolved constituents, either organic dyes or water (Figures 8-9). Pouches with waste liquids that were oily developed a film of oil on their outside surfaces.

Permeability to Organic Dyes

Three water-soluble and two organic-soluble dyes were studied in pouch experiments at 1% concentrations. The water-soluble types included sodium fluorescein, Sevron Red, and methyl violet. The sodium fluorescein showed positive transmission through PVC-OR, and trace transmission through HDPE, PB, and PVC. Sevron Red in water showed no detectable transmission through HDPE-A and PB. Methyl violet in the 50:50 acetone:water mixture showed no detectable transmission through HDPE-A. In contrast to the water-soluble dyes, the two organic-soluble dyes, Automate Red and Fluorescent Yellow, in pouches containing acetone and xylene quickly permeated the HDPE-A pouch walls (Figure 7). For pouches immersed in DI water, these organic dyes precipitated on the outside as they are insoluble in water.

Comments on the Pouch Procedure

A major feature of the pouch procedure is its ability to assess, over extended periods, the permeability of a liner material simultaneously to several waste constituents in a configuration that simulates some aspects of the membrane liner in service in a disposal facility.

PERMEATION OF ORGANICS THROUGH MEMBRANES IN SOIL ENVIRONMENTS

In actual service a membrane liner is placed on a porous layer (e.g. soil or geotextile), the permeability of which can range greatly. Also, a liner often is covered or becomes covered with soil, sand, silt, or sludge. These materials will tend to retard the transmission of waste constituents, as has been demonstrated in preliminary experiments.

ACKNOWLEDGMENTS

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Impermeability: The Myth and a Rational Approach

Impermeability is a legitimate goal at a time when conservation of resources and prevention of pollution are a priority. This goal is difficult to achieve because all liner materials are permeable. The first part of the paper provides practical information on geomembrane permeability and leakage evaluation. The second part of the paper shows how appropriate design can lead to almost zero leakage through the use of double or triple liners.

"Dear Henry: Thank you for your very pertinent letter on the use of the adjective impermeable... I agree...that the adjective "impermeable" should be given a relative value, not an absolute one... I am well aware of the damages caused by misunderstanding of the word impermeable. I am presently involved in the repair of a reservoir, "guaranteed impermeable" by the installer, where leakage triggered approximately one million dollars in damage." This excerpt of a letter by this paper's author responded to a letter from H. Haxo, Jr., of which excerpts follow.

"Dear Jean-Pierre: I have received a copy of Bulletin No. 1 announcing the International Conference on Geomembranes... I feel that the use...of the word "impermeable" in describing polymeric membranes is perpetuating a basic and serious misconception regarding these lining materials. No membrane, polymeric or otherwise, is completely impermeable or impervious to all gases, vapors, and liquids. All polymeric compositions are permeable in different degrees to different fluids - to gases, vapors, or liquids. The question is how much of a given waste constituent is transmitted through a specific membrane under the conditions of test or service exposure and in which direction. Even though membranes are not impermeable, because of their low permeability to many permeants they can be used effectively to control pollutant migration... The concept that membranes are impermeable is already causing considerable misunderstanding in the liner industry and among regulatory personnel, engineers, designers, users, and owners of waste impoundments... I also suggest that the International Conference on Geomembranes would be an appropriate forum at which to clarify this misconception and place this property of membranes and other liners in its proper perspective."

This exchange of correspondence, in May of 1983, inspired the following paper which is intended to illustrate a key sentence written by H. Haxo, Jr.: "Even though membranes are not impermeable,...they can be used effectively to control pollutant migration." Accordingly, the paper has two parts: in the first part it is shown that all materials are permeable, and in the second part it is shown that appropriate design using geomembranes can ensure a certain level of "impermeability".

1 PERMEABILITY OF MATERIALS

All materials used as liners are permeable to fluids (liquids and gases) and a certain amount of leakage should be expected through liners. Additional leakage results from defects such as cracks, holes and faulty seams.

The first part of this paper is intended to provide designers with practical information regarding evaluation of permeability and leakage.

1.1 Darcy's Equation

The permeability of soils is evaluated using Darcy's equation:

$$v = ki \qquad (1)$$

where: v = fluid velocity (m/s); k = conductivity (also called coefficient of permeability) (m/s); and $i = h/T$ = hydraulic gradient (dimensionless); h = hydraulic head (m); and T = geomembrane thickness (m).

The conductivity k depends on the fluid (and its temperature) and on the soil. When the fluid is water, k is the hydraulic transmissivity. The following relationship exists between conductivities related to two different fluids at different temperatures flowing in a same soil:

$$k_1/k_2 = (\rho_1/\rho_2)\ (\eta_2/\eta_1) \qquad (2)$$

where: ρ_1 and η_1 = density (kg/m³) and dynamic viscosity (kg/(ms)) of fluid 1 at temperature 1; ρ_2 and η_2 = density (kg/m³) and dynamic viscosity (kg/(ms)) of fluid 2 at temperature 2.

Darcy's equation is often called Darcy's law, which is improper. In mechanics of continuous media, a law is a relationship between stresses and strains, derived from tests. Examples of laws are: (i) Hooke's law which is a linear relationship between stresses and strains, describing the linear elastic behavior of a solid; (ii) Newton's law which is a linear relationship between shear stress and rate of strain, describing the linear viscous behavior of a liquid.

Integration of a law for given boundary conditions leads to an equation. Examples are: (i) integration of Hooke's law in a loaded semi-infinite soil mass leads to Boussinesq's equation giving a relationship between load and settlement of the soil, depending on the elastic modulus of the soil; (ii) integration of Newton's law in a pipe leads to Poiseuille's equation giving a relationship between discharge of liquid and geometry of the pipe, depending on viscosity of the liquid.

Darcy's equation is a special case of Poiseuille's equation for a porous medium which is a special case of a tortuous pipe. As Poiseuille's equation, Darcy's equation (as well as Eq. 2 above) is valid only for laminar flow.

The mechanisms by which fluids flow through geomembranes and soils are different. Darcy's equation can be used for geomembranes only if it is made clear that the coefficient of permeability used for the geomembrane depends on the flow conditions, hydraulic gradient or hydraulic head. Similarly, the relationships presented hereafter are applicable only within the regime of laminar flow as well as other limits of validity of Darcy's equation.

1.2 Evaluation of Permeability

Users of geomembranes are confused by the variety of terms used to evaluate permeability. A review of these terms is presented below.

Conductivity (or Coefficient of Permeability), k. Volume, V, of fluid passing per unit area, A, of geomembrane, per unit hydraulic gradient, i, per unit period of time, t.

$$k = V/(Ait) \qquad (3)$$

Dimension: LT^{-1}. Units: m/s; cm/s; perm-inch; perm-mil; "metric" perm-cm. Relationships between units:

1 cm/s = 0.01 m/s
1 perm-inch = 1.67 "metric" perm-cm = 1.42×10^{-11} m/s
1 perm-mil = 10^{-3} perm-inch = 1.42×10^{-14} m/s
1 "metric" perm-cm = 0.598 perm-inch = 598 perm-mil = 8.51×10^{-12} m/s

Permittivity, Ψ. Volume, V, of fluid passing per unit area, A, of geomembrane, per unit hydraulic head, h, per unit period of time, t.

$$\Psi = V/(Aht) \qquad (4)$$

Dimension: T^{-1}. Unit: s^{-1}.

Impedance, I. Time t necessary for a unit volume, V, of fluid to pass through a unit area, A, of geomembrane, per unit hydraulic head, h.

$$I = Aht/V \qquad (5)$$

Dimension: T. Unit: s. While other terms presented in this section are proportional to the quantity of flow, impedance is inversely proportional to the quantity of flow and can be used as a measure of the relative "impermeability".

Permeance, ω. Mass, M, of fluid passing per unit area, A, of geomembrane, per unit pressure, p, per unit period of time, t.

$$\omega = M/(Apt) \qquad (6)$$

Dimension: TL^{-1}. Units: kg/(m^2.Pa.s); perm = grain/(ft^2. inch Hg. hour); "metric" perm = g/(m^2.mmHg. 24 hours). Relationships between units:

1 perm = 5.72×10^{-11} kg/(m^2. Pa. s)
1 perm = 0.659 "metric" perm
1 "metric" perm = 1.517 perm

Water Vapor Transmission Rate, WVT. Mass, M of water vapor passing per unit area, A, of geomembrane, per unit period of time (under a specified pressure which may vary from one test to another).

$$WVT = M/(At) \qquad (7)$$

Dimension: $ML^{-2}\ T^{-1}$. Units: kg/(m^2.s); g/(m^2. 24 hours); grain/(ft^2. hour). Relationships between units:

1kg/(m^2.s) = 8.64×10^7 g/(m^2.24h)
1g/(m^2.24h) = 0.0598 grain/(ft^2.hour)
1kg/(m^2.s) = 5.17×10^6 grain/(ft^2.hour)

Relationships Between Various Terms. When Darcy's equation is applicable, the following relationships exist between the terms defined above:

$$k = \Psi T \qquad (8)$$
$$I = 1/\Psi = T/k \qquad (9)$$
$$\omega = \Psi/g \qquad (10)$$
$$WVT = p\,\omega = p\Psi/g \qquad (11)$$

where : T = geomembrane thickness; g = gravity (9.81 m/s^2); p = vapor pressure in the vapor transmissivity test.

The relationship between conductivity on one hand, and permittivity and impedance on the other hand, depends on the thickness of the geomembrane. If two geomembranes are made from the same polymeric compound, they have the same conductivity, k . Their impedances are proportional to their thickness; and their permittivities are inversely proportional to their thickness (provided Darcy's equation is applicable).

Since gravity has a fixed value, relationships can be established between the units of permeance and permittivity, using Eq. 10:

1 kg/(m^2.Pa.s) = g s^{-1} = 9.81 s^{-1}
1 perm = $9.81 \times 5.72 \times 10^{-11} s^{-1}$
= $5.61 \times 10^{-10} s^{-1}$
1 "metric perm" = $8.51 \times 10^{-10} s^{-1}$

Eq. 11 depends on the pressure which may vary from one test to another. Therefore it is impossible to establish a relationship between units of WVT, on one hand, and permittivity and permeance, on the other hand.

To deduce the permittivity or the permeance of a sample from a WVT measurement, it is necessary to know the vapor pressure in the WVT test. Sometimes, only the temperature and the relative humidity of the vapor are given. The vapor pressure can then be deduced using the following formula:

$$p = H \times p_s \qquad (12)$$

where: p_s = pressure of saturated vapor at a given temperature (which can be found in physics tables); and H = relative humidity.

Example. A WVT of 15.6 g/(m^2.24h) has been measured at a temperature of 24°C and a relative humidity of 55%. According to physics tables:

$$p_s = 22.377 \text{ mmHg} = 22.377 \times 133.3 = 2983 \text{ Pa}$$

The vapor pressure is:

$$p = 2983 \times 0.55 = 1641 \text{ Pa}$$

The permeance of the sample is:

$$\omega = (15.6/8.64 \times 10^7)/1641 = 1.1 \times 10^{-10} \text{ kg/(m}^2\text{.Pa.s)}$$

1.3 Permeability of Geomembranes

Tests were conducted 1973 to 1977 under the responsibility of the author at the University of Grenoble, France, to evaluate the permeability of geomembranes.

Influence of Pressure. The tests were conducted with deaired water subjected to pressures ranging from 50 to 1000 kPa. Results are presented in Fig. 1. Also presented in Fig. 1 are values of conductivity of geomembranes derived using Eqs. 8 and 11 from values of water vapor transmission found in various chemistry manuals and manufacturer's brochures. These values were usually measured according to ASTM E-96, i.e. with pressures typically less than 10 kPa. Values obtained with water and values obtained with vapor are of the same order of magnitude, and the ranking of Hypalon, PVC and Butyl rubber is the same.

Results of tests obtained with water show that the coefficient of permeability of Butyl rubber exhibits a peak for a pressure of 250 kPa and then decreases with increasing pressures. All other geomembranes tested, polymeric and asphaltic, have a coefficient of permeability decreasing with increasing pressure. If Darcy's equation were applicable to geomembranes, coefficients of permeability would be independent of pressure. Nonetheless, Darcy's equation can be used with certain precautions as explained in section 1.4.

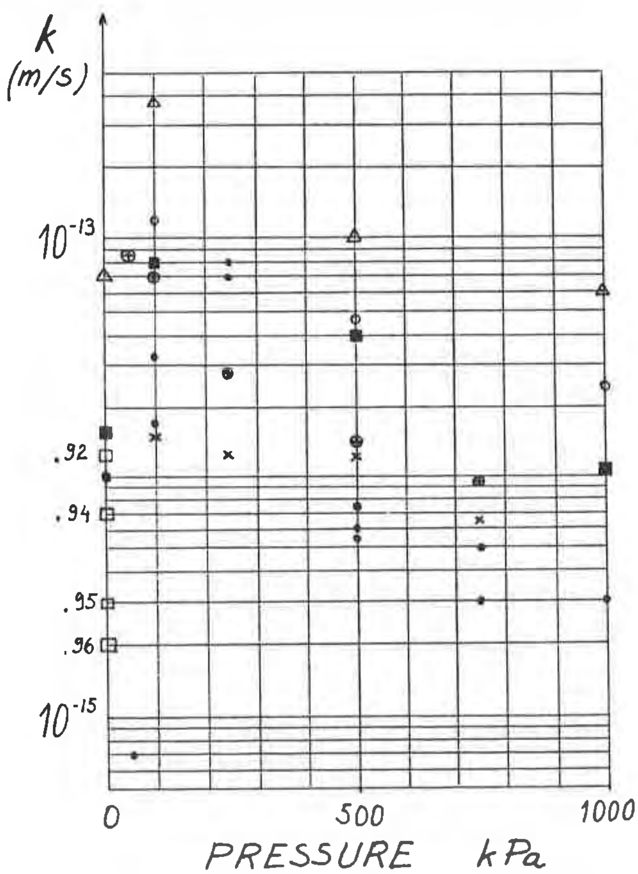

Fig. 1 Coefficient of permeability to water (hydraulic conductivity) of various geomembranes. Values shown on the vertical axis have been deduced from water vapor transmission tests. Other values were obtained with deaired water. Legend: ● Butyl rubber; O EPDM; △ CSPE (Hypalon); □ Polyethylene (value of density is indicated); ■ PVC; ⊕ and × asphaltic geomembranes.

Influence of Geomembrane Elongation. Tests were conducted with deaired water on Butyl rubber samples subjected to the following elongations in both directions: 0%, 4.9%, 9.5% and 22.5% resulting in area increase of 0%, 10%, 20% and 50%, respectively. Test results are presented in Fig. 2. It appears that the elongation, although it affects the shape of the curves, does not significantly affect the order of magnitude of the coefficient of permeability. Additionally, a few tests were conducted with an 80% area increase. Results of these tests were not significantly different from the 50% area increase results.

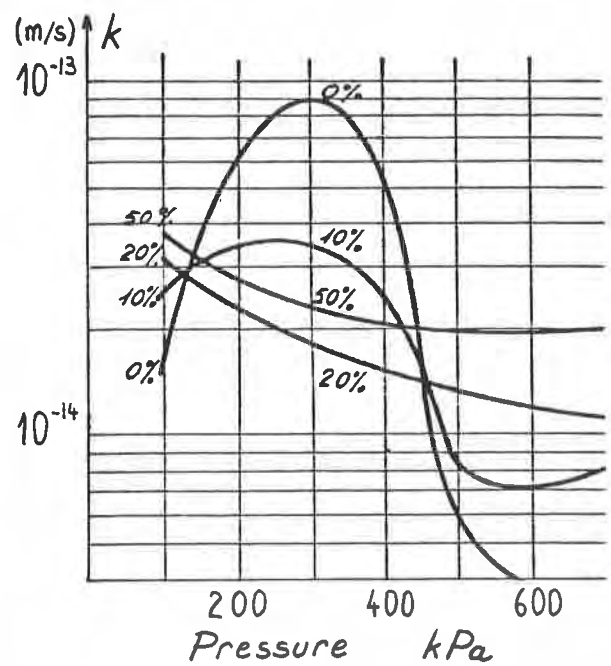

Fig. 2 Coefficient of permeability of a Butyl rubber geomembrane subjected to elongation. (Percentages are increases in area of the Butyl rubber geomembrane)

1.4 Leakage Evaluation

Leakage is due to permeability of the geomembrane and holes through the geomembrane.

Leakage due to permeability can be evaluated approximately using Darcy's equation (see section 1.1) if the coefficient of permeability has been measured on a geomembrane sample of the same thickness which has been subjected to the same fluid pressure as the considered geomembrane.

To evaluate leakage due to holes, two cases should be considered: pinholes and large holes. Pinholes can be defined as holes having a dimension (e.g., diameter if circular) significantly smaller than geomembrane thickness. For leakage calculation purpose, such a hole can be considered as a pipe and Poiseuille's equation can be used. Large holes are holes having a dimension (e.g., diameter if circular) significantly larger than geomembrane thickness. The classical Bernouilli's equation for flow through an aperture can therefore be used. In conclusion, the following three equations are useful for leakage evaluation:

. Darcy's equation (geomembrane permeability):

$$Q/A = k\ (z/T) \tag{13}$$

. Poiseuille's equation (pinholes):

$$Q = \pi \rho\, g\, z\, d^4/(128\, \eta\, T) \tag{14}$$

. Bernouilli's equation (large holes):

$$Q = Ca\sqrt{2gz} \tag{15}$$

where: Q = discharge (m^3/s); Q/A = discharge per unit area (m/s); z = depth below water level (m); k and T = coefficient of permeability (m/s) and thickness (m) of the geomembrane; d = pinhole diameter (m); a = hole surface area (m^2); ρ and η = density (kg/m^3) and dynamic viscosity (kg/m.s) of water; and g = gravity (9.81 m/s^2). C is a dimensionless coefficient, valid for any Newtonian fluid, related to the shape of the edges of the aperture; for sharp edges, C = 0.6.

By integration along bottom and slope of a geomembrane-lined reservoir, Eqs. 13, 14 and 15 lead to the following equation giving leakage from the entire reservoir:

$$Q = (H/T)\ [k + \pi\rho\ gd^4 n\_(128\eta)]\ (S + S'/2) + 0.85\ a\ N\sqrt{gH}\ (S + 2S'/3) \tag{16}$$

where: H = height of liquid in reservoir (m); n = number of pinholes per unit area (m^{-2}); N = number of holes per unit area (m^{-2}); S = reservoir bottom surface area; and S' = reservoir slope surface area.

Integrations leading to Eq. 16 have been made with the assumption that slopes are rectangles which is true only for reservoirs with vertical slopes and for canals. However, Eq. 16 is an excellent approximation when bottom area is equal to or larger than slope areas.

An example of the use of Eq. 16 is presented in (4). This example shows that leakage due to geomembrane permeability is negligible compared to leakage through holes. Leakage due to pinholes is also negligible, unless there is an unusually large number of pinholes.

The last term of Eq. 16, which is related to leakage through holes, is valid only if the medium located underneath the geomembrane is free draining. The discharge through holes would be much smaller if the medium located under the geomembrane became saturated with the leaking liquid. This case is discussed in (1, 2, 3).

Leakage through a geomembrane with a hole is drastically reduced if the geomembrane is placed on a low permeability soil such as clay. Composite liners made of a geomembrane placed on a layer of compacted clay are sometimes used to decrease the risk of leakage. Such composite liners are effective only if geomembrane and clay are in close contact over their entire surface. This requirement is difficult to fulfill due to geomembrane wrinkles and clay surface irregularities and cracks. As a result, a leak in the geomembrane, instead of being stopped by the clay, may travel a long way between the two liners, toward the lowest spot of the lined facility, usually the sump area, where a water content increase softening the clay is less than desirable. In conclusion, the use of composite liners can be detrimental as well as beneficial.

Also, a typical mistake with composite geomembrane - clay liners consists of placing a geotextile under the geomembrane as a protection against puncturing by stones. Such a geotextile would facilitate flow of liquid between the two liners.

A composite geomembrane-clay liner is by no means a double liner. As discussed in the next section, a double liner includes a drainage layer.

2 IMPERMEABILITY THROUGH DESIGN

2.1 Parameters Governing Leakage

The test results presented in section 1.3 show that all geomembranes are permeable. In addition, experience shows that there are always holes in geomembranes caused by accidents or improper installation. However, leakage must be prevented in all cases where leaking liquids would pollute ground water, deteriorate the ground supporting the liner (thus provoking a major failure of the liner), or cause unacceptable economic loss (5).

As shown in the previous section, leakage potential is mostly through holes in the geomembrane. Eqs. 15 and 16 show that the two parameters governing leakage through holes are number and size of holes, and head of liquid over the geomembrane. Therefore, to minimize leakage, the number and the size of holes should be minimized and the head of liquid on the geomembrane should be as small as possible.

The number and size of holes can be minimized by appropriate quality control of geomembrane installation and by minimizing stresses likely to damage the geomembrane through careful design. The head of liquid on the geomembrane can be kept small as a result of adequate conceptual design as discussed in the following two sections, i.e., the head can be minimized through the use of double or triple liners.

2.2 Double Liner

Description. The double liner concept has been presented in an earlier paper (6). A double liner consists of two liners with an intermediate drainage layer connected to a sump from where the collected leakage is eliminated by gravity or, more generally, by pumping. If the drainage system is properly designed for the amount of liquid seeping through the upper liner, the head of liquid on the lower liner should be small at any time and leakage through the secondary liner should be small, unless the lower liner has many holes. In addition to leakage prevention, another function performed by a double liner is leakage detection.

No portion of the lower liner of a double liner system should be horizontal. A slope is necessary to prevent accumulation on the lower liner of liquid leaking through the upper liner. Theoretically, the lower liner of a double liner does not need to be waterproof. Elements of geomembranes overlapped as shingles on a roof should be sufficient. In fact,it is recommended to seam the lower liner as carefully as the upper liner: (i) to prevent or, at least, minimize leakage through the lower liner if the drainage layer becomes saturated; and (ii)

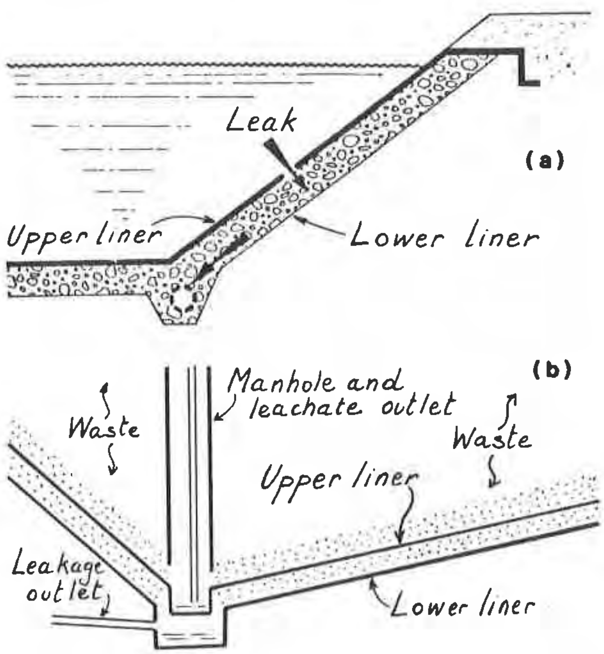

Fig. 3 Examples of double liners: (a) reservoir - original drawing published in 1973 (6); (b) solid waste disposal facility

also because even in the case of free flow, the head is never zero.

It should be emphasized that "double liner" does not mean merely putting two liners together. Leakage control through the use of double liners results from the combination of waterproofing and drainage.

Examples of Double Liners. Double liners are used in all types of liquid containments (reservoirs also called "liquid impoundments", dams and canals) and in solid waste disposal facilities (also called landfills) (Fig. 3). A double liner case history is presented in (4).

The upper liner in a solid waste disposal facility functions like the lower liner in a double liner system. If the drainage layer placed on the liner to collect leachate is properly designed and if the sump and the pump are properly operated, leachate should not accumulate on the liner and leakage should be small. Maximum head in drainage layers mentioned in USEPA (U.S. Environmental Protection Agency) recommendations is 0.3 m (1 ft.). However, cases where large quantities of leachate have accumulated in the bottom of landfills are not rare. Therefore, the upper liner of a landfill's double liner should be designed as if it were the upper liner of a liquid impoundment.

Problems with Double Liners. Ideally, a slope of the order of 5% should be selected at the bottom of a reservoir to ensure effectiveness of drainage between the two liners. Such a slope would significantly decrease the capacity of a reservoir or a landfill, and slopes of the order of 2% are usually selected. Such slopes require a large amount of care during earthwork. Often the slope is significantly less than 2% and ponding of liquid may occur in the drainage layer.

Geomembrane wrinkles, especially in the case of stiff geomembranes, may constitute a significant obstacle to drainage (7). The use of stiff wrinkle-prone geomembranes should be avoided for liners located under drainage layers. If for some reason a stiff wrinkle-prone geomembrane is used, precautions should be taken to minimize the development of wrinkles during and after installation.

A delicate part of the lower liners is their connection with sumps or other outlet structures. An example of the complexity of such structures is given in (4).

Conclusions on Double Liners. The principle of a double liner is to place a drainage layer on the geomembrane through which leakage is to be prevented. Drainage design should be such that the head on the liner is as small as possible. However, the head is never zero in a drain, so therefore a double liner minimizes but does not prevent leakage.

2.3 Triple Liner

As discussed above, a double liner does not offer a total guarantee of leakage prevention. When a higher level of impermeability is required, a triple liner can be used. Two cases can be considered.

Liquid Impoundment. A triple liner designed to prevent loss of liquid is shown in Fig. 4a. Such a triple liner was contemplated for the Proton Decay Experiment (4) once it was realized that a double liner did not offer a total guarantee. The triple liner solution was not selected when it was determined that liquid loss was not absolutely critical.

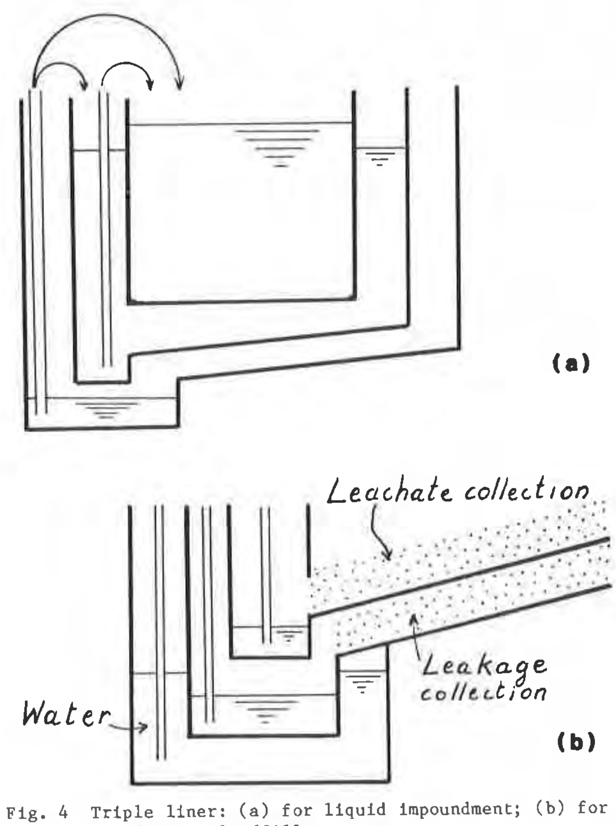

Fig. 4 Triple liner: (a) for liquid impoundment; (b) for solid waste landfill.

The principle is as follows. The space between the upper liner and the intermediate liner is filled with a porous medium containing the same liquid as in the upper liner, or a liquid having the same density. Leakage through the upper liner is almost zero because the pressure difference between both sides of this liner is kept very small. Leakage is expected to occur through the intermediary liner and then be collected by the drainage layer placed over the lower liner. Collected liquids can be recycled as shown in Fig. 4a. A refinement would consist of adjusting the compressibility of the porous medium located between the upper liner and the intermediary liner so that if the liquid level tends to move down in this medium, the effective stress increase compresses this medium which raises the liquid level.

A secondary feature of this type of triple liner is that the upper geomembrane is subjected to very small stresses which minimizes the risk of damage to the geomembrane but facilitates movements of the geomembrane. Therefore ballasting is necessary at the bottom, and some device to restrain movements may be useful on the sides.

Solid Waste Disposal Facility. A triple liner designed to prevent leakage of a hazardous liquid from a landfill sump is presented in Fig. 4b. The same systems could be used for the whole landfill.

The principle is as follows. Water is maintained around the sump at a higher pressure than the hazardous liquid. If leakage is governed only by pressure difference the hazardous liquid cannot leak out of the sump. In addition, in most circumstances, osmotic gradient should be from the water to the hazardous liquid. However, further studies should be done to evaluate if osmotic pressure may cause some components of the hazardous liquid to permeate through the geomembrane. Such a phenomenon would be detected by chemical analysis of the water, and if necessary, the water could be pumped out and replaced before significant leakage occurs through the lower liner.

The sump must be filled with heavy porous material such as gravel or must have structural resistance not to be uplifted by water pressure.

Conclusion on Triple Liners. Triple liners have not been used in actual projects to the author's knowledge. It is suggested that research be undertaken to assess the applicability of this concept. The feasibility of prefabricated triple liner sumps should also be investigated.

CONCLUSION

Impermeability is both a myth and a legitimate goal.

Impermeability is a myth because all liner materials are permeable. This myth is propagated by those who take advantage of the lack of information of many users and designers. This myth is also propagated by official bodies using the term impermeable to designate committees and conferences.

Impermeability is a legitimate goal at a time when conservation of resources and prevention of pollution are a priority. Although liner materials are not absolutely impermeable, impermeable containment or, more accurately "zero leakage" containment can be achieved with appropriate design. In fact, design of liquid or solid containments should always be done with the assumption that some leakage will occur through liner materials, because even if a liner is impermeable at the completion of installation, it may leak after some time.

The combination of adequate design and proper selection of materials is the only way to ensure impermeability. In fact, everyday life teaches us not to rely solely on material properties. Flying is a convenient way to travel. If we relied only on material properties to fly, we would be inclined to use balloons because their flying capability is based essentially on material property, (that is, their density is less than that of air). The fact is, however, that we use airplanes which fly primarily as a result of design. Of course, material selection is important in airplane construction. The materials used should have appropriate mechanical properties and be as light as possible, just as geomembranes should have appropriate mechanical properties and be as impermeable as possible.

It is tempting to pursue the comparison between geomembranes and airplanes. The remarkable reliability of the airline industry results essentially from the amount of care involved in the detailed design of airplanes, and the quality control of construction, maintenance and operation of airplanes. The liner industry faces today a challenge similar to the challenge faced years ago by the airline industry when it was plagued by an unacceptable number of accidents.

Adequate conceptual design, proper selection of materials, careful detailed design and quality control during construction, maintenance and operation of geomembrane-lined facilities: these are the necessary steps towards impermeability.

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Permeabilities of Commercially Available Polymeric Liners for Hazardous Landfill Leachate Organic Constituents

A measuring system is presented which is able to determine permeation rates of organic solvents, mixtures and dilute aqueous solution of them in contact with polymeric liner materials. Data of permeation rates of different organic agents and municipal leachate permeating through commercial liner materials (HDPE, ECB, CPE, PVC, EPDM, thickness 0,8 mm to 2,7 mm) have been presented. Similar HDPE liner materials with only small differences in crystallinity show a great amount of differences in permeation rates. - The qualitative composition of the permeated liquids are strongly dependent upon the chemical constitution of the liquid organic compound and of the polymeric material (selective permeation).- Comparable immersion tests show that changes in weight and dimensions are significant for PVC-liners.

Large plastic sheets have been successfully used for many years as a sealing layer in roadbuilding projects, tunnel construction and canal construction. Depending on the condition of perfect manufacture and careful installation, these films are "moisture-tight". Permeation of fluid by pressure dependent flow processes, which are typical features of clay liners, do not play an essential part to plastic liners. Polymeric-based synthetic liners are by nature nonporos and free of cracks and fissures. This assumption (1) is at least correct for plastic films with a thickness of 0.8 mm and more. Because of these favorable properties, it is state of the art to use polymeric membrane liners as leachate barriers in landfills, to prevent migration into soils, thereby preventing contamination of groundwater.

However, low or even zero permeation rates of water do not mean that there is no transport of hazardous and toxic compounds, which may be constituents of municipal and industry based waste. Instead of flow processes there are other physical phenomena like absorption, diffusion and desorption of liquid media, which make their transport through the polymeric material possible. All polymeric liner materials will allow many chemicals, first of all organic compounds, to permeate into and to migrate through them. - The intensity of absorption or solubility depends above all on the chemical affinity of the liquid media to polymeric material. Beside the solubility, the mobility of the permeating molecules is of great importance. Mobility means the ability of the molecules to change the place (theory of free volume (2)). Finally the molecules have to leave the polymeric membrane and must be picked up by the soil beneath the landfill.

Though these processes are of physical nature, provided the polymeric materials are resistant to the particular liquid media, this kind of transport is often called "Chemical Permeation". Therefore, this term will be used in the following.

Chemical permeation, applied to the use of polymeric bottom liners for domestic and industrial refuse dumps means that residual permeabilities must be expected for the so-called percolating water or leachate, which occurs at the base of landfills.

In connection to this, this paper reports about a recently completed research project (3), (4), carried out on the instruction and supported by the Federal Environmental Office (UBA) on commercial plastic sheeting, examining permeation rates, i. e. the substance permeabilities (measured in g or ml) relative to the surface area (m^2) of the sealing material and the reaction time. In addition to percolating water of municipal landfill, organic solvents and aqueous solutions were used as test liquids.

Table 1. Selected permeation substances

Permeation substances	Density ρ g.cm^{-3}	Concentration in aqueous solutions weight %
Toluene	0.87	0.05
Xylene	0.87	0.02
Acetone	0.79	5
Methanol	0.79	5
Acetic acid	1.05	5
Acetic acid ethylester	0.90	5
i-Octane	0.69	0.001
Carbon tetrachloride	1.59	0.08
Chloroform	1.49	0.08
Trichloroethylene	1.47	0.1
Tetrachloroethylene	1.62	0.015
Chlorobenzene	1.11	0.05
Tetrahydrofuran	0.89	-

GOAL OF THE EXAMINATION

The goal of this research was to provide fundamental data which will make it possible to estimate the long-term pollution of the soil and groundwater below dumps.

SELECTION OF PERMEATING AGENTS

It would not be sensible to include leachate exclusively in the studies because there actually is no representative percolating water in the sense of a standardized test fluid. As a further complication, an arbitrarily selected random sample is subject to constant changes due to ongoing unaerobic and aerobic transformations. Essential selection criteria were:

The presence of these substances on the landfill and as a component of leachate must be expected.

Actual danger which the substances represent for the groundwater (5), (6).

Type and amounts of solvents produced and used on an annual basis and their residues (e. g. toluene, xylene in the paint industry), (7).

Influence of these solvents on the plastic sealing sheeting.

In order to get some basic information concerning the permeation behaviour of substances and commercial polymeric membranes, some important classes of substances (esters, ketones, alcohols etc.) had to be tested.

Although the concentrated occurence of these media along the base layer must be considered an exception, permeation rates were also determined for concentrated solutions, provided the stability of the lining material itself allowed this. Only polyethylene liner (HDPE) could be used for this purpose.

SELECTION OF POLYMERIC MEMBRANES

In order to get practice-oriented results only commercial available polymeric bottom liner materials were tested. The materials were selected according to prior agreement with the supporting UBA and representatives of german membrane liner industry, first of all liner fabricators. - Proceeding in this way was necessary to regard all characteristic parameters of the materials (e. g. plasticizer, filler, stabilizer etc.) which might be essential for the permeation behaviour without having to determine them in each individual case. The result of the selection shows table 2: high density polyethylene (HDPE - two sheeting processors); plasticized polyvinylchloride (PVC-p - four sheeting processors); chlorinated polyethylene (CPE - one sheeting processor); ethylene copolymer with bitumen (ECB - one sheeting processor); ethylene-propylene-diene-terpolymers (EPDM - one sheeting processor).

Table 2. Selected polymeric membranes

Liner-material	Thickness of liner mm	Remark
HDPE 1/1.5/2.5	1/1.5/2.5	black surface shiny
HDPE 2/2.7	2/2.7	black topside: smooth backside: rough
$PVC-p_{0.8}$	0.8	dark gray
$PVC-p_{1.0}$	1.0	light-gray/dark gray fabric-reinforced
$PVC-p_{1.5}$	1.5	beige
$PVC-p_{2.0}$	2.0	yellow, doubled
$CPE_{1.5}$	1.5	light-gray/black
$PVC-p/CPE_{1.5}$	1.5	light-gray, doubled
$CPE/PE_{1.5}$	1.5	light-gray/black composed: CPE/PE/CPE
$EPDM_{1.8}$	1.8	black
$ECB_{2.6}$	2.6	black knopped surfaces

EXPERIMENTAL

The experimental procedure used to study permeation rates had to guarantee that multi-substance systems such as mixtures of organic solvents or aqueous solutions of organic solvents could be examined quantitatively and qualitatively in addition to single substance systems.

A further requirement placed on the performance of the measurement apparatus was that the precisely reproducible peripheral conditions were to be selected in the sense of simple known solutions to Fick's Laws of diffusion.

This condition was met by an experimental arrangement which guaranteed a constant concentration C on one side of the liner, while a concentration of C = 0 was guaranteed on the other side. (Fig. 1)

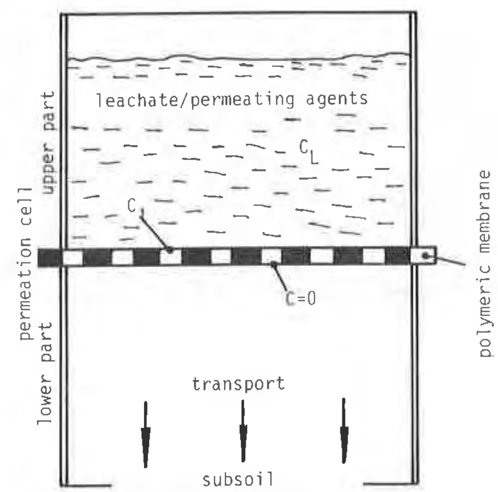

Fig. 1. Prinzip of permeation cell

Applied to the actual practice of sealing the dump base, C = 0 means that any substance which occurs on the underside of the film will again be removed from it and transported into the more or less moist soil. The peripheral condition C = 0 is practice orientated as long as the transport processes in the foil and not those in the soil beneath the dumping ground determine the speed of the total process.

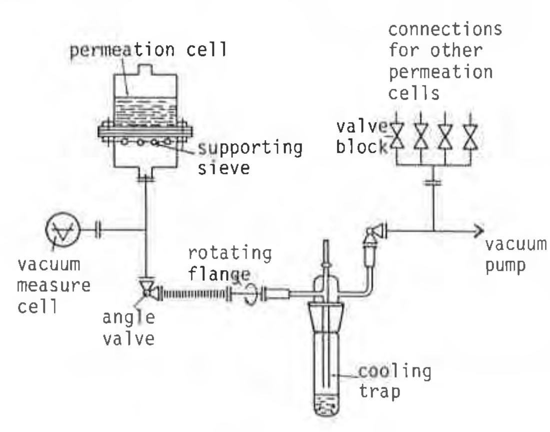

Fig. 2 Permeation measurement arrangement

According to our experience C = 0 is justified for wet soil. In all other cases the permeation rates measured with C = 0 might be regarded as a maximum estimation of permeability.

The experimental condition C = 0 can be achieved experimentally without difficulties, either by passing an inert gas along the back side of the liner or applying a moderate vacuum (10^{-2} mbar to 10^{-1} mbar). The only function of both procedures is to remove the permeated liquid from the back side of the polymeric liner.

Both measurement methods will result in the same permeation rates and can basically be considered equilvalent. However, in the case of the vacuum method, it should be noted that the permeation rates must be corrected with a factor since the support of the vacuum side of the film by a sieve which is necessary due to the underpressure which reduces the effective liner surface.

Long induction times (start up phases) were expected due to the considerable thickness of the films (0.8 mm to 2.7 mm). The induction time is that period which elapses prior to the adjustment of constant conditions (steady state). As a result, a discontinuous measurement method was selected. The permeation fluids which were removed were frozen in a cooling trap (N_2 liquid) and could then be determined quantitatively at any given time using gas chromatography (GC). Fig. 2.

RESEARCH RESULTS

Permeating Liquid: Leachate

Leachate permeation of a municipal landfill through different polymer liners was tested. The experiments lasted approximately half a year. The percolating water was stabilized with sodium azide.

The results of these tests are shown in Fig. 3 . Chromatograms, determined by headspace gas chromatography, for percolating water before, Fig. 3a), and after permeation, Fig. 3 b) to e), through different liners are compared to one another. Corresponding peaks are identified by A/A', B/B' etc. Comparing qualitatively chromatogram a) with chromatogram b) etc. to e) one can see:
some peaks vanished or attenuated, some peaks intensified after permeation.

As a result of selective permeation the composition of leachate has changed.

Comparing the chromatograms a) to e) shows:
the nature of selective permeation strongly depends on the type of polymer liner.
The impulse rates are a measure of the overall quantities of hydrocarbons. These rates are related on the same amount of undiluted liquids before permeation (leachate) and after permeation. - Water contents of permeated liquids were tested with help of the Karl Fischer method (8). These show that no essential amount of water got through the polymer membranes. Because of selective permeation the liquids consist of nearly 100 % hydrocarbons.
These results made a rough quantitative estimation of overall permeation rates of hydrocarbons through the different polymer liners possible. In case of $HDPE_{1.5}$ liners the estimation led to 0.5 - 1.6 ml · $m^{-2} \cdot d^{-1}$ and in case of $ECB_{2.6}$-liners to 1 - 3 ml · $m^{-2} \cdot d^{-1}$.

An exact analysis of the permeated hydrocarbons was impossible within the scope of the research project.

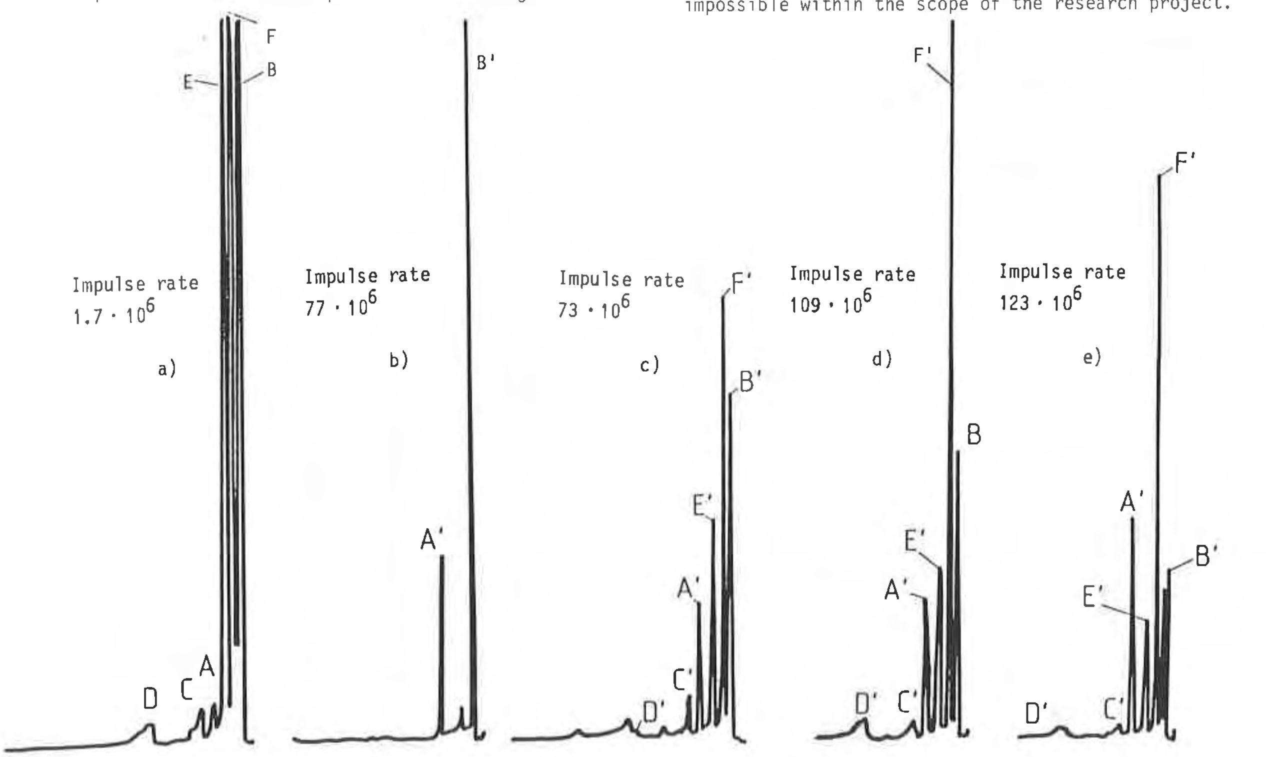

Fig. 3. Gas chromatograms:

a) Leachate undiluted

b) After permeation through $HDPE_{1.0}$ (attenuation 1 : 4)

c) After permeation through $PVC\text{-}p_{2.0}$ (attenuation 1 : 4)

d) " " " $EPDM_{1.8}$ (" 1 : 4)

e) " " " $ECB_{2.6}$ (" 1 : 8)

Permeating liquids: Concentrated organic solvents

Because of the chemical resistance these investigations could only be carried out with HDPE-liners as mentioned before. Fig. 4. demonstrates how the permeation rates change as a function of the reciprocal liner-thickness L^{-1} and the actual permeating agents.

In the case of tetrachloroethylene the permeation rates are clearly lower than they are for trichloroethylene due to the higher chlorination and the bulkier molecule in the latter. The steric influence on the permeation rates is also easily recognized in the cases of chloroform/carbon tetrachloride and toluene/xylene.
The courses of curves clearly indicate that the permeation rate J_S is reduced more than proportionally as the reciprocal liner thickness decreases. The simple solution to Fick'slaw which states that there should be a proportionality between J_S and L^{-1} does not apply in this case. - Linear extrapolation of permeation rates determined by thin films, to thick liners may lead to incorrect values.

Permeating Liquids: Mixtures of Hydrocarbons

In order to simulate the case of a "barrel filled with remains of solvents", deposited in a landfill against the rules, which becomes leaky sometimes, permeation rates for a mixture consisting of equal parts of volumes of six hydrocarbons (table 3.) were determined.

Strong selective permeability causes very different permeation rates for the components of the mixture. The permeation rates of the individual components and the overall permeation rate are listed in table 3.

Table 3. Permeation rates of a mixture of hydrocarbons

Serial no. n	Hydrocarbon	Permeation rate $g \cdot m^{-2} \cdot d^{-1}$
1	Trichloroethylene	9.4
2	Tetrachloroethylene	8.1
3	Xylene	3
4	i-Octan	0.8
5	Acetone	1.4
6	Methanol	0.7
$\sum_{n=1}^{6}$	Hydrocarbons	23.4

Permeating Liquids: Aqueous solutions of organic solvents

Concentrated hydrocarbons in contact with polymeric bottom liners sound rather unlikely and must be regarded as accident. This is right at least in connection with municipal landfills.

That is why the investigations were extended on diluted hydrocarbons with concentrations you can find in table 1.

The results of these measurements are listed in table 4. It is of interest to compare the permeation rates received from concentrated hydrocarbons with those for diluted hydrocarbons. Fig. 5. shows in the case of $HDPE_{1.0}$ in a log scaled picture the permeation rates of the concentrated liquids (columns unhatched) in contrast to those of diluted solvents (columns hatched). In the latter case the measured values are only lower than the permeation rates of the pure solvents by a factor 1/20 to 1/30. The measurements clearly show that significant

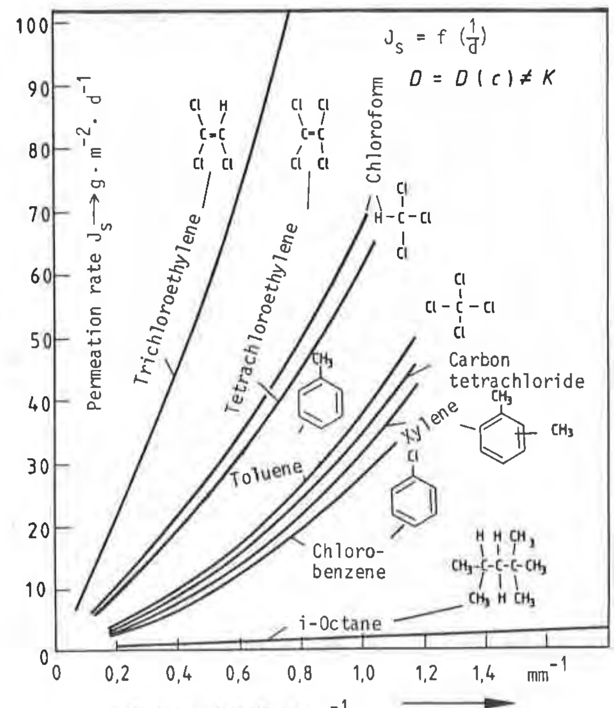

Fig. 4. Permeation rates of hydrocarbons through HDPE-polymeric materials

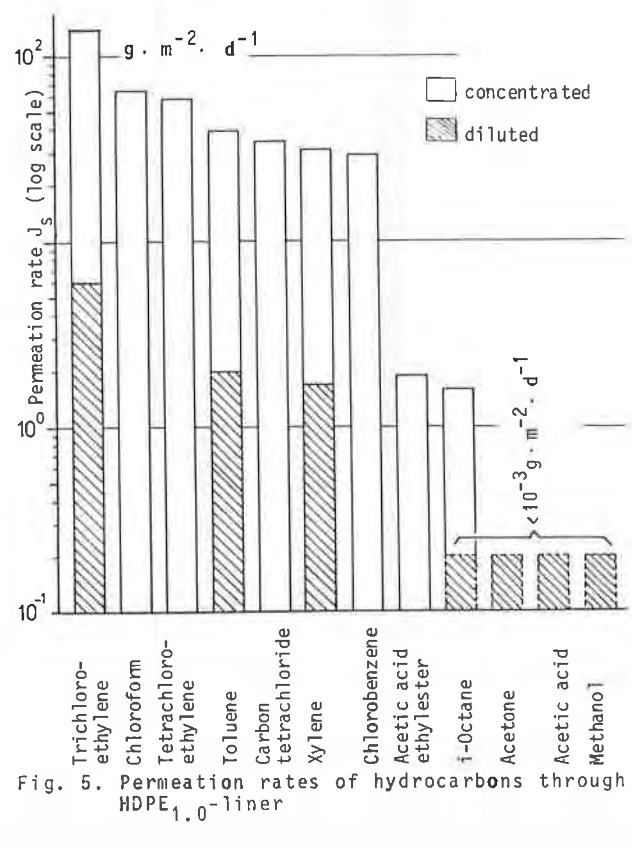

Fig. 5. Permeation rates of hydrocarbons through $HDPE_{1.0}$-liner

quantities of hydrocarbons permeate through the liner materials due to selective permeation, even at very low concentrations in water. As stated shortly before, water practically does not permeate in presence of hydrocarbons. The permeated liquid consists of nearly 100 % solvents.

The following picture, Fig.6 , shows an extreme example for the fact that the various liner materials have very different permeation rates. The ratio of permeation rates of $HDPE_{1.0}$ to $PVC_{1.0}$ to $EPDM_{1.8}$ to $ECB_{2.6}$ to $PEC/PE_{1.5}$ to $PVC_{2.0}$ is equal to 1 : 30 : 6 : 5 : 4 : 1.6.

Further permeation values you can find in table 4 , but we draw your attention to the fact that these values are strictly speaking only valid for those types of liners, we have tested. The application of the measurement values to liner products of other manufactures must be considered in each individual case.

IMMERSION TESTING OF POLYMERIC MEMBRANE LINERS

In the course of the permeation measurements the various lining materials showed very different behaviour in contact with the 5 % and saturated aqueous solutions of hydrocarbons respectively. Some liners stiffened, showed shrinkage during the permeation tests or after drying and discoloured. These modifications were irreversible in contrast to pure swelling as could be observed in case of high-density polyethylene in contact with even concentrated hydrocarbons. Because of these facts immersion tests were carried out testing changes in weight and dimensions after immersion of 540 days and after drying the specimens up to constant weight.

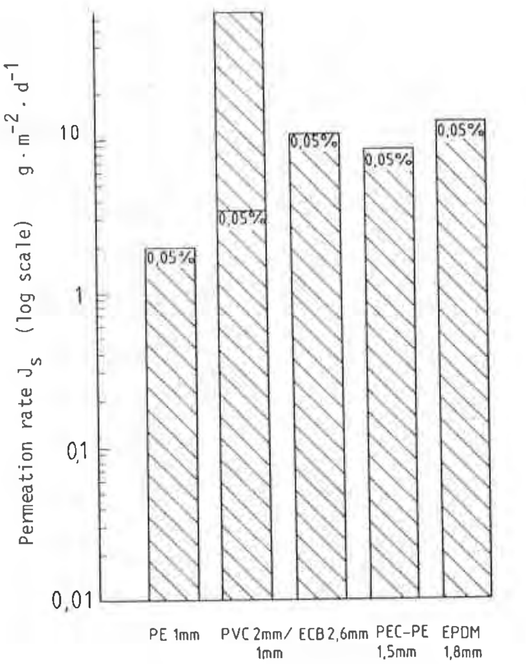

Fig. 6. Permeation rates of aqueous solutions of toluene through various polymeric liners

Table 4. Permeation rates

Solvent	Concentration of solvent weight %	[Permeation rates $g \cdot m^{-2} \cdot d^{-1}$]													
		$HDPE_{1.0}$	$HDPE_{1.5}$	$HDPE_{2.5}$	$HDPE_{2.0}$	$HDPE_{2.7}$	$ECB_{2.6}$	$EPDM_{1.8}$	$PVC\text{-}P_{0.8}$	$PVC\text{-}P_{1.0}$	$PVC\text{-}P_{1.5}$	$PVC\text{-}P_{2.0}$	$PVC\text{-}p/PEC_{1.5}$	$PEC_{1.5}$	$PEC/PE_{1.5}$
Trichloro-ethylene	100 0.1	150 6	86	47	109	81	8							6	
Chloro-form	100 0.08	65					18					53			
Tetrachloro-ethylene	100 0.015	60	38	20	41	22									
Toluene	100 0.05	40 2	19	10	18	13	15		8	17	7	4	5	5	8.6
Carbon tetrachloride	100 0.08	35					2	12				1.4			
Xylene	100 0.02	32 1.6	16	9	12	9	7	3	5	1.4	4.4	2	2.5	2.5	4
Chloro-benzene	100 0.05	29			13										
Acetic acid ethylester	100 5	1.9			1.3		8	60			79	83		23	
i-Octane	100 0.001	1.7 < 0.001		0.4	0.7		0.7					< 0.001			
Acetone	100 5	< 0.001					0.2	2				1.0			
Acetic acid	100 5	< 0.001					<0.001	< 0.001				< 0.1			
Methanol	100 5	< 0.001					<0.001	0.1				<0.1			
Tetrahydro-furan	100 5	24													

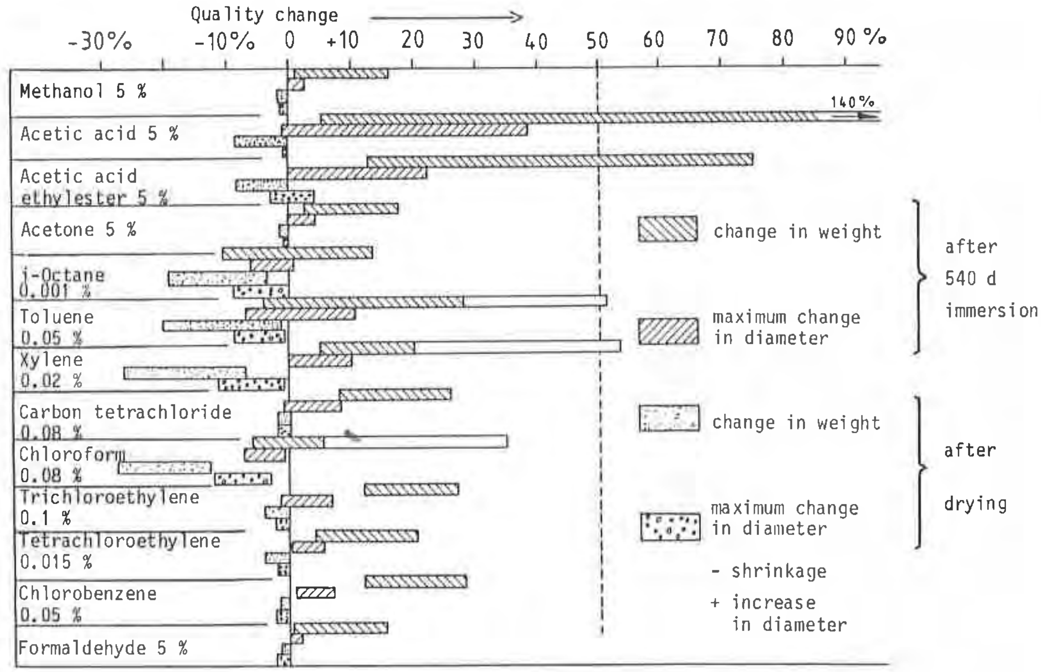

Fig. 7. Immersion testing, four different PVC-P-specimens

The basic procedure of tests was as follows: The specimens, 57 mm Ø, were totally immersed in saturated aqueous solutions and five percent by weight solutions of hydrocarbons respectively, according to table 1. Only HDPE-specimens were immersed in concentrated hydro - carbons. Test temperature, 23 ° C. After an immersion time of 540 days changes in weight and in diameter in machine and in transverse direction were determined. Then the specimens were dried in a vacuum chamber at 50 ° C. By means of tests with nonimmersed specimens, dried in the same way, it was guaranteed that no essential losses of highly volatile polymer additives occured.

In case of HDPE-, CPE-, ECB- and EPDM-liner materials the specimens increased in weight more or less. The maximum changes were: 20 % (HDPE); 30 % (ECB); 43 % (EPDM) and 80 % (CPE). If the specimens gained weight the diameters of the samples increased in general, but the change of dimensions differs mostly between machine and transverse direction. The last one changed more. The maximum changes were: 4 % (HDPE); 6 % (ECB); 15 % (EPDM) and 42 % (CPE). After drying only CPE, ECB and EPDM membranes showed in some cases small losses in weight (1 to 4 %), related to the initial weights of the specimens. The same thing applies to the change of diameter.

Fig. 7 shows the percent change of weight and dimensions after 540 days immersion and drying. The length of columns represents the variation of test results of four PVC-membranes of different suppliers, but all landfill liner-grad materials. These four PVC-membranes exhibit considerable differences in their responses to the different dilute aqueous solutions of hydrocarbons. - The changes in weight during immersion ranged from significant loss, i. e. 11 % to essential gains, i. e. 140 % depending on the liquid. Maximum changes in diameters ranged from shrinkage, i. e. -7 % to an increase in diameter, i. e. 38 %. In some cases the liner materials initially increased in weight and then lost weight (columns without hatches). - After drying the changes in weight were mostly negativ and ranged from -28 % to 2 %, the maximum changes in diameter from -12 % to 4 %.

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Water Repellency Requirements for Geomembranes

Geomembranes are used extensively as moisture barriers for the containment of water and the control of moisture movements in soils. As such,it is essential to establish their relevant property requirements, in particular, those dealing with water repellency. Two water repellency properties govern the selection of a suitable geomembrane - wetting pressure and permeance. Wetting pressure governs the mode of moisture flow through the geomembrane (liquid or vapor) while permeance governs the rate of moisture flow (vapor). These water repellency requirements are examined and design equations presented which allow for controlled migration of vapor through the geomembrane. The practical implications governing the water repellency requirements for geomembranes are also discussed.

INTRODUCTION - BASIC CONCEPTS

The ASTM Draft terminology for a geomembrane describes it as "an essentially impermeable membrane" used in geotechnical engineering structures. As moisture migrates through a soil in two phases - liquid and vapor - it is important to consider just what constitutes a geomembrane - is it 'essentially impermeable' to liquid moisture flows, or is it 'essentially impermeable' to liquid and vapor flows?

The term impermeable normally implies complete repellency (to both liquid and vapor). However, the antonym of impermeable - permeable - normally implies the passage of liquid and vapor. Thus, there is a region in between that of complete liquid and vapor repellency (implied impermeability) and complete liquid and vapor passage (implied permeability) where liquid is repelled and vapor is allowed passage. Lawson(1) termed a membrane which allows the passage of controlled amounts of water vapor but repels liquid water a "controlled-permeability membrane". These controlled-permeability membranes perform the same functions as commonly available geomembranes composed of continuous synthetic films in that they repel liquid moisture; the only difference being, they are designed to allow controlled passage of moisture vapor (even commercially available geomembranes composed of continuous synthetic films allow some moisture migration due to imperfections at the point of manufacture, etc.). A prime example of a controlled-permeability geomembrane is a geotextile which has been partially saturated.

Clearly, there is a need for an improved definition of geomembrane which includes the region of controlled vapor passage as well as complete liquid water repellency.*

In order to select a geomembrane with the required degree of moisture repellency for a particular geotechnical application it is important to consider the basic principles governing moisture flows through geomembranes.

When a small amount of liquid water is placed in contact with a membrane, initially, no liquid water passes through the pores (intermolecular and/or structural) of the membrane. This is because the water in contact with the membrane does not have the ability (pressurewise) to overcome the resistance to flow through the membrane. This resistance to liquid water flow is a function of the liquid water-membrane contact angle and the size and shape of the pores in the membrane. As the external water pressure on the membrane is increased the resistance to liquid water flow is exceeded and liquid water passes through the membrane. The external water pressure at the onset of liquid water passage is called the "wetting pressure" (sometimes termed the "water head support") of the membrane. Highly porous membranes, such as geotextiles, exhibit relatively low wetting pressures (up to 10cm) while membranes composed of continuous synthetic films exhibit extremely high wetting pressures - in excess of their mechanical rupture resistance (i.e. these membranes fail by mechanical rupture before they allow liquid water to pass). When external water pressures are less than the membrane wetting pressure, moisture may still migrate through the membrane in the form of water vapor. Thus, whether a membrane behaves as a water repellent layer (geomembrane) or a water permeable layer (geotextile) depends solely on the external water pressure in relation to the wetting pressure of the membrane. By inference, the wetting pressure of a geomembrane must be always greater than the external water pressure (to prevent liquid water migration) while the wetting pressure of a geotextile must be always less than the external water pressure (to enable liquid water migration). It is also possible to have the same membrane performing a geotextile function (liquid water passage) for one set of external moisture conditions and a geomembrane function (liquid water repellency) for another set of external moisture conditions.

Once establishing this inherent water repellency principle governing geomembranes, it is necessary to consider the migration of water vapor through the geomembrane, and the effect it has on geomembrane selection. When using geomembranes as moisture barriers, the passage of moisture vapor through the geomembrane is not necessarily detrimental, provided it does not lead to failure of the protected geotechnical structure, or to excessive moisture losses. To ensure this does not occur, design techniques are required which permit geomembrane selection based on

*It is well known that fine grained, especially cohesive, soils are very susceptible to moisture content change by vapor transfer due to capillarity.

minimum allowable water vapor migration.

THEORETICAL APPRAISAL

The rate which water vapor migrates through a geomembrane is determined using Fick's Law:

$$\frac{\partial W}{\partial t} = K.A_s.\frac{\partial C}{\partial x} \quad (1)$$

where $\partial W/\partial t$ is the rate of water vapor diffused through the geomembrane, $\partial C/\partial x$ is the vapor pressure gradient acting across the geomembrane, A_s is the gross surface area of the geomembrane, K is a diffusion or permeability constant (for water vapor).

If K is independent of the vapor pressure the boundary conditions are:

$$\text{at } x = 0,\ C = C_1$$
$$x = L,\ C = C_2$$

Thus, integrating equation(1) gives:

$$W = \frac{K.A_s.\Delta C.t}{L} \quad (2)$$

where W is the weight of water vapor diffused through the geomembrane, $\Delta C=(C_1-C_2)$ is the vapor pressure difference across the geomembrane, t is the time for diffusion, L is the thickness of the geomembrane.

For geomembranes, the term K/L in equation(2) is commonly known as the membrane diffusivity or permeance. Thus, equation(2) may be rewritten as:

$$W = K'.A_s.\Delta C.t \quad (3)$$

where K' is the membrane permeance or diffusivity.

For geomembranes composed of continuous homogeneous films the permeability constant K is determined based on a standard material thickness. The permeance of different geomembrane thicknesses of that same material can then be determined by dividing the permeability constant by the thickness of each of the geomembranes concerned. For geomembranes composed of discontinuous films (e.g. partially saturated geotextiles) however, the membrane permeance should be determined directly from the works supply material. For these geomembranes, the use of a permeability constant value (K) to determine the permeance for any thickness of membrane other than the prototype is not applicable because discontinuities in one geomembrane are not necessarily identical to those in another geomembrane (composed of the same material) of different thickness, and consequently, the reciprocal relationship between membrane permeance and thickness as depicted in equation(2) does not apply.

For geotechnical structures where the geomembrane divides two relatively large areas whose permeabilities are much greater than that of the geomembrane, the vapor pressure difference acting across the geomembrane can remain constant for long periods of time (provided the temperature does not change significantly). For this condition equation(3) can be applied directly to determine the amount of moisture passing through the geomembrane. Equation(3) applies to situations where there is no nett build-up, or loss, of soil moisture on either side of the geomembrane. An example of this would be where a geomembrane is used to control seepage losses from a canal founded in an extensive sand stratum.

For geotechnical structures where the geomembrane abuts soils with permeabilities which restrict the movement of moisture away from the geomembrane surface, or where one side of the geomembrane forms an enclosed mass, and where the external vapor pressure changes relatively quickly compared to that in the protected zone, the vapor pressure difference at a particular point in time will depend on the nett flow of moisture through the geomembrane to that point in time. An example would be where a geomembrane is used to protect a clay canal bank from wetting-up and collapsing. To allow consideration of this condition it is necessary to modify equation(3) to the following:

$$\frac{\partial W}{\partial t} = K'.A_s.\Delta C_0.e^{-(\frac{K'.n.t}{R})} \quad (4)$$

where ΔC_0 is the initial vapor pressure difference across the geomembrane (at time t=0), R is the depth of moisture penetration on the contained,or soil permeability vapor pressure,side of the geomembrane, e is the Napierian constant, t is the time over which diffusion has occurred, n is a thermodynamic constant which relates vapor density to vapor pressure ($n=1.033m^3.mmHg/g$ at 25°C).

The change in vapor pressure on the side of the geomembrane which is wetting-up or drying-out is:

$$C_2 = \Delta C_0 - \Delta C_0.e^{-(\frac{K'.n.t}{R})} \quad (5)$$

where C_2 is the change in vapor pressure after time t on the side of the geomembrane which is wetting-up or drying-out.

When zero vapor pressure difference is achieved no further moisture vapor is diffused. Further vapor migration can occur only if, either the vapor pressure difference is altered physically on one side of the geomembrane, or the water vapor on one side of the geomembrane starts to condense thus lowering its corresponding vapor pressure.

To observe the effect of geomembrane permeance on the rate of moisture transmitted through the geomembrane when the protected soil is wetting-up, as an example, suppose the initial vapor pressure difference across the geomembrane (ΔC_0) was 0.71mmHg, and the depth of moisture penetration into the protected soil layer was limited to 2m. The relationship between the rate of water diffused per unit geomembrane surface area, time, and geomembrane permeance is shown in Figure 1. As anticipated, use of equation(4) shows the rate of water vapor transmission through the geomembrane reducing with time. The reason for this is because the vapor pressure increases on the lower vapor pressure side of the geomembrane, due to a nett build-up of moisture, thus, reducing the vapor pressure difference. This rate of reduction is more pronounced with relatively high permeance geomembranes.

If this change in vapor pressure difference with time was not considered, and instead,equation(3) used to determine the vapor transmission rate across the geomembrane, it could lead to an overestimate (see Figure 1), especially in the region where $K'.t>1g/m^2/mmHg$. In the region where $K'.t<1g/m^2/mmHg$ equation(3) provides good approximation to equation(4).

Integrating equation(4) gives the weight of water vapor diffused through the geomembrane:

$$W = \frac{\Delta C_0.A_s.R}{n}\left[1 - e^{-(\frac{K'.n.t}{R})}\right] \quad (6)$$

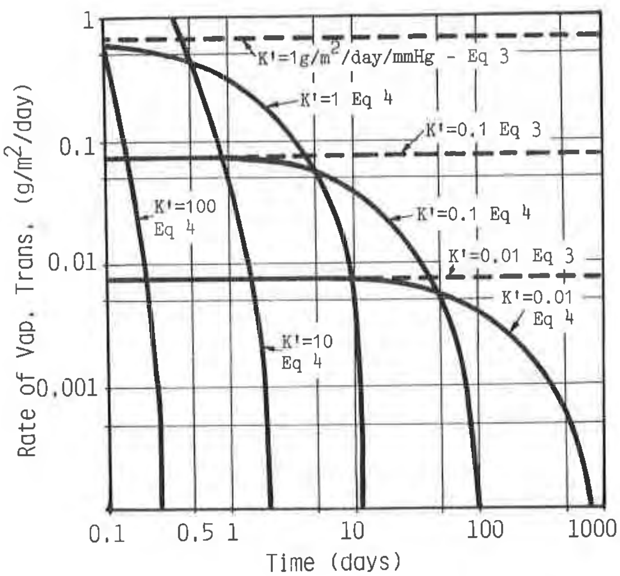

Figure 1: Water Vapor Transmission Rates Versus Time and Geomembrane Permeance for Given Boundary Conditions.

DESIGN CONSIDERATIONS

In considering the use of geomembranes as moisture barriers it is critical to ensure the wetting pressure of the geomembrane always remains greater than any external water pressures which are likely to come into contact with the geomembrane during service. *By far the most damaging condition which can occur is if the external water pressure becomes greater than the geomembrane wetting pressure and liquid water passes through into the protected soil stratum.* This can lead to very rapid water losses, or wetting-up (and possible collapse) of the protected soil stratum. This situation may occur either, by initial poor choice of the geomembrane (its wetting pressure may not be high enough to begin with), by structural flaws in the geomembrane (caused by manufacturing quality control problems or damage during installation), or by durability defects (the geomembrane ages prematurely).

Geomembranes composed of continuous synthetic films have very high wetting pressures (provided there are no structural flaws) and as such, wetting pressure is not normally a prime consideration, outside that of standard quality control procedures at the point of manufacture and on the job site. The prime consideration for these geomembranes is structural integrity to ensure that the likely external water pressures do not rupture the geomembrane and that it will not be damaged during installation.

For geomembranes composed of discontinuous films it is important to consider the geomembrane wetting pressure in relation to the in-service external water pressures. Geomembranes composed of discontinuous films exhibit wetting pressures significantly less than those having continuous films. Figure 2 shows the results of initial studies on the wetting pressure of geomembranes manufactured from various nonwoven geotextiles saturated with varying amounts of asphalt. The results show that for relatively small increases in the amount of asphalt retained in the geotextile the wetting pressure increases markedly. Because of their compressed, denser structure, melt bonded nonwoven geotextiles require smaller amounts of asphalt to achieve a given wetting pressure, than do needlepunched nonwoven geotextiles of similar mass per unit area.

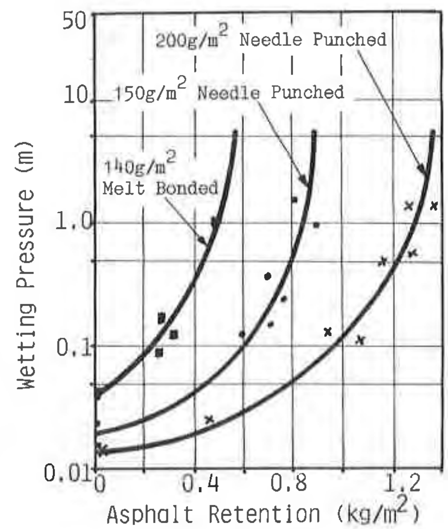

Figure 2: Wetting Pressures for Various Nonwoven Geotextiles Partially Saturated With Asphalt.

The design of geomembrane moisture barriers also requires the determination of a design geomembrane permeance. In the case of a containment structure, a design maximum geomembrane permeance is required to ensure unacceptable seepage losses do not occur. The design geomembrane permeance for a containment structure, founded on a permeable stratum, can be determined utilising the following equation:

$$K'_{design} = \frac{W_{allow}}{A_s.\Delta C.t.F_s} \qquad (7)$$

where K'_{design} is the design geomembrane permeance, W_{allow} is the allowable loss of water vapor, F_s is the design safety factor (normally equal to 2).

For geotechnical structures where the geomembrane is required to protect the wetting-up or drying-out of a soil:

$$K'_{design} = \frac{R}{n.t.F_s} \ln\left[\frac{(w_t-w_o).\gamma_d.n}{\Delta C_o} - 1\right] \qquad (8)$$

where (w_t-w_o) is the allowable moisture content change in the protected soil mass, γ_d is the dry density of the protected soil mass.

Many continuous organic films exhibit very low membrane permeance. Some examples are shown in Table 1. The good water repellency of asphalt should be noted.

Geomembranes composed of discontinuous films, unlike those of continuous films, may exhibit relatively high membrane permeance. This is because relatively large amounts of water vapor can migrate through the structural discontinuities, compared to that able to migrate between

Table 1: Water Vapor Permeabilities for Various Geomembrane Films, (2).

	Water Vapor Permeability (K) (g-m/m²/day/mmHg)x10^5
Asphalt, Oxidised	1.4-2.8
Hard Rubber	3.6
Neoprene, Vulcanised	6.2
Plasticised Vinyl Chloride	9
Soft Vulcanised Rubber	17
"Waterproof" Cellulose Film	190

or along, the molecular structure of the membrane material. Lawson(1) determined permeance values for geomembranes composed of a single geotextile substrate partially saturated with varying amounts of asphaltic materials (see Figure 3). Relatively small additions of saturant reduce the geomembrane permeance markedly (by a factor of 10). From Figure 3, it is observed that a general relationship exists between membrane permeance and saturant retention, viz:

$$K' = B.e^{-b.S_{ret}} \qquad (9)$$

where S_{ret} is the saturant retention weight, B and b are impregnation constants dependent on the type of substrate and saturant used.

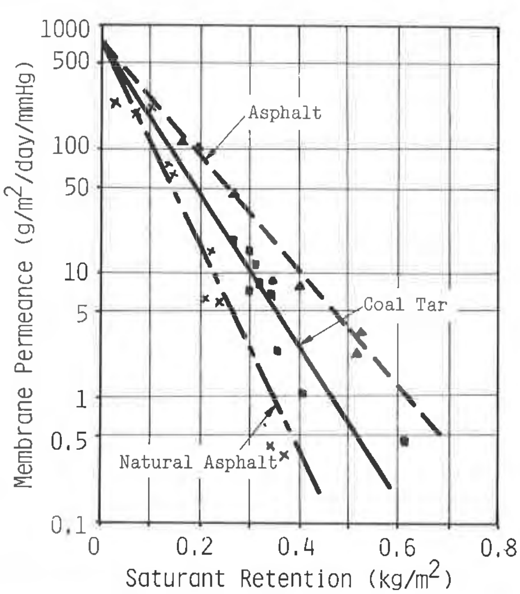

Figure 3: Relationship Between Geomembrane Permeance and Saturant Retention Weight for an Impregnated 140g/m² Melt Bonded Nonwoven Geotextile.

The constant B is dependent on the permeance of the substrate in isolation, while the constant b is dependent on the type of saturant. Values of the impregnation constants (B,b) for the 140g/m² melt bonded nonwoven geotextile impregnated with the relevant saturants can be obtained from Figure 3, and are listed in Table 2. It is observed, that of the three saturants tested, the natural asphalt is the most efficient, from the viewpoint of the amount of saturant required to provide a specified membrane permeance. Obviously, as well as saturation efficiency, the cost of materials has to be considered in the selection of a suitable saturant-substrate combination for a geomembrane application.

Table 2: Impregnation Constants for 140g/m² Melt Bonded Nonwoven Geotextile Impregnated with Natural Asphalt, Coal Tar, or Asphalt.

Saturant Type	Impregnation Constants		
	B (g/m²/day/mmHg)	b (m²/kg)	coeff.var.(%)
Natural Asphalt	700	19.6	22
Coal Tar	700	14.3	26
Asphalt	700	10.3	24

PRACTICAL IMPLICATIONS

Geomembranes are well known for their use as water repellent linings in water containment structures (e.g. dams, canals, ponds, etc) and under foundations. For these structures, their water repellency requirements are generally well understood. More recently, however, geomembranes have found use, on an increasing scale, as moisture barriers to protect soils from the adverse effects of moisture fluctuations. Their water repellency requirements for this application are not well understood because the moisture regimes can be highly variable and complex, and thus difficult to predict.

The presence of moisture in the soil affects many of its properties, namely, shear strength, volume change, stress-controlled deformation, and fatigue characteristics. In designing geotechnical structures it is important to ensure soil moisture changes do not lead to critical changes in the soil properties throughout the design life of the structure. This is of particular importance in applications involving foundations, slopes, earth dams, retaining walls, and pavements. Geomembranes may be used to control the ingress and egress of moisture in these structures, and thus maintain their integrity. It is important to note that the geomembrane has to control moisture movements only during periods of external moisture fluctuation, and as such, need not be completely water repellent(c.f.(3)). The required geomembrane permeance has to ensure that external moisture fluctuations do not affect the integrity of the protected soil mass.

For pavements, geomembranes may be used to stabilise frost susceptible soils, expansive clays, substandard granular materials, and protect from failure due to "random events" (e.g. flooding). Depending on the moisture conditions and the design requirements, the geomembrane may be utilised either as a single layer, or to fully encapsulate the soil (e.g. Membrane Encapsulated Soil Layer-MESL).

CONCLUSIONS

The geomembrane properties pertaining to water repellency are wetting pressure and permeance. Wetting pressure governs the mode of moisture transfer (liquid or vapor) through the geomembrane while permeance is a measure of the rate of moisture vapor transfer. For geomembranes, the wetting pressure must be always greater than the external water pressure to ensure no liquid water can be transmitted.

Common specifications citing minimum possible geomembrane permeance (approaching impermeability) may be overconservative in many cases, especially where the protected soil mass is exposed to fluctuating external moisture conditions. In these cases the geomembrane is required only to ensure the protected soil mass does not undergo a moisture change which leads to a critical change in the relevant soil properties.

Guidelines for determining the design geomembrane permeance are based on thermodynamic and geotechnical relationships. These relationships are particularly useful in the design of geomembranes composed of discontinuous films and as a means of assessing structural defects in geomembranes composed of continuous films.

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The Behaviour of Geomembranes in Relation to the Soil

Manufactured geomembranes are characterized by very high watertightness performances. It's obvious that they have to keep such characteristics once set up and to resist the water head predicted for the structure. However, it is difficult to extrapolate the behaviour of a geomembrane subjected to stresses resulting from both the putting into service and the operation if only taking conventional mechanical tests into account.
The behavioural tests presented in the paper are aiming at simulating in laboratory the main stresses undergone by the geomembrane "in situ" :
-"mechanical puncturing"of the punching membrane when the protective layer is laid down
-the head of the water, which pushes the geomembrane against the supporting structure in operation.
Such effect is simulated through rupture and "hydraulic puncturing"tests on a granular supporting structure.A first approach to geomembrane dimensioning in relation to the support (granulometry of the support of a given diameter, dimensioning at a potential fissure) is drawn from both the above-mentioned tests and the conventional tests.

Applied in the field of civil engineering, geomembranes have remained, a priori, somewhat incongruous in most people's minds, primarily because of their thinness, but also because of their flexibility and a certain susceptibility to damage. Geomembranes are, in fact, characterized by a certain number of physical, mechanical and hydraulic parameters which do not come within the field of conventional values : consequently, this necessarily involves the imparting of specific information to those involved in civil engineering and it also requires a special approach to the design and building of structures.

Whereas for conventional materials laboratory tests are used to measure the specific characteristics of the materials, for example modulus or tensile strength, geomembranes undergo not only these conventional tests but also functional tests whose aim is to simulate, in a laboratory, phenomena which may happen in situ. The blistering and puncturing tests described in this report can be situated in this context (1, 2)

I- BLISTERING TEST

The apparatus used for this test is circular and made of aluminium. The effective diameter of the apparatus is 29 cm (distance between two facing screws). A plate is placed beneath the membrane. There is a circular hole in the centre of this plate (diameters 1.25 ; 2.5 and 5 cm).

For a slow increase in pressure (500 kPa for 5 minutes), here are the failure pressure obtained for holes 1.25 and 2.5 cm in diameter.

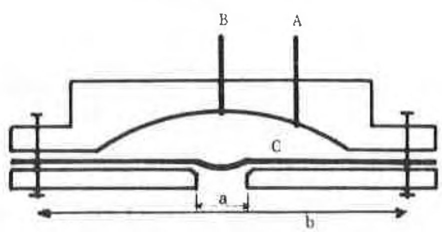

Fig. n° 1 - Blistering test apparatus
a = 1,25 2,5 or 5 cm ; b = 29 cm
A : water input
B : air output
C : geomembrane

Table 1 - BLISTERING PRESSURE

Film or sheet	thickness (mm)	Failure pressure (Kpa) hole Ø 2.5 cm	hole Ø 1.25 cm
butyl	0.75	200	500 to 550
butyl	1	350 to 400	650 to 700
butyl	1.5	450 to 500	200 to 350
P.V.C.	0.5	550 to 600	1150
polyethylene B.D	0.45	600 to 650	1200 to 1250
hypalon	0.70	500 to 850 (+)	1000 and more

(+) The spead noted for the hypalon is due to the use of sheets of different manufacture.

In the butyl the failure was star-shaped without measurable plastic deformation and rather like the arc of a circle in the hypalon ; in the plastomers it was a straight slit (with intense residual deformation and considerable reduction in thickness).

II - HYDRAULIC PUNCTURING

This is the case when the stored liquid pressing the membrane against its support. Puncturing of the membrane can then occur due to sharp edges in the support, or blistering of the membrane in the voids or fissures of the support.

This situation corresponds to a normal functioning of the structure and was therefore the test which we, like other laboratories, carried out first of all.

2.1 - The apparatus used at the CEMAGREF

Three types of apparatus are in general use

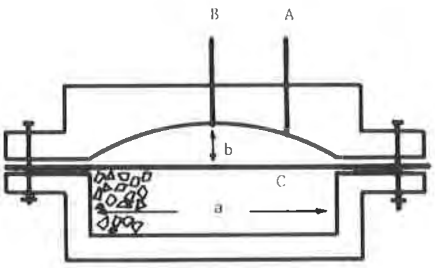

Fig. n° 2 - Puncture test apparatus A
a = 20 cm ; b = 4 cm
A : water inpout
B : air output
C : geomembrane

Apparatus A is made of aluminium. Apparatus B, of similar but simpler design, is reserved for bituminous membranes and has the same effective diameter, that is to say 20 cm. Given the difficulty we experienced in reaching 1.5 mPa with apparatus A and B, and also a general conviction that the edge action was too significant, a third apparatus was made with the collaboration of the SIPLAST company. Apparatus C (see fig 3) has an effective diameter of 60 cm and in order to carry out the tests it is necessary to have a sample more than 80 cm in diameter.

Apparatus A and B are fitted with a volume change tube which is used to pass from air pressure, arising from a compressor, to water pressure ; this tube makes it possible first to follow the deformation of the membrane and then to estimate the maximum amount of leakage through the membrane (in effect, the residual creep of the membrane due to the action of the pressure can increase the volume changes in the tubes).

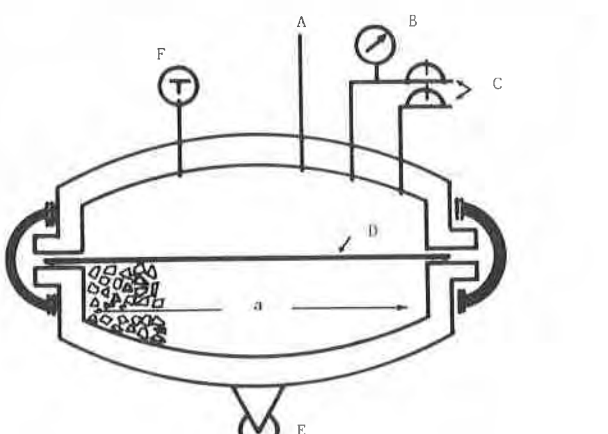

Fig. n° 3 - Puncture test apparatus C
a = 60 cm (diameter) ; b = 20 cm (water thickness)
A : water input
B : manometer
C : air output
D : geomembrane
E : leakage signal
F : security valve

2.2 - Some results of rapid puncturing tests on a standardized support

The standardized support consisted of a bed of crushed 20-40 quartzite stones.

In the course of rapid tests the pressure is increased by stages of a 100 kPa after each one minute test. The test ends either when there is failure of the membrane, shown by leakages through the support, or when the pressure limit of the apparatus is reached (1 300 to 1 500 kPa depending on the apparatus).

A series of fifteen tests carried out on a sheet 0.8 mm thick gave a mean failure pressure of 1 000 kPa and a standard deviation of 60 kPa. It is therefore evident that this type of test cannot constitute a manufacturing control but, on the over hand, makes it possible to classify the linings.

The table below gives the approximate mean results obtained for several hundred tests.

Table 2 - FAILURE PRESSURE OF THE MEMBRANES DURING RAPID HYDRAULIC PUNCTURING TESTS.

Product	Thickness (mm)	Failure pressure (kPa)
Reinforced hypalon	0.8	1000
Butyl	1	550 to 700
Butyl	0.8	500 to 600
E.P.D.M.	1	1100 to 1300
Polyethylene LD	0.4	300 to 400
Polyethylene HD	2.5	> 1300
P.V.C.	0.5	800
P.V.C.	1	900 to 1300 and more
Reinforced P.V.C.	1.	> 1300
P.V.C.	1.5	> 1300
Reinforced elastomer bitumen	3	1200 to 1300
Reinforced elastomer bitumen	4	> 1300
Reinforced bitumen	3.5	600 to 700
Reinforced bitumen	4.6	900 to 1000
Reinforced bitumen	6	> 1300
Reinforced bitumen	5	> 1300

Generally speaking, failure of the thinnest elastomer or plastomer linings happens due to "lack of support" because of the spaces between the stones. On the other hand, the thicker, more rigid linings have a pronounced tendency to correctly span the defects in the support, the failures occuring where the stones are pointed or have sharp edges.

The susceptibility of the thinnest membranes to support defects was confirmed at the time of tests carried out on supports comprised of rolled pebbles of similar size, in the course of which failure occured at a lower pressure.

In the report, the effect of geotextiles is not dealt with, but is should be remembered that association with geotextiles considerably modifies the failure pressure.

2.3 - Puncturing tests of long duration

The creep tests carried out by certain laboratories have shown that the membranes have a visco-elasto-plastic type behaviour.

In connection with this, one could suppose that the failure limit of hydraulic puncturing was also dependent on the load duration.

Puncturing tests of long duration are caried out using the same apparatus as previously.

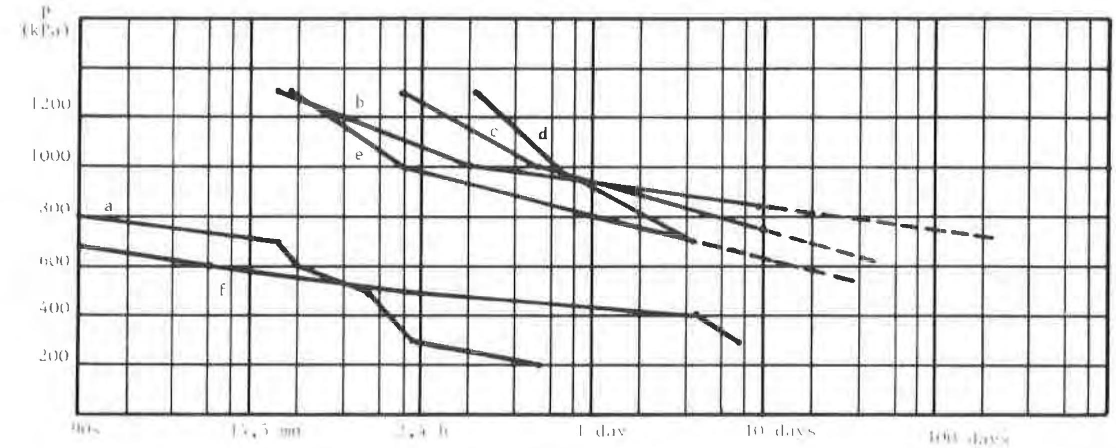

Fig. n° 4 - Long time puncture test
a-b : bituminous geomembrane
c-d-e : PVC geomembrane
f : butyl

After an application of a load taking place in the same way (100 kPa per minute), the pressure is stabilized to a constant value and the application time of the pressure necessary for failure -established by leakage through the support- is measured.

After these tests the manufacturer modified the composition of membrane (a), and the latter is no longer made. Butyl is comparatively less susceptible to load duration.

Compared to the failure pressure in rapid tests, a minimum coefficient of 0.5 is applied in order to obtain the failure pressure after 10 days of application of a load.

We have the impression that there is a pressure limit within which the failure ought not to occur, whatever the load duration (less than 30 years) ; it would appear to be in the region of 500 kPa for the bituminous membrane (b) and 250 kPa for the butyl (f).

It should be noted that the failures are very different depending on the products : there is a clean break with the thinnest membranes, but on the other hand the leakages remain very slight with bituminous membranes.

2.4 - Hydraulic puncturing tests with varying supports

Comparison of support agressiveness is a difficult task and the approach suggested in § 2.1 merits further study.

At the request of J.P. GIROUD, puncturing tests on butyl one mm thick were carried out at the time the PONT DE CLAIX basin was designed, on a porous concrete support comprised of rolled granulates.

The failure times were as follows : 3 hours at 1250 kPa, 10 days at 900 kPa and 40 days at 600 kPa ; the failures occured either in the region of voids in the support or over sand grains stuck to the gravel. In these circumstances, it appeared advisable to add a geotextile between the porous concrete and the butyl.

On 5-10 gravel, the failure of the butyl occurs after 2 hours at 1250 kPa and 120 hours at 750 kPa.

Tests on pyramidal shapes carried out by University of Liège and presented by RIGO (ref. 1) seem to present a more theoretical but more rigorous approach to the problem of puncturing. It should be remembered that in this test the support was comprised of sand out of which three regularly-spaced pyramidal shapes emerged. At a given pressure, there was failure at a particular height of emergence of the pyramids. This height made it possible, at a given pressure, to estimate the support aggressiveness which the membranes could withstand.

At request of the COLAS company, tests were carried out with a support comprised of a thick metal plate pierced with cylindrical holes 10 mm in diameter and a spacing between their axes of 20 mm. Water pressure is applied to the upper surface of the sample. It was possible to establish that this support is less aggressive than the bed of 20-40 quartzite. In a rapid test there is failure of the bituminous membrane at 900 kPa on quartzite, no damage at 1300 kPa with the plate and the same membrane. In a test of long duration (8 days) at a pressure of 1300 kPa a phenomenon of bitumen extrusion occured, where by the latter passed through the plate like toothpaste. Under these circumstances, this type of support did not undergo any subsequent tests.

III - MECHANICAL PUNCTURING

This refers to circumstances where the membrane is stressed by solids on both sides. These stresses generally occur during installation. In particular one can cite :

- action of metallic tubes (scaffolding for example) which can pierce the membrane like a punch. Tests simulating this behaviour have already been carried out. They chiefly concern the waterproofing of buildings.

- dynamic action corresponding to a block or a concrete hollow brick falling into the membrane. The effect of such an impact depends on numerous parameters : weight, shape, hardness and speed of the falling body, modulus and aggressiveness of the support etc... since the risks of damage or rupture are considerable, it would therefore seem reasonable to advocate systematic repair of any zone subjected to this type of impact (falling of hollow bricks, for example) at the end of the construction works. Development of a specific test does not therefore seem to us to be of priority.

- action of two stones positioned either side of the membrane under static stress. This is particularly the case with a membrane covered by a protective layer of granular materials. The "mechanical puncturing" test that we present corresponds to this type of stress. Its objective is to enable one to choose the geomembrane, the materials either side of it and the conditions for installing the protective layer so that there is no risk of loss of watertightness as a result of the hydrostatic pressure.

3.1 - The apparatus

This is derived from hydraulic puncturing apparatus A, by interposition of a level raiser which makes it possible to place the covering material on the membrane (fig. 5).

The membrane is placed on its support. After positioning of the level raiser, it is covered with the covering material.

Loading and unloading are executed using a CBR press at a rate of 1.27 mm/mn. The stress is maintained at its maximum level for 1 minute when one is seeking to simulate the action of compactors and longer still when simulating the action of a permanent load (10 to 30 mn). After unloading, removal of the covering materials and the replacement of the level raiser by the original cover, a conventional hydraulic puncturing test is carried out. For this test the membrane is not shifted from its support and the test result is given in the form of hydraulic pressure corresponding to the appearance of a leakage.

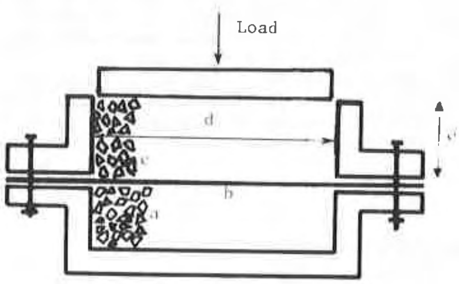

Fig. n° 5 - Mechanical puncturing apparatus
a) supporting layer
b) geomembrane
c) covering material
d) diameter of 20 mm
e) level raiser
f) base of apparatus A

For the systematic study of membrane behaviour under this type of stress, we wish to show the influence of the following parameters :
grain size distribution and shape of support material
grain size distribution and shape of covering material
eventual geotextile protection (upper, lower)
thickness of the membrane
stress applied

3.2 - Some results

A first study was carried out on the PVC (Terlin) on the following range of thicknesses
0.8 ; 1 ; 1.2 ; 1.5 ; 2 mm
The characteristics of these membranes are as follow :
- Tensile strength :(norm DIN 53455)
  Tensile stress > 18 MPa
  Elongation at rupture 300 %
  Modulus at 100 % elongation > 8 MPa

- Hydraulic puncturing on 20/40 quartzite
  Thickness 0.8 mm : failure after 20 mn at 13500 kPa
  Thickness 1 mm : failure after 60 mn at 13500 kPa

Support material : 20/40 quartzite
20/40 rolled gravel
Covering material : fine limestone gravel (mixture of crushed and rolled Seine limestone)
10/25 quarry run gravel (mixture of rolled gravel and silex)
10/20 quarry run gravel (mixture of rolled gravel and silex)
10/20 crushed quartzite
10/20 rolled gravel
20/40 rolled gravel

Geotextiles : at the beginning we used the Bidim range of products (needlepunched nonwoven geotextile made of continuous polyester filaments).

Stress applied : from 100 to 500 kPa.

These results are still too incomplets to be able to draw any general conclusions, but it is possible at the moment to make several remarks.

Simple visual inspection, after mechanical puncturing, is not sufficient to estimate the deterioration of the membrane.Leakages can occur with faint, scarely visible traces on the membrane. In other cases the membrane has very distinct marks due to mechanical puncturing but does not have a diminished resistance to hydraulic action.

- The extreme fragility of the 0.8 mm thick membrane to this type of stress, even with respect to fine 0/10 gravel, can be noted. This membrane remains fragile, even after the placing of a geotextile on the side where there is the coarsest material. The 1 mm thick membrane still remains fragile, although less susceptible ; geotextile protection seems necessary.

- In this first study, we have been carrying out tests on three samples for given parameters. The surface to be tested is therefore very small (each sample representing 1/32 m2). When the membrane resists a pressure greater than 13500 kPa, one can consider that it has not undergone any deterioration in its watertight properties. When the membrane is pierced in the course of every test, it is evident that the solution is not a satisfactory one. If, by slightly varying one of the parameters it seems to resist very well, whereas it became pierced just before, there is no guarantee that there would be no perforations over a large area. Moreover, for the calculation of the dimensions of real structures, we are considering using ten samples for a specific group of parameters.

It should be noted that the effect of one geotextile below the geomembrane is very important. The characteristics of tested geotextile are as follow :
thickness : 2,3 mm
tensile strenght : 22,3 kN/m
elongation at rupture : 44 %

- Much completer results and the first conclusions drawn, will be presented in June during the Congress.

Table 3 - mechanical puncturing - first results on 20/40 quartzite support, giving failure pressure in kPa in the hydraulique puncturing tests carried out after mechanical puncturing tests.

Failure pressure in kPa			without use of geotextile			with use of Bidim U 34 beneath the membrane		
	mechanical puncturing stress		100 kPa	300 kPa	500 kPa	100 kPa	300 kPa	500 kPa
	thickness membrane	covering material						
	0,8 mm	2/10 crushed limestone gravel	0 100 > 1350	700 0 0		> 1350 > 1350 > 1350	0 > 1350 > 1350	
		10/30 mixture rolled gravel/silex	> 1350 > 1350 > 1350	0 0 0		> 1350 > 1350 > 1350	> 1350 300 1000	
		10/20 mixture rolled gravel/silex	> 1350 > 1350 > 1350	0 0 0		> 1350		
		10/20 rolled gravel	> 1350	0 0 0		> 1350		
	1 mm	2/10 crushed limestone gravel	> 1350 > 1350 > 1350	100 0 > 1350		> 1350 > 1350 > 1350	> 1350 > 1350 > 1350	> 1350 > 1350 > 1350
		10/30 mixture rolled gravel/silex		> 1350 0 > 1350				
	1,2 mm	2/10 crushed limestone gravel	> 1350 > 1350 > 1350	> 1350 0 > 1350		> 1350		
		10/30 mixture rolled gravel/silex		0 0 > 1350			> 1350 > 1350 0	
		10/20 rolled gravel		0 > 1350 > 1350				
	1,5 mm	2/10 crushed limestone gravel	> 1350	> 1350 > 1350 > 1350	0 0 0		> 1350	

IV - METHOD OF APPROACH FOR CALCULATING THE DIMENSIONS OF A MEMBRANE IN RELATION TO ITS SUPPORT

4.1 - Calculating the dimensions above a potential crack

The first stage of the calculation consists of finding the stresses and deformations in the membrane. It is evident that the most unfavourable mechanism for the membrane is when the crack opens out after application of a load since the membrane is tretched to its maximum tautness.

The calculation is based on the following hypotheses :
- in the initial state, the fissure is closed, the membrane is not fixed and hydrostatic pressure is applied.
- in the final state the fissure has opened out by separation of the two sides.
- during the stage of the opening of the crack, friction is mobilized first of all at the angular points then beyond these angular points over an anchorage zone of length l.
- over the crack, the stress in the membrane equals hydrostatic pressure p multiplied by the radius of curvature R (one can then suppose the membrane to have a cylindrical form of deformation). (Fig. n° 8)

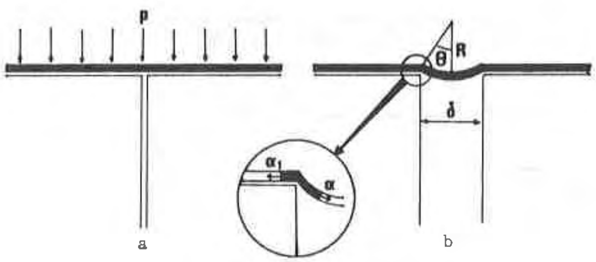

Fig. n° 6 - Puncturing test of long duration

a : before opening of the crack b : after opening of the crack

One has the following relationships :

$R = \frac{\delta}{2 \sin \theta}$ Geometry of the membrane above the crack

$\alpha = p R$ Membrane stress above the crack

$\alpha_1 = \alpha e^{-\theta \operatorname{tg} \phi_1}$ Transmission of the membrane stress to the angular point
tg ϕ_1 is the friction coefficient of the membrane in contact with the angular point

$\alpha_1 = l\, p \operatorname{tg} \phi_2$ Anchorage of the membrane
tg ϕ_2 friction coefficient of the membrane on its support
l length of anchorage

$2 R \theta = \left(\frac{l^2}{2} \operatorname{tg} \phi_2 + R^2 \theta\right) \frac{P}{E'e}$ Calculation of the elongation of the membrane
E' modulus taken in relation to the length after deformation (measured at zero lateral deformation)
e thickness of the membrane

This state cannot be resolved explicitly from the problem data, that is to say p, E, e, δ, tg ϕ_1 and tg ϕ_2, but a simple software based on the resolution of the equations by iteration makes it possible to solve this problem easily.

Here is an example of a solution :

$\phi_1 = \phi_2 = 27°$; E' = 10 MPa ; e = 1 mm ; δ = 10 mm ;
p = 100 kPa

One finds :

θ = 13° ; R = 22,2 mm ; l = 39,6 mm

α = 2,22 kN/m ; α_1 = 1,98 kN/m

This example is intended, above all, to make clear the relatively weak value of the anchorage length, and therefore of the zone affected by the crack (4 cm each side of the crack).

Having calculated the normal stress in the membrane α one can choose the membrane by adopting a failure value $\alpha_2 = F \alpha$ with F safety factor of between 3 and 5.

4.2. Calculating the dimensions in relation to hydraulic puncturing

One calls P_{lab} the failure pressure obtained in puncturing tests of short duration with 20/40 mm quartzite support.

One puts down $\overline{p} = \alpha \beta \gamma \delta P_{lab}$
where $\overline{p}$ is the allowable stress of the real structure.

α, β, γ, δ : coefficient taking into account the difference between laboratory testing and the real structure.

The following very approximate values are suggested.

α = uncertainty factor or intrinsic safety factor, α depends on the uncertainties of the other coefficients and some phenomena which they do not take into account.

β time factor

γ grain size distribution coefficient of the support
1 - log $d_{50}/20$ (d_{50} is the median diameter of the support expressed in mm, only to be used between 1 and 40 mm).

δ coefficient linked to the deformability of the support

δ = 1 low deformability
δ = 0.8 mean deformability
δ = 0.6 high deformability, risk of "pot holes"

If this method of approcach is applied to the Codole dam, one can put down a posteriori : $\overline{p}$ = 250 kPa ; α = 0,3 ; β = 0,2 ; γ = 1,6 (d_{50} = 5 mm) one finds P_{lab} = 3,3 MPa
This solution appears likely for a 2 mm thick PVC on a geotextile of 400 g/m².

V - SOME CONCLUSIONS

The tests described are not meant to serve as manufacturing controls. Their greatest merit is that they are applied to all the products whereas different norms exist- for tensile strength tests for example - for each major kind of product.

Creep and reduced resistance for stress of long duration are clearly shown by these tests. To improve resistance to puncturing and blistering, geotextiles are very efficacious, particularly when bonded. The puncturing test made it possible to compare the aggressiveness of the supports do not seem very satisfactory.

There is much left to be done before it will be possible to calculate the dimensions of membranes in relation to the risks of puncturing or tearing above a crack and the suggested method of approach must not be considered as definitive.

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Report on Two Dimensional Strain Stress Behaviour of Geomembranes With and Without Friction

Three-dimensional tests with geomembranes were made in a way of a bursting test for 9 different materials. The values are described and compared with the values obtained in the one-dimensional testing procedure. Deformation tests were made under different load and different conditions of friction. The values obtained in these tests are discussed in this report.

1. Introduction

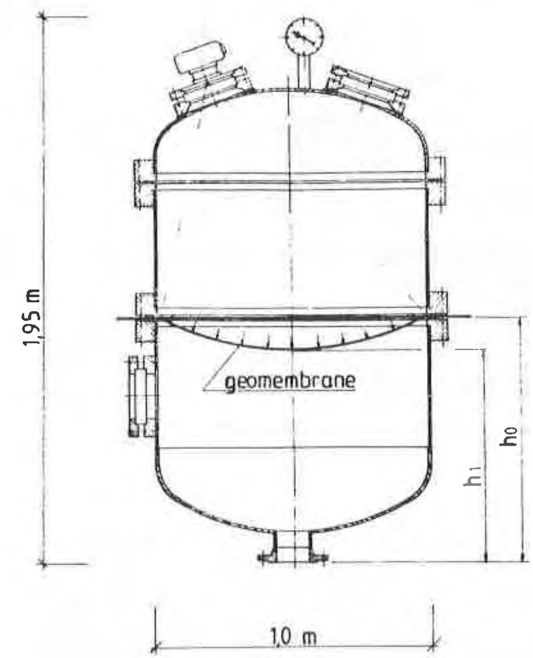
Since 1978 in my institute we made a lot of tests dealing with the strain stress behaviour of geomembranes under three-dimensioned stress conditions. The aim of the research is to find an explanation for some failures of geomembranes and looking for a possibility to design geomembranes under given conditions. These tests were made with large sized bursting pressure tests and deformation tests under pressure up to 15 bar and different embankment conditions.

2. Testing device and procedure

The testing ist device is a pressure vessel with a diameter of 1 metre which allows bursting tests on circular specimens with 1 metre diameter with and without a seam and deformation tests with a pressure up to 15 bar under different conditions.

2.1 Testing procedure for bursting tests

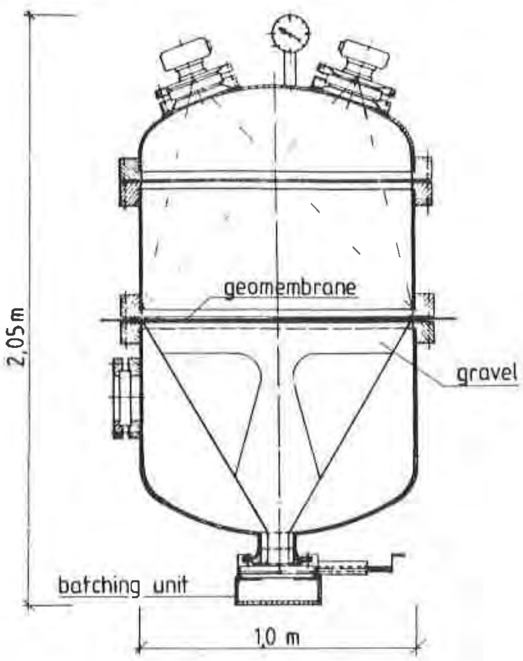
In the pressure vessel (pict. 1) the geomembrane is fixed between the lower and the middel section. The geomembrane is loaded with pressure from the upper side and the deformation ist measured with gauges from the upper side and on one additional point from the under side. The gauges can be observed through two windows. Through these windows the deformations are photographed on different loads. The form of the deformation was controlled by photogrametric methods and it was found that the deformation line is very near to the form of a ball section. The form of the geomembrane and the pressure at the same moment is measured and the strain and stress for the different stages of the tests are calculated. Normally the test is continued up to the bursting point.



Pict. 1 pressure vessel device for three-dimensional stress-strain tests

2.2 Deformation tests

For the deformation tests the same pressure vessel is used with an additional device for filling a gravel or other material as it is shown in pict. 2. The embankment material is filled up to the top of the lower part of the pressure device. The geomembrane is placed on this embankment and the pressure vessel is closed. The pressure is chosen corresponding to the expected pressure in the building that means under reservoir conditions the water pressure is used and by conditions for instance in solid waste disposals double the pressure ist used, as friction work from the upper and lower side of the geomembrane. It is possible to load up the upper part of the pressure vessel to 15 bar. After loading with the wanted pressure parts of the embankment material are drawn up through the batching unit on the lower part of the pressure vessel.



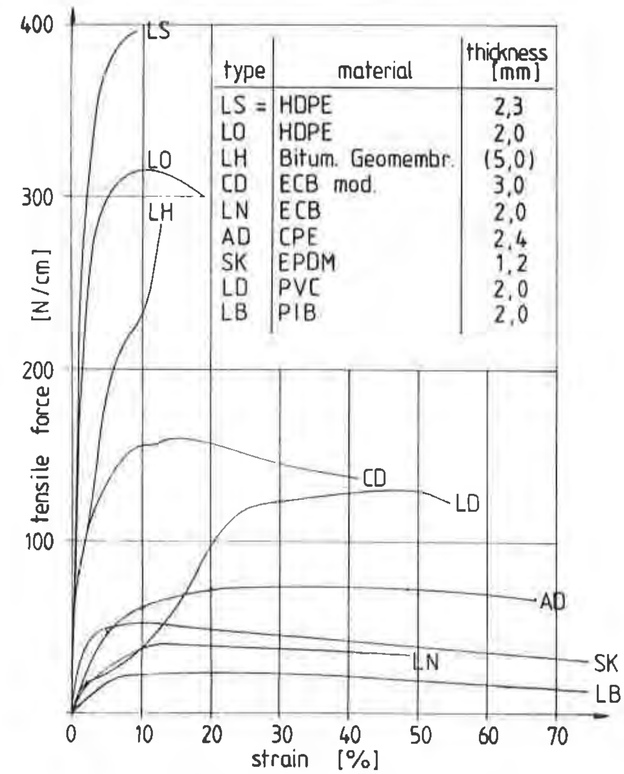
Pict 2 pressure vessel device for deformation tests and load

After the settlements, caused by drawing the material out of the embankment, the deformation of the geomembrane is measured with gauges and the surface is photographed by one or two cameras from the windows on the upper side. The settlement is increased in small steps up to bursting of the geomembrane.

3. Test results

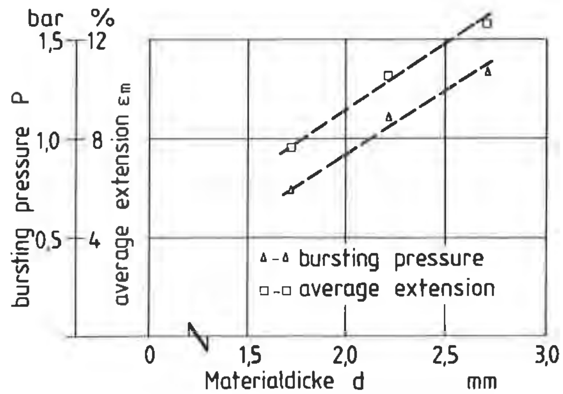
Pict. 3 shows the relation between strain and tension force for 9 different geomembranes. The thickness of the geomembrane is named in the picture so it is possible to find the strain/stress relation for these materials. The tension force is chosen to give realistic values for a bituminous geomembrane with a net reinforcement for which the stress gives only small information. The tests were made with two different HDPE materials, one PVC, one EPDM, one normal ECB material, one modified ECB and bituminous geomembrane with reinforcement with a net and polyester foil and one PIB geomembrane. The diagrams show that the HDPE material has a strain at failure of 9% or 15% this possible strain is about 1% of the strain which is usual measured by one -dimensional strain stress tests and at about 50% of the strain by the tensile yield point. This tremendous difference to the strain at failure which was normally praised by the manufacturers of geomembranes in the late sixties and early seventies seems to be at least one of the reasons for failure of some geomembranes in practice. The differences in the failure load between one-dimensional test and three-dimensional test are not so high. The difference between the strain at failure in the one -dimensional and three-dimensional testing procedure is not so high for the materials with a lower failure load. Here normally the values in the three dimensional tests are about 10% of those in the one-dimensional tests.

The failure happened with the HDPE materials in a small strip either after a high elongation in this small strip or with a spontaneous break. The bituminous geomembrane failed in a line after failure of the reinforcement. The materials with a lower failure load and a higher strain at failure normally failed after a high elongation in wide areas of the test specimen.

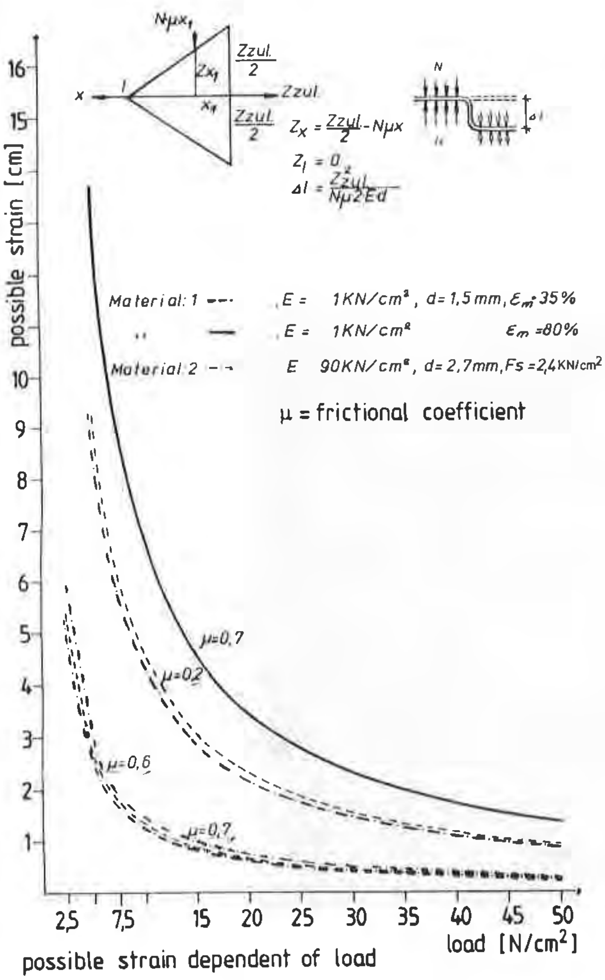


Pict. 3 strain-stress relation under three-dimensional conditions

To find a relation between the thickness of material and the strain at failure tests were made with a HDPE in different thicknesses between 1,6mm to 2,7mm. The results are shown in pict. 4. It is to be seen that the 1,68mm geomembrane has a strain at failure of 7,4%, the 2,10mm geomembrane a strain at failure of 10,2% and the 2,70mm geomembrane a strain at failure of 12,4%. These values show very clearly that the thickness of the geomembrane has an important influence on the allowed deformation in the embankment. These values confirm that we are on the right way to ask for geomembranes in Germany with not less than 2,0mm thickness when they are used as a permanent element in earth construction.



Pict. 4 relation between thickness of geomembrane and strain at failure

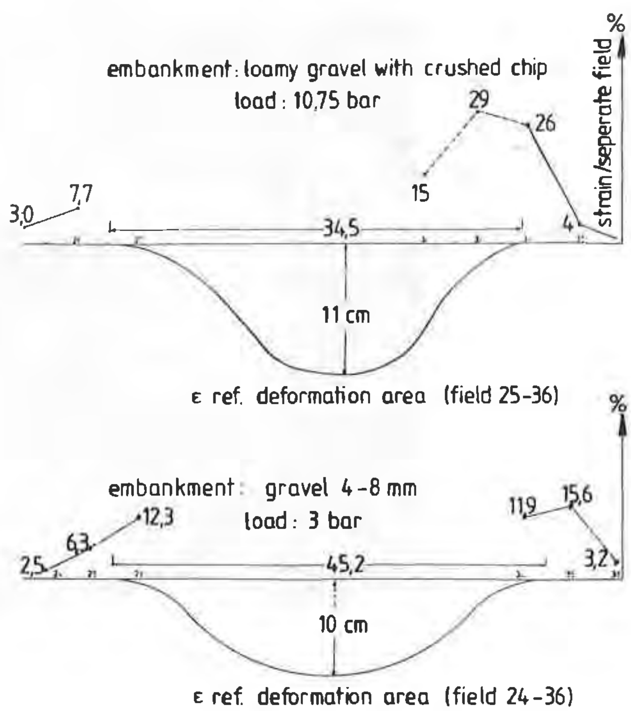


Pict. 5 relation between load and possible elongation

To design geomembranes for conditions in earthwork it is necessary to know the dependence between load, friction and strain stress behaviour. 1977 I showed the pict. 5 in Liège which gives a calculated possible elongation for geomembranes with different modules of elasticity, different failure load and different friction values between geomembrane and embankment. This diagram shows that a geomembrane with an allowed high tensile stress and a high modulus of elasticity has nearly the same values for the possible elongation as a geomembrane with a low tensile stress at failure and a low modulus of elasticity but with a high strain at failure. The reason is, that a strong material can use materials in the neighbourhood of the deformation area for the elongation too whereas the other materials have only a small possibility to share areas outside the deformation area on the strain of the geomenbrane, but as the allowed strain is a lot higher as for the strong geomembranes the deformation can be followed by the geomembrane in the place of deformation itself. This theoretical calculation can only be a very rough one as the modulus of elasticity is not a constant value for these materials and the friction depends on load and duration of load. To find a practice orientated answer to this question we made the under described deformation tests with geomembranes with friction under load.

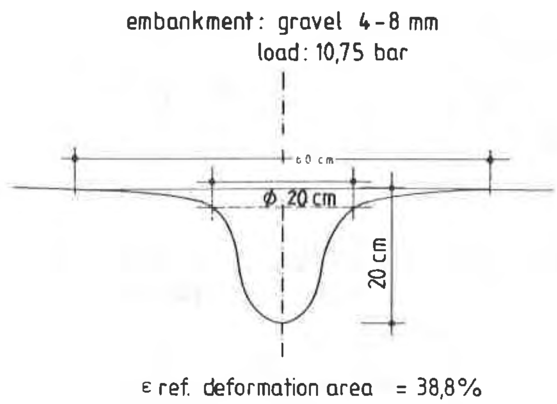
3.3.2 Results of Deformation Tests

The behaviour of the materials after deformations shall be explained on the results of three different materials, the HDPE material, the modified ECB material and the CPE material. These materials are the same as in the pict. 3 the materials LS, CD, and AD. These tests were made for different buildings, so the load is different and two materials for the embankment are used in adaption to the conditions in practice.

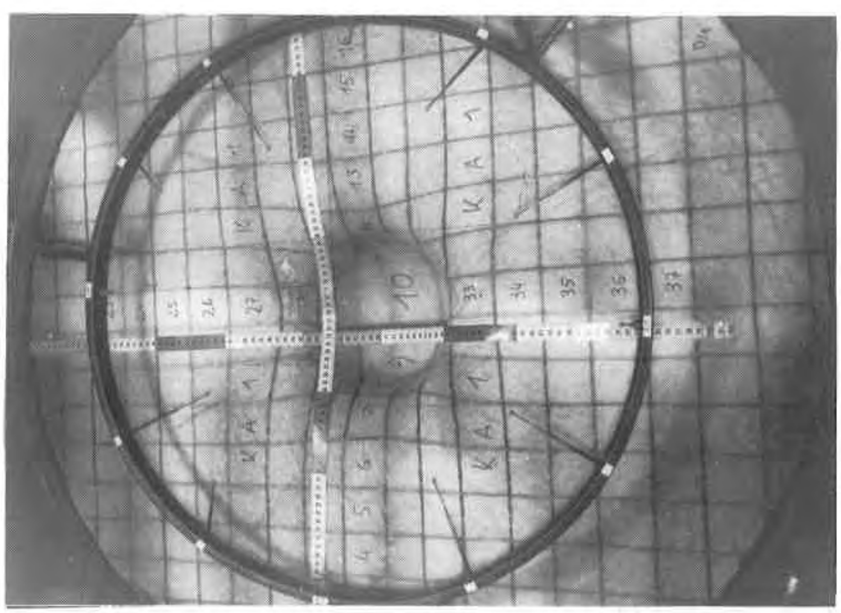


Pict. 6 cross section deformation area before failure -HDPE-

The HDPE material was tested on a single sized gravel 4-8mm under a load of 3 bar and on a loamy gravel with some percentage of crushed chips under a load of 10,75 bar. The elongation over the area of deformation was on the gravel about 13% and on the loamy gravel about 15,8%. Over the cross-section of the deformation the strain was different. There are relative high values with a maximum of 25% - 29% in the transition between undisturbed and disturbed area. The strain in the main part of the deformation area was lower (see pict. 6). It is of interest, that with both tests the strain reached the neighbourhood of the deformation area up to 15 cm, so that it is proved that the geomembrane share materials of the neighbourhood to follow the deformation. The higher strain on the embankment with loamy gravel seem to have the reason in the smaller diameter of the deformation area which caused, that on a smaller length of the geomembrane in the deformation area itself, the same helping length out of the neighbourhood could be shared, so that it was possible to follow a settlement with 11 cm instead of 10 cm bringing the increase of strain from 13% to 15,8%.



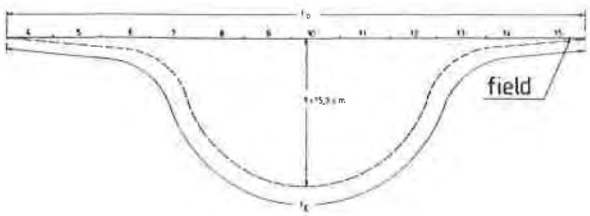
Pict 7 cross section deformation area before failure -ECB modified-



Pict 8 CPE geomembrane deformation test just before failure

The modified ECB was tested on the gravel 4/8mm with a load of 4 bar. The possible settlement before failure was 20 cm with a diameter of 60 cm in the upper aera, essential strain in the neighbourhood of the deformation area couldn't be observed. It is, however, to mention, that following to the high amount of withdrawn gravel the form of the deformation area was a little bit more friendly to the material as in the test before. The average strain of the deformation area for this material was 38,8%. This shows, that a high deformation is also possible with materials with a medium tensile force at failure, here a value of about 160 N/cm. The reason is that the allowable strain is so high ,that the material can follow the deformation without the necessity of sharing material in the neighbourhood (pict. 7).

embankment: gravel 4 - 8 mm
load: 4,0 bar



ε ref. deformation area (field 4-15)
= 29,1 %

Pict. 9 cross section deformation area before failure -CPE-

The third example ist the CPE geomembrane. The embankment material was a gravel 4/8 mm and the load was 4 bar. On the photograph of the geomembrane just before failure (pict. 8) it is to be seen that the strain has happened up to the end of the deformation area but with only very small amounts in the outer parts of the deformed area. As in this case the amount of withdrawn gravel was higher than those of the test with the HDPE material the form of the deformation in the upper part is similar to that in the test with the ECB material. The pict. 9 shows the cross-section of the deformed material just before failure. The possible elongation over the deformed area was 29% and the depth of the settlement is 15,3cm. Here the material could follow the deformations in the area of settlement itself.

4. Conclusion

The three-dimensional bursting test shows that under these conditions a strain stress behaviour is different from that which happens under the one-dimensional stress strain test. Especially the strain at failure is much lower as it is possible in the normal test procedure in the one-axial way and is smaller than the strain at the tensile yield point. The materials with a high elasticity modulus and a high load at failure have a higher difference between these above mentioned values. The deformation test shows that the highest flexibility can be obtained with materials with a medium stress at failure and a medium strain at failure in the three-dimensional bursting test (aprox 160 N/cm and 40%). The materials with a higher strength and a low strain at failure and the materials whith a low strength and a high strain at failure have a lower flexibility as the first mentioned materials. It is to emphasize, that all the three materials can follow deformations which normally can occur in earth construction. Under very difficult conditions in respect of settlements, it semes to be worthwhile to prove that the material chosen can follow the proposed deformations.

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Frictional Characteristics of a Geomembrane

SUMMARY

This investigation is an attempt to determine the frictional characteristics of a high density polyethelene geomembrane. The friction between the geomembrane and three materials has been investigated. The materials are: sandy soil, clayey soil and ballast. Both dry and saturated sands have been used in the study.

A specially designed Direct-Shear apparatus was developed for this investigation. The frictional characteristics under different normal loading have been evaluated. The study also provides an estimate of threshold deformations to mobilize the friction.

Since the shear box can accomodate and test geomembrane samples 11" x 11", it is believed that the values of frictional coefficient may be more realistic to the in situ conditions.

INTRODUCTION

The need to appropriately manage solid and liquid waste materials have brought increasing use of impermeable liners called geomembranes for many geotechnical projects. The use of these geomembranes is increasing; their mechanical behaviour in the soil and the behaviour of the membrane-soil system needs extensive study. This study involves an investigation of the frictional characteristics of a high density polyethelene geomembrane with three different geo-materials -- sand, clayey soil and ballast. The study of frictional characteristics was conducted by a specially designed direct-shear apparatus.

DESCRIPTION OF SHEAR APPARATUS

A special Direct Shear apparatus was designed to perform the experiments and investigate the frictional behavior. Most elements of this apparatus were made of aluminum. As shown in Fig. 1-1, the Screw Jactuator drives the horizontal load and displacement. The jack has a driving capacity of 10 tons, the worm gear ratio is 24:1, 50.667 turns of worm for 1 inch raise, starting torque at full load is 202 kg-cm (175 lb-in.), and the efficiency of jack ratings is 42%.

The Jactuator is mounted to the chassis, (Fig. 1-24), through frames that consist of aluminum plate and angle section steel. The Jactuator itself is driven by a gear motor (Fig. 1-2). The input shaft of screw Jactuator is 2.54 cm (1 in.) and the output shaft of gear motor is 1.9 cm (0.75 in.), hence a Morse coupling with different bore holes at each side is used to couple the gear motor and the Jactuator.

The gear motor is mounted to the base plate (Fig. 1-27) of the chassis. To control the speed range of the gear motor, a control box (Fig. 1-3) is used. This model has an extended speed range potentiometer in addition to the basic speed range potentiometer. Turning the basic speed range potentiometer full clockwise activates the extended speed range potentiometer. Full rated torque is delivered over the basic range and full rated horsepower over the extended speed range.

The magnitude of the horizontal load which is driven by the Jactuator is calibrated through a loading cell, (Fig. 1-4). The strain of the load cell indicates the horizontal load applied. The load cell connects the Jactuator and the saturation tank, (Fig. 1-6). The saturation tank has a wall of 6 mm (0.25 in.) thick aluminum plate and a 19 mm (0.75 in.) thick base plate. The sidewalls of the shear box are 13 mm (0.5 in.) thick, front and rear walls are 2.54 cm (1 in.) thick. The reason for using more rigid walls in the front and rear is to eliminate or minimize the deflection while shearing.

The shear box consists of upper and lower boxes, (Figs. 1-11 and 1-12). The base of lower box is tied by bolts to the base of saturation tank and will move together with the saturation tank, which the upper box is fixed to the holding bar (Fig. 1-8). Since the saturation tank is connected to Jactuator that moves with a constant rate of speed, the lower box which is tied to saturation tank will also move, and the relative shears will be at the interface of the lower box and upper box. During preparation of the sample the upper box and lower box were tied together. This is done by putting bolts through the guiding groves, Figure 2. The guiding grooves also prevent any sway movement during the shear. The holding bars consist of two angle sections of steel which are supported and tied to the columns (Fig. 1-20). The columns are of square structural steel tube of 32 mm (1.25 in.). An appropriate gap could be provided between the upper box and lower box to eliminate friction during shear, by providing a couple of washers or shims (Fig. 1-5) between the top of the columns and the holding bars before they are bolted together. The thickness of the washers or shims depends on the gap needed to separate any contacts at the interface of the upper box and lower box.

The membrane, (Fig. 1-17), is fixed to the lower box by eight screws. The displacement reading dial gage is mounted to the saturation tank while the arm is touching the upper box. The saturation tank bears on two bearings (Fig. 1-21). The bearings rest on roundways stainless steel shafts (Fig. 1-22). The dual roundway mounting block forming V-shape support system prevents the lateral or sideway movement. Hence, only forward and backward movements are allowed. The shaft are supported by support rails (Fig. 1-23).

A limit switch is attached to each side of columns. This is to prevent any damage if the saturation tank hits the columns. When the saturation tank touches the limit switch it will automatically cut off the electricity to the motor. All these components are put together on an aluminum chassis which consists of 6 channels, the channels providing required stiffness yet light in weight.

To apply the vertical load, this direct shear apparatus can be put into any of the vertical loading machines with necessary spacing, Fig. 2. The load is applied to the specimen via a rigid aluminum top plate, (Fig. 1-13). The apparatus was first used by Budiman (1981).

PROGRAM OF INVESTIGATION

The purpose of the experiments was to test soil-membrane and ballast-membrane interface friction. The membrane was fixed to the lower box to evaluate the coefficient of interface friction. Since the interface friction being the point of main interest, all tests were conducted with shear applied at a very low rate - 0.76 mm/min. (0.03 in/min). As such all the stress measurements can be termed 'effective' at the interface. In this study the word "undisturbed" has been used when the sample is subjected to initial shear stress in one direction; when the stress is reversed from the initial direction the sample is called "disturbed".

Soils Used in the Investigation: One of the soils used in the study was sandy clay synthesized in the laboratory with 50% of kaoline clay, 45% Ottawa sand, 5% bentonite and 16.5% water (by weight). This soil had a specific gravity of 2.66, liquid limit of 40% and plasticity index of 28%. The standard compaction test data porvided an optimum moisture content of 16% and maximum dry density of 17.6 kN/m^3 (112 pcf). The other two materials used in the study were low grade limestone ballast with a dry unit weight of 20.1 kN/m^3 (128 pcf) and Ottawa sand with specific gravity 2.66 and unit weight 18.8 kN/m^3 (120 pcf). The sand is passing sieve #60.

The sandy clay was compacted in the shear box with a standard Proctor Hammer for 89 blows per layer of 3.8 cm (1.5 in.) thick for two layers. The 89 blows were used because it provides an equivalent energy of the standard proctor test. The sandy soil and ballast was vibrated for densification to eliminate the friction between soil used and inner walls, the inner walls were greased.

Testing Procedures: All tests were conducted with the shear box placed into a saturation tank and tied together. In all the tests the impermeable liner - high density polyethelene (156830-SB1-C1-35) supplied by Schlegel Linings Technology, Inc., of Woodland, Tx. was used. When testing saturated sands, water was placed into the saturation tank to allow the sands to saturate. All tests were conducted with vertical normal loads of 68.9, 137.8 and 206.7 kN/m^2 (10, 20 and 30 psi). During shear, the vertical and horizontal displacements and horizontal loads were recorded. The initial shear was applied until a horizontal displacement of about 4.6-5 cm (1.8 in. to 2 in.) was achieved. This meant the completion of test for "undisturbed" sample. The shear direction was then reversed to study "disturbed" conditions.

TEST RESULTS

Four series of tests were conducted on dry sand, saturated sand, sandy clay and ballast in "undisturbed" and "disturbed" conditions to evaluate the interface friction with a high density polyethelene geomembrane. The tests results (Figs. 3 and 4) indicate that the friction between dry sand and moist sand is mobilized (peaked) at a relatively small displacement less than 50 mm (0.2 in.) in the undisturbed state. The frictional resistance is related to the normal stress as well. The higher the normal stress, the higher the peak strength. The residual strength does not vary significantly from the peak strength except at higher normal loads. The frictional strength of the saturated sand-membrane interface is lower than that of dry sand due to lubrication effects.

The tests on "disturbed" dry and saturated sand indicates that the interface friction are lower and relatively more displacement are needed to achieve peak strength for dry sands, but for saturated sands the strength of the disturbed samples reach close to that of undisturbed sample. The curves of "disturbed" saturated sand samples are crossing each other. It may be noted that in the undisturbed state the moist sand requires higher strain (than dry sand) for full mobilization of the strength. During the test in undisturbed conditions, higher hornal stress caused higher strain (in the central portion of membrane because at the edges it is clamped). Therefore, the samples which underwent higher strain when shear - loaded in reverse direction required a small shear force.

In the tests with sandy clay interface (Fig. 5), as noted in other cases, the "undisturbed" condition provides higher strengths -- peak and residual. The test in "disturbed" state did not bring any peaks and the interface strengths were lower at all levels of normal load.

The tests on membrane-ballast interface provide similar results qualitatively, but different quantitatively. The interface strengths are higher in comparison to that of other soils. The relative displacements for full mobilization are also higher. At the highest normal load of 206.7 kN/m^2 (30 psi), the peak was not seen until 3.8 cm (1.5 in.) movement. (See Fig. 6)

In the "disturbed" state, peak was not clearly visible for all three normal loads. For attaining some interface friction as in the "undisturbed" state, higher displacement was needed. As in other tests, the membrane moves with the lower box at the edges where it is clamped, while at the center, the friction of ballast membrane interface resists the movement of the membrane until a threshold shear force is mobilized. The relative displacement of the membrane to the lower box is more at the central portion and less near the edges, thus making the lateral strands deformed in a curved shape at the threshold stress. The test in "undisturbed" state begins with the membrane in a curved shape and the shear force will invert the membrane shape in the opposite direction. This provides an explanation of flatter curves for tests in a "disturbed" state. (See Fig. 7)

ACKNOWLEDGMENTS

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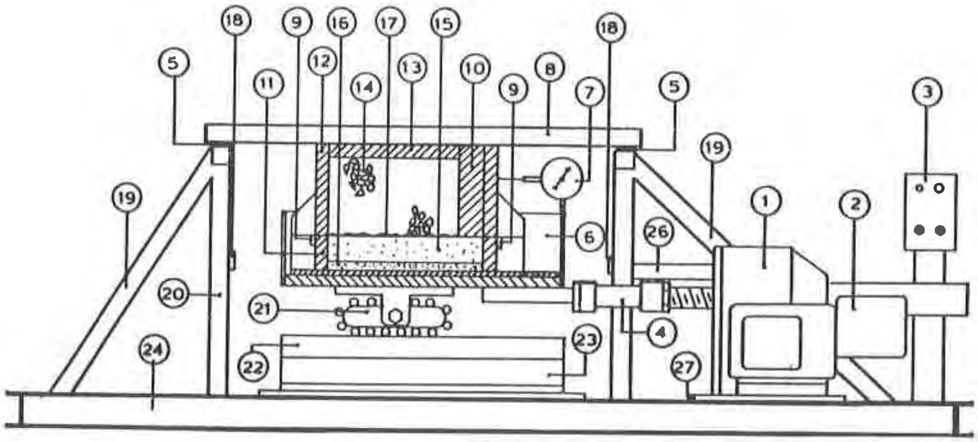

Figure 1. Direct Shear Apparatus

Legend for Figure 1.

1. Screw Jactuator
2. Gear motor
3. Control box
4. Loading cell
5. Washers/shims
6. Saturation tank
7. Dial gage
8. Holding bars
9. Clamps
10. End plate
11. Lower box
12. Upper box
13. Top plate
14. Ballast or soil
15. Soil or ballast
16. Sand layer
17. Fabric
18. Limit switch
19. Bracings
20. Columns
21. Bearings
22. Shafts
23. Support rails
24. Chassis
25. Guiding groove
26. Bars
27. Base plate

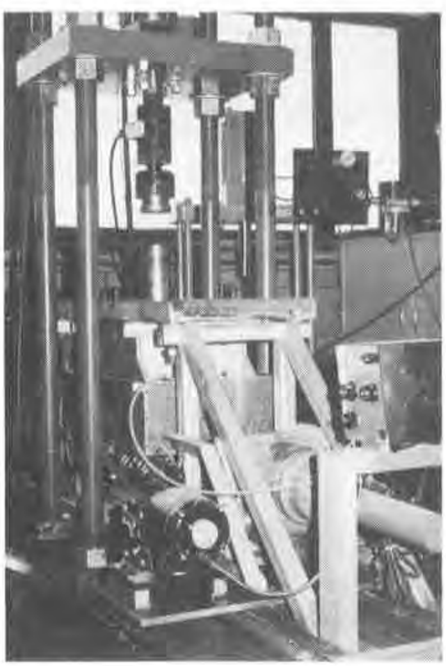

Figure 2. Direct Shear Apparatus in Vertical Loading Machine.

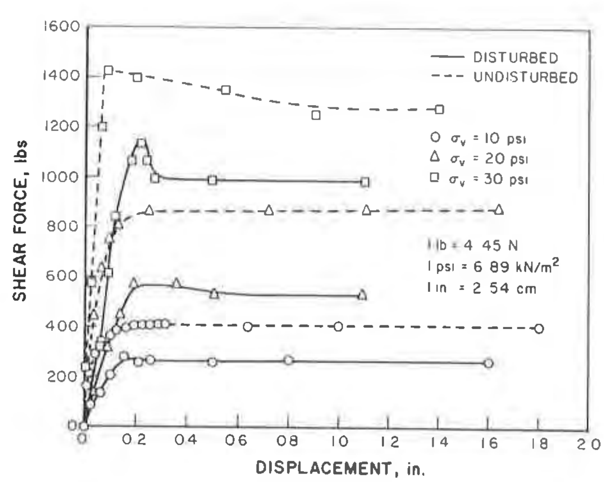

Figure 3. Shear Force vs Horizontal Displacement for Dry Sand-Membrane Interface

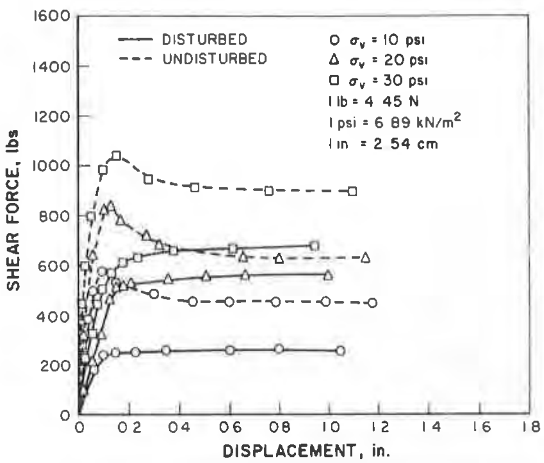

Figure 4. Shear Force vs Horizontal Displacement For Saturated Sand-Membrane Interface

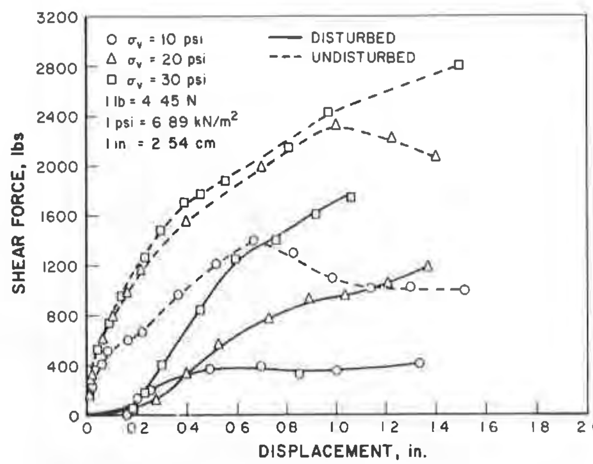

Figure 6. Shear Force vs Horizontal Displacement For Ballast-Membrane Interface

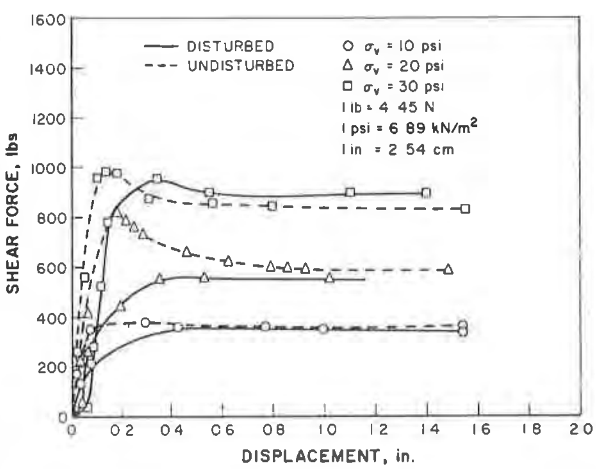

Figure 5. Shear Force vs Horizontal Displacement For Sandy Clay-Membrane Interface

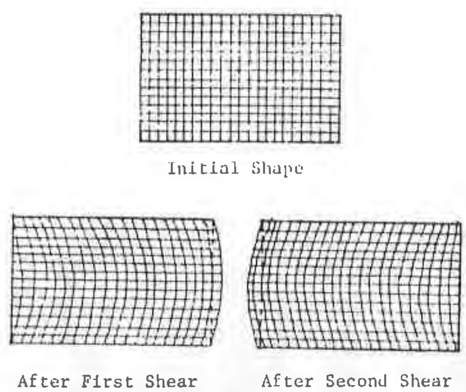

Figure 7. Membrane Deformation

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Experimental Friction Evaluation of Slippage Between Geomembranes, Geotextiles and Soils

A common failure mechanism of geomembrane lined side slopes of impoundments and reservoirs is by slipping of components within the liner system or of the cover soil. While safe design is indeed possible, the friction values between individual components are required and are essentially not available to date. This study focuses on presenting a test methodology and data base for friction values between three soil tyes, four geomembranes and four geotextiles. Seen is that the values vary widely in accordance with the materials being used. Mobilized friction values from 60% to 100% of the intrinsic values of the material by itself were determined. Details of the tests and individual values are reported.

INTRODUCTION

The usual design goal of excavated or built-up impoundments is to build the side slopes as steeply as possible. This is particularly true at sites of high water table or in containing large volumes with respect to the available land area. To eliminate, or minimize, the loss of the contained liquids or generated leachates it is usually necessary to line both the bottom and sides of such impoundments. For the purpose of this study, the primary liner will be assumed to be a flexible membrane liner (FML), i.e., a geomembrane, made from polymeric materials into relatively thin sheets, 20 mils to 100 mils thick, and adequately seamed together wherever joints are necessary. In some circumstances it is necessary to sandwich this geomembrane between one or two geotextiles, which are porous woven or nonwoven fabrics that serve the following functions:

The geotextile underliner:

- prevents underlying stones and sharp objects from puncturing the geomembrane
- provides a clean working surface for placement of the geomembrane and the making of field seams
- provides some support (reinforcement) over weak areas in the subgrade
- acts as a lateral transmitter of water and gas which may come up from the subsurface soil beneath the geomembrane -- in this case, one must select a bulky, needled nonwoven geotextile which possesses adequate transmissivity.(1,2)

The geotextile overliner:

- protects the geomembrane from puncture of stones in the cover soil or in the landfilled material itself
- provides some load spreading capability for heavy objects in the landfill, i.e. reinforcement
- protects the geomembrane from ozone and ultraviolet attack for cases where the liner system is not soil covered

Usually, but certainly not always, the sandwiched geomembrane liner is covered with a layer of soil. This cover soil should be select material with good gradation and strength characteristics so that it can be easily placed and compacted in as thin a layer as possible. Usually its thickness is from 30.48 to 91.44 centimeters. In many cases it serves a dual role as protection to the liner system and as a leachate collection system containment media, i.e., pipe underdrains are placed within it.

With the above thoughts in mind, the general cross section of the side slopes of lined impoundments containing liquids and/or solids is presented in Figure 1. Note that the following alternates for the liner system can be used:

- geomembrane alone (GM)
- geomembrane plus cover soil (GM/CS)
- geotextile underliner plus geomembrane (GTU/GM)
- geotextile underliner plus geomembrane plus cover soil (GTU/GM/CS)
- geotextile underliner plus geomembrane plus geotextile overliner (GTU/GM/GTO)
- geotextile underliner plus geomembrane plus geotextile overliner plus cover soil (GTU/GM/GTO/CS)

Upon the decision as to the choice of above liner system and a knowledge of the depth of the impoundment, the critical variable becomes the slope angle and the general stability of the lined side slopes.

The analysis of slope stability for both homogeneous and heterogeneous soil masses is well developed in geotechnical engineering practice. However, the analysis of stability when flexible synthetic sheets (geomembranes and geotextiles) under tension are placed on the slope face is still in its infancy. This situation falls into the general classification of soil-structure interaction problems. The three major elements necessary to extend organized slope stability analysis into membrane-lined impoundments are:

(a) Data on limiting shear strength along interfaces between soils, geomembranes and geotextiles.
(b) Effect of tension in the liner system (provided for by the anchor trench) on the overall slope stability.

(c) Effect of slippage between soils, geomembranes and geotextiles and its relationship to the general stress-strain behavior of the materials.

This paper is a report of experimental work that concentrates primarily on item (a). It extends published data on friction between geotextiles and soils, and presents new data on frictional behavior between soils and geomembranes and also between geotextiles and geomembranes. Item (b) is more analytical than experimental, and is only considered briefly herein. However, a review of analytical methods is included since it provides a basis for further work in this area with the experimental data obtained and presented. A brief discussion of item (c) is included, but it is actually a summary of a more extended report soon to be available.(3,4)

ANALYSIS OF STABILITY OF LINED SIDE SLOPES

There are two major areas of concern with respect to stability of the side slopes of a lined enclosure: slope stability of the soil subgrade, natural formations and compacted embankment under the liner and slippage within the liner system consisting of geomembrane, geotextiles and cover soil.

Analysis of general slope stability involves determination of the factor of safety against shear failure along an undefined critical surface, usually assumed to be circular. Design centers around the selection of the appropriate geometry, materials and other measures to obtain the desired factor of safety. The driving force for most slope failures is the applied stress along a continuous surface that results from body and surcharge forces. The resistance is provided by the cohesive and frictional strength of the soil and other materials along a slip surface. Schematically, this type of a failure is shown in Figure 2(a). Proper design to prevent this situation from occurring is well within the state-of-the-art of geotechnical engineering. It is, indeed, an important consideration but it is beyond the scope of this paper.

Slippage between the various components of the liner system, however, is of very real concern and is the general thrust of this study. It is shown schematically in Figure 2(b) for both the geomembrane liner system and the cover soil over the liner. The design procedure in this case of a liner failure along a known surface is straighforward once the values of friction are known between the various interfaces involved. Assuming these values are known a force polygon can be drawn consisting of the following items which are shown and illustrated in Figure 3.

- The weight of the liner system and cover soil (if present); which act vertically downward (W_A and W_{NB})
- The tensile strength of the liner system (geomembrane plus geotextiles, if present); which acts along the slope and is eventually mobilized in the anchor trench (T)
- The possible resistance to failure of a small wedge of cover soil at the toe of the slope; which also acts along the slope (E_A and E_{NB})
- The unknown frictional forces (F_A and F_{NB}) which act at different friction angles (δ_A and δ_{NB}), where the friction angle δ_A is the minimum value between any interface in the liner system and must be determined experimentally (this item is the specific focus of this paper) and the friction angle δ_{NB} which is completely within the cover soil and is generally equal to the friction angle of the soil.

This type of problem is best solved by assuming a factor of safety and applying it to δ_A and δ_{NB}. A force polygon for the neutral block is drawn to obtain a trial value for E_{NB}. This value is then made equal to E_A and is used in construction of a force polygon for the active zone. If closure of the active zone polygon is obtained, the initially assumed factor of safety is correct. If not, successive trials using different values will be required until a graph can be drawn to accurately assess the actual factor of safety. Usually three or four trials are necessary.

Critical in this design process, and not available in the required form as far as the authors are aware, is the value for interface friction between components of the liner system, i.e., δ_A values. The design value will be the minimum value between any component of the liner system; soil, geomembrane or geotextile. It is, of course, material dependent so that each specific material will have to be experimentally evaluated. This paper describes such experiments and presents data on a wide range of soil types, geomembranes and geotextiles.

TEST DETAILS AND PROCEDURES

A modified direct shear apparatus was used to evaluate friction values between soils, geomembranes and geotextiles in various combinations. In this type of test, the two materials being evaluated were placed in a split shear box, as shown in Figure 4. The shear box used had dimensions of 10.16 X 10.16 centimeters. For soil testing the depth in each part of the shear box was 2.54 centimeters of soil. For composite soil and geomembrane or soil and geotextile testing, the soil was placed in the upper half of the shearbox and the fabric was in the lower half. Rather than laying loose in the lower half of the shear box, the geomembrane or geotextile was firmly attached to a plexiglass block so that wrinkling could not occur. For geomembrane on geotextile testing, each material was attached to a separate plexiglass block and placed opposing one another in the two parts of the test device. All materials were tested in saturated condition, with the soils being placed at about 90% of their maximum density (ASRM D-698). This apparatus and techniques appears to be easier to perform than other shear box tests and be more representative of field boundary conditions than pullout tests, see Collios, et a.(5)

The normal stress range used in these tests was varied from 2.0 psi to 15 psi. These values are somewhat lower than in normal geotechnical testing but probably better reflect the low normal stresses that shallow cover soils impose on typical liner systems. The shear phase of the test was deformation controlled at a displacement rate of .127 millimeters/min. This low deformation rate assured complete dissipation of pore water pressures during the test. Typical data that resulted from these tests are shown in Figure 5. Here a set of different types of geomembranes were each tested with a concrete sand (sieved through a #10 sieve) at 6.0 psi of normal stress. Typical elastic-plastic response curves are observed, each having a well defined maximum value of shear stress.

Upon testing these same sets of materials at different normal stresses one can plot the peak shear stress versus applied normal stress on Mohr's stress space, as shown in Figure 6. Note that all failure envelopes pass through the origin attesting to the fact that there is no (or non-measurable) cohesion in the soils tested nor adhesion between these soils and the fabrics evaluated. (This would not have been the case if fine grained soils such as clays or cohesive silts had been used). The slope of these curves, often presented as an angle, is the desired value for design purposes. In all cases in this study, the response was linear and the data spread in a given locus of points was nominal.

After each shear failure, the direction of deformation was reversed, and the test repeated. The purpose of this exercise was to indicate residual friction angles where membrane tension is alternately increased and reduced as the level of a storage lagoon changes. Such reversals of strain direction may tend to align particles along the shear plane, and reduce slip resistance. However, the difference between initial and repeated shear strengths was negligible in all cases.[3]

MATERIALS TESTED AND RESULTS

Three granular soil types were used in these tests:

(1) Ottawa sand (SP) with d_{10} = 0.42 mm; CU = 1.9 and rounded particle shapes.
(2) Concrete sand (SP) with d_{10} = 0.20 mm; CU = 2.6 and angular particle shapes.
(3) Mica schist silty sand (SM) with d_{10} = 0.057 mm; CU = 5.1 and angular particle shapes.

Thus the three soil types selected give a contrast in particle shape, size and uniformity. They are limited however, to granular soils with essentially no plasticity.

Four types of geomembranes (using five separate surfaces) were used in these tests. They were all tested in their manufactured directions.

(1) High density polyethylene (HDPE) which was 20 mils thick and can be characterized as being stiff, hard and smooth as far as physical or frictional characteristics are concerned.
(2) Ethylene propylene diene monomer (EPDM) which was 30 mils thick and can be characterized as being flexible, soft and smooth.
(3) Polyvinyl chloride (PVC) which was 30 mils thick and characterized as being of medium stiffness and hardness and rough on one side while smooth on the other side. Both sides were used during these tests.
(4) Chlorosulfonated polyethylene (CSPE) which was reinforced with a fabric scrim and was 36 mils thick. It is characterized as being of medium stiffness and hardness, but was of wavy roughness due to the laminated 10 x 10 scrim reinforcement contained within it.

Four types of geotextiles were used in these tests which represented each of the general manufacturing classifications of these materials.[6] They were all tested in their manufactured directions.

(1) Woven monofilament polypropylene fabric (Carthage Mills Polyfilter X) which is characterized as being a thin, stiff fabric with a relatively high percent open area as far as physical or frictional characteristics are concerned.
(2) Woven silt film (tape) polypropylene fabric (Mirafi 500 X) which is characterized as being a thin, flexible fabric with a low percent open area.
(3) Nonwoven heat set polypropylene fabric (duPont 3401) which is characterized as being a thin, flexible fabric with a relatively low open area.
(4) Nonwoven needled polypropylene fabric (Crown Zellerbach 600) which is characterized as being a compressible, thick, bulky, very flexible fabric with a relatively high open area.

These three soil types, four geomembranes types and four geotextile types were tested within their own categories and against one another in the manner described in the previous section. The results are given in Table 1 in two ways. The principal information (for design purposes) is given as angular values of friction angle; "ϕ" values for the soil by itself and "δ" values for the composite behavior. In parenthesis is given the relative amount (for comparison purposes) of mobilized soil strength that the geomembrane or geotextile gives, i.e.,

$$E = \frac{\tan \delta}{\tan \phi}$$

where

E = efficiency ratio
$\tan \delta$ = tangent of soil to material friction angle
$\tan \phi$ = tangent of soil friction angle, where
$\tau = c + \bar{\sigma}_n \tan \phi$
c = cohesion (zero for these granular soils)
$\bar{\sigma}_n$ = effective normal stress

Table 1 - Summary of Friction Angle and Efficiencies (in Parentheses) For Soils, Geomembranes and Geotextiles Testing in this Study

(a) Soil to Geomembrane Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
EPDM	24° (.80)	20° (.71)	24° (.92)
PVC (Rough)	27° (.90)	-	25° (.96)
PVC (Smooth)	25° (.83)	-	21° (.81)
CSPE	25° (.83)	21° (.75)	23° (.88)
HDPE	18° (.60)	18° (.64)	17° (.65)

(a) Soil to Geotextile Friction Angles

Geomembrane \ Soil	Concrete Sand (ϕ = 30°)	Ottawa Sand (ϕ = 28°)	Mica Schist (ϕ = 26°)
CZ 600	30° (1.00)	26° (.93)	25° (.96)
Typar 3401	26° (.87)	-	-
Polyfilter X	26° (.87)	-	-
500 X	24° (.80)	24° (.86)	23° (.88)

(c) Geomembrane to Geotextile Friction Angles

Geotextile \ Geomembrane	EPDM	PVC (R)	PVC (S)	CSPE	HDPE
CZ 600	23°	23°	21°	15°	8°
Typar 3401	18°	20°	18°	21°	11°
Polyfilter X	17°	11°	10°	9°	6°
500 S	21°	28°	24°	13°	10°

INTERPRETATION OF RESULTS

Table 1, parts "a" and "b" show the results of the direct shear tests for friction between various soils and synthetic materials in terms of friction angle (ϕ or δ)

and relative efficiency (E). It can be seen that the friction between all soils and the geotextiles or geomembranes is less than that of the soil itself. Consequently, soil to fabric friction governs the design of a slope, recall Figure 3. Soil to geotextile friction generally exceeds soil to geomembrane friction. Therefore, placement of a geotextile over or under a liner (as discussed in the introduction) will tend to allow a steeper slope, provided that both fabrics are securely anchored. If the anchor fails, then the safe slope angle will obviously be decreased. Part "c" of this table shows that geotextile to geomembrane friction is relatively low and depends greatly on the particular type of geomembrane being used.

Certain additional trends can be inferred from the data of Table 1 that allow prediction of the behavior of other materials not represented in the testing program. The three soils were selected to indicate the influence of particle angularity and gradation. For instance, EPDM is a smooth, flexible and surficially soft material. The friction angle with angular soil is higher than that with rounded soil. Here, the higher friction resulted from surface penetration, and surface scratches in the geomembrane were noted with the concrete sand tests. A high relative efficiency (92%) was obtained with the well graded silty sand probably due to the high contact area between the soil and the geomembrane and the surface roughness induced by distorting, but not piercing, the soft surface. Thus, it is worthwhile to use angular and well graded cover soils on soft membranes.

In contrast, the stiff, hard and smooth HDPE was fairly insensitive to soil type. Surface roughness is not induced by normal stress on the HDPE to soil interface, and low friction angles and relative efficiency indexes result. It would appear that it is necessary to place and anchor a geotextile over the material in order to build a steep slope with HDPE.

As expected, the angular soil readily penetrated into most of the geotextiles, and the relative efficiencies of all geotextiles and particularly, the needled-punched fabric, are particularly high. One generalization that can be made is that is it easier to estimate soil to geotextile friction for nonwoven than woven fabrics. There are a wide range of fabric openings in the non-wovens, whereas the woven geotextiles have a more regular pattern and limited opening size range. Hence, while the specific gradation of one soil type may allow considerable fabric penetration, a slightly coarser soil will not interlock as well. However, this analysis does not take into account the tensile strength or puncture resistance of woven materials; parameters which may be of equal importance in a particular situation.

Certain additional trends are evident in part "c" of Table 1. The pliable EPDM readily takes on the imprint of the opposing geotextile during conducting of the test, producing a surface roughness resulting in improved behavior. Hence, special care must be taken to assure that an overlying or underlying geotextile is securely anchored. The relatively stiff woven monofilament geotextile, substantially interacts (mechanically) with only the EPDM. The effect of geotextile stiffness is particularly evident with the scrim-reinforcement CSPE, such that the relatively stiff monofilament geotextile imprints the CSPE material around the reinforcement grid, but does not deform sufficiently to contact much of the soft material below and between the grid.

It must be noted, however, that the selection of a liner system (geomembrane, geotextile and soil cover) is dependent not only on the above friction behavior but also on the basis of chemical resistance to the impounded materials, availability and cost. As noted in the introduction, geotextiles are employed with liners for purposes other than friction. Finally, the subgrade soil is usually that which is native to the site. Consequently, the cover soil is often the only material of concern which can be selected largely on the basis of its mechanical properties.

SUMMARY AND CONCLUSIONS

Proper design of geomembrane lined side slopes is necessary whenever slopes greater than approximately 4 (horizontal) on 1 (vertical) are contemplated. Since this usually is the case (except in areas where large land areas are available), one must consider at least two different failure mechanisms. One is a general slope stability failure of a large mass consisting of the liner system and subsoils which is an area beyond the scope of this paper but well within the state-of-the-art. The other is linear slippage between individual components of the liner system or of the cover soil. This latter aspect was the concentration in this study. Elements of the general design were presented illustrating the need for experimental data on friction between soils, geomembranes and geotextiles. Toward supplying this needed data base, a modified direct shear test was used on a variety of materials of different interfaces.

Three soil types, four geomembranes and four geotextiles were evaluated, where the geomembranes mobilized from 60% to 86% of the soil friction and the geotextiles mobilized from 80% to 100% of the soil friction of those soils tested. Friction values for geomembranes on geotextiles were relatively low, suggesting the need for careful choice between materials when used in a composite manner and high assurance of anchor integrity. The need for additional data in this regard seems justified.

Concerning additional investigations on this subject, the lack of data using soils with cohesion is obvious. Indeed, such soils are encountered as subgrade materials, and their shear strength values (cohesion and friction) should be evaluated. Regarding design much remains. Included here was a limit equilibrium method of analysis. Needed is a method which is based on the entire stress vs. strain behavior of the materials involved. Work is currently ongoing in this regard.

ACKNOWLEDGEMENTS

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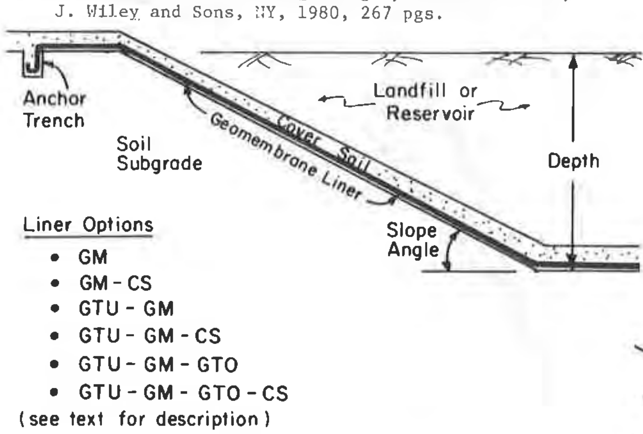


Fig. 1. - Typical Cross Section of Impoundment or Reservoir Slope with Geomembrane Liner System and Cover Soil

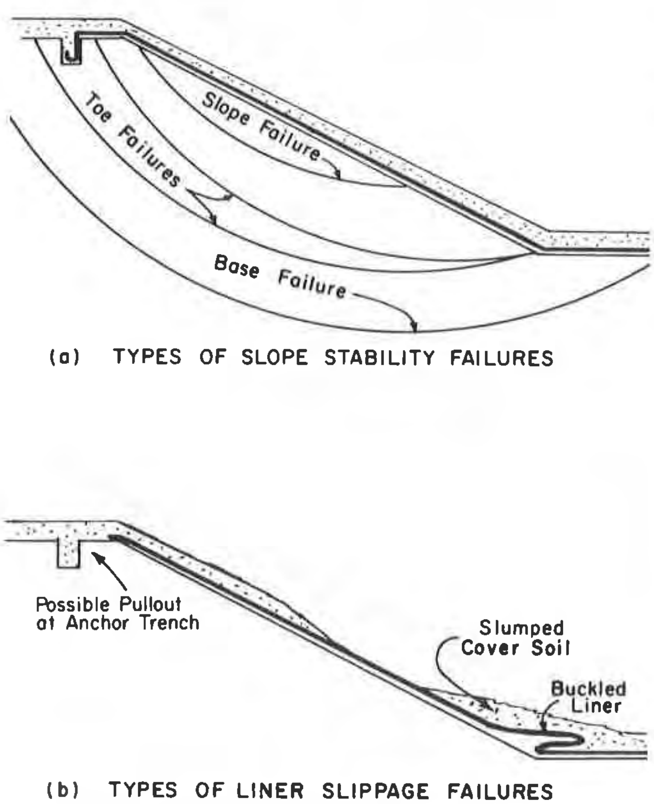


Fig. 2. - General Types of Failures of Lined Impoundments or Reservoir Slopes

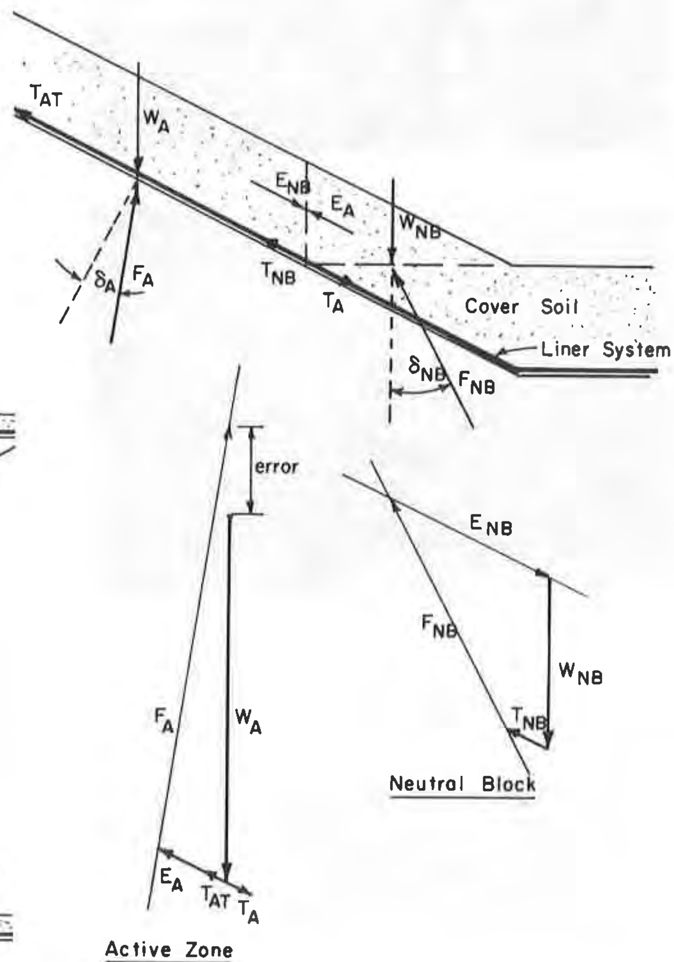
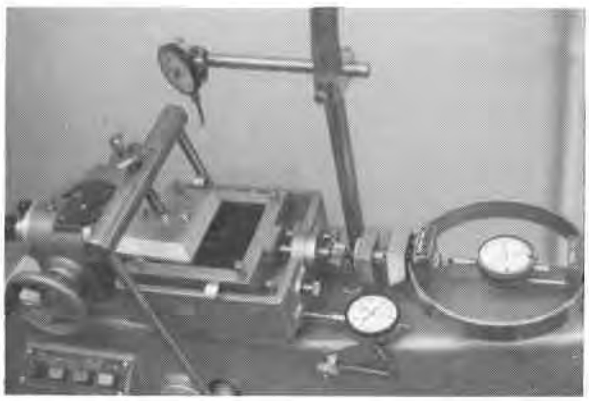
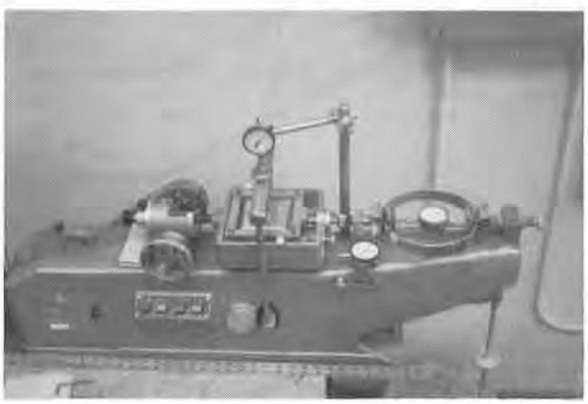


Fig. 3 - Design Details of Geomembrane Liner and Cover Soil Under Incipient Slippage Failure with Corresponding Force Polygons





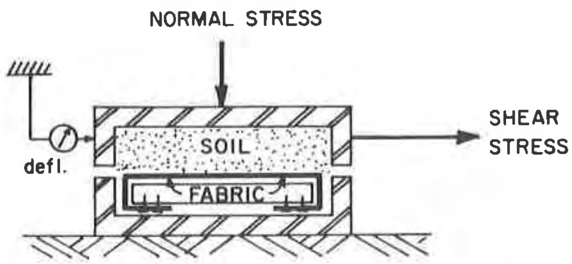


Fig. 4 - Photograph and Schematic Drawing of Direct Shear Device Used to Determine Friction Values in this Study

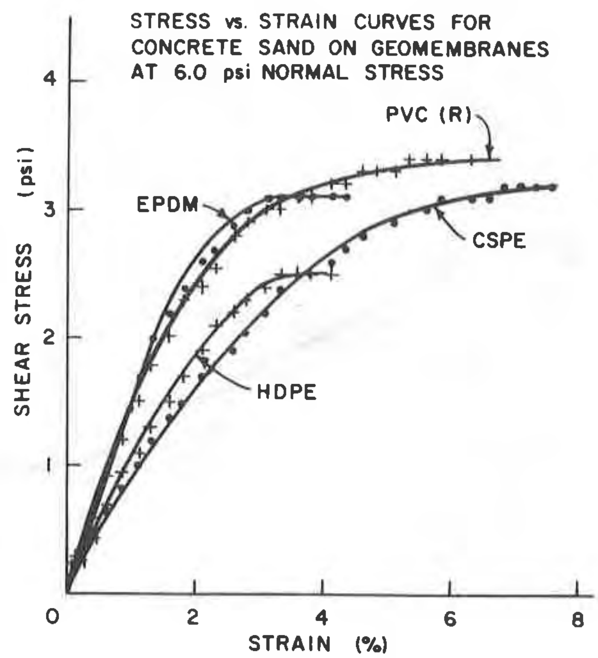


Fig. 5. - Typical Shear Stress vs. Strain Curves Generated in this Study. Illustrated is Concrete Sand on Four Geomembranes at 6.0 psi Normal Stress (Values Include 0.50 psi Machine Tare)

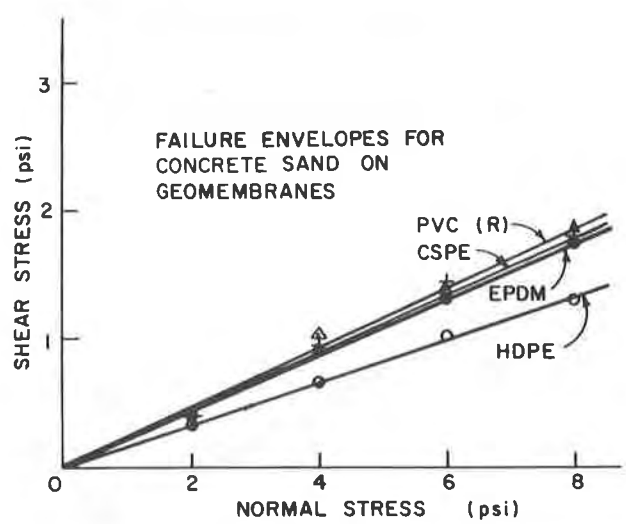


Fig. 6. - Failure Envelopes for Concrete Sand on Four Geomembranes Where Resulting Friction Angles Range from 18° to 27° (Figure 5 Values are Included here at 6.0 psi Normal Stress)

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Work-to-Rupture—Using the Stress-Strain Relationship to Optimize Scrim Reinforcement of FML

The introduction of thermoplastic elastomers as FML requires a reinforcing scrim to improve the modulus or "green strength" of the uncured membrane.
Selection of the "proper" reinforcement is important to the overall performance of the FML. Improvement in Tensile Strength, Tear and Puncture Resistance must be weighed against a reduction in Elongation, strike-through and ply bond.
The stress-strain curves, generated by a tensile tester according to ASTM D-751, provide a means of comparison of various scrim reinforcements. The total area under the load-elongation (stress-strain) curves from the start to the point of failure (rupture) gives a readily measured comparison. This area, expressed in inch-pounds, represents the total capacity of the sample to absorb stress.

In a recent article on Geomembrane Liner Design, our Co-chairman, Jean-Pierre Giroud, made the following statement:

> "Many designers have a tendency to consider that the stronger is always the better, and they apply this simplistic approach to geomembranes. In fact, there are cases where geomembranes with high tensile strength are recommended, and cases where geomembranes with large elongation are recommended." (Ref.1).

The relationship between tensile strength and elongation is well defined in a single test by the ASTM D-751 Grab Tensile Test for reinforced geomembranes (Ref.3). A 4" x 6" sample is clamped with 1" square jaws, centered on the narrow direction and separated by 3" at the start of the test. Although only the threads between the jaws (1" width) receive the full strain as the jaws separate to the thread breaking point, the remaining unbroken threads on each side (1-1/2" each side)

Flexible Membrane Lining Materials (or Pre-fabricated Geomembranes) are generally available both as unsupported sheet and as reinforced sheet. The reinforced sheet usually encapsulates one or more plies of an open weave fabric or scrim between two or more plies of rubber (or plastic). The physical properties of the unsupported sheet are changed by the reinforcement, and different test methods are used to define the physical properties of the reinforced sheet.

Several types of reinforcement are available, most of them being an open weave of filament yarns. The thread count (threads/inch) and thread weight (denier) determine the strength of the scrim, as well as the "window" area or openings between threads. As the thread pattern and weight increase, so does the strength of the fabric, but the openings between the threads decrease accordingly.

Most reinforced membranes rely on the rubber-to-rubber bonding through the openings in the woven fabric to achieve good ply adhesion and to resist delamination. The rubber is not chemically bonded or fused to the fabric directly, as is the case with coated fabrics such as upholstery, tarpaulins, etc.

Therefore, increasing the strength of a flexible membrane lining material with a reinforcing fabric leads to a corresponding decrease of the ply lamination due to reduced strike-through area. Other properties are also affected by the incorporation of a reinforcing fabric - some increase, some decrease. Selection of the reinforcement that will optimize the geomembrane's performance in a given installation requires a comprehensive analysis.

restrict the elongation of the rubber. Continuing the jaw separation to the point of rubber rupture (failure of the liner), completes the picture of the reinforced geomembrane's ability to absorb stress by including elongation.

This test gives a two-stage result - with an initial peak at fabric break, followed by a drop-off and a second peak at the rupture of the membrane. It is similar to the tensile performance of crystalline unsupported materials, such as HDPE, where the yield point corresponds to the fabric break, and the breaking point being the point of rupture in both cases.

The standard testing procedure for industrial filament yarns and fabrics used in the reinforcing of geomembranes is ASTM D-885 (Ref. 3). One of the tests used to evaluate these filament yarns is "work-to-break", which is defined as "the total energy required to rupture a specimen during a tensile test. Work-to-break is proportional to the area under the load-elongation curve from the origin to the breaking point and commonly is expressed in inch-pounds (centimeter-grams)".

Applying the same definition to the ASTM D-751 Grab Tensile Test from the origin to the point of rupture gives a measurement in inch-pounds (centimeter-grams) of the total energy required to rupture the geomembrane. "Work-to-rupture" of the geomembrane enables the designer to optimize the balance between tensile strength and elongation in a single, repeatable test that is already part of specification requirements. (Ref. 5).

To illustrate the idea of "work-to-rupture", tests of the three commonly used fabric reinforcements for geomembranes were run, comparing Type 1-light weight, Type 2-medium weight, and Type 3-heavy weight laminates in 45 mil thickness (the only common thickness to all types in CSPE Geomembranes). The results are illuminating (tests were run in both the warp and fill direction, with the lightest weight fabric absorbing the greatest energy).

The same approach can be used to optimize the geomembrane reinforcement selection in the Bonded Seam Strength-Shear, and in the Puncture test. Both are essentially tensile-elongation tests to the point of rupture. "Work-to-Rupture" analysis of the area under the tensile-elongation curve indicates the total energy absorbed.

Bonded seam strength-shear uses the same ASTM D-751 Grab Method, as the Breaking Strength test, but with a sample that is 4" wide x (6" plus bonded seam width) long (Ref. 4). Puncture testing by Federal Test Method Standard 101C, Method 2031 (Ref. 6) uses a sample that is 4" wide x 12" long. Both ends are clamped together around a tetrahedron point, and jaw separation forces the looped material over the tetrahedron point.

These same principles have been applied to fiber-reinforced concrete to create a new "Toughness Index" based on the load-deflection curve obtained in fracture tests (Ref. 2). The test is run to twice the deflection at first crack, then the area under the load-deflection is stated as a percent of the theoretical area if no crack occurred (minimum/maximum ranges from 25% to 100%). The area under the curve measures the capacity of the specimen to absorb stress prior to failure.

To summarize, a reinforced geomembrane's capacity to absorb stress is a function of both breaking strength and elongation. By measuring the area under the load-elongation curve from start to rupture, it is possible to compare the relative toughness of different strength reinforcements, and to optimize the balance between strength and elongation for the conditions of use.

This technique is valuable in Bonded Seam Strength tests and puncture tests, as well as for Grab Tensile tests. In each case, the pounds of force required to rupture the specimen is tempered by the elongation permitted by the reinforcement. "Work-to-rupture" is a valuable tool for the Design Engineer, as he compares the available reinforcements against the specific requirements of the installation.

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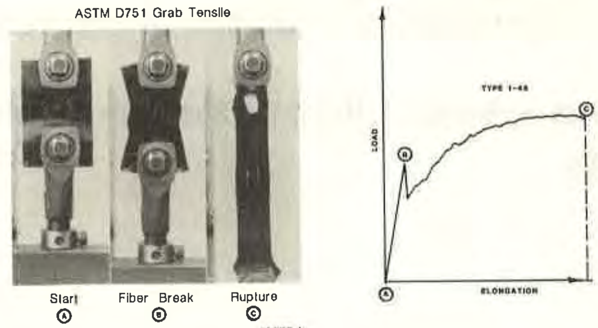

FIGURE 1.

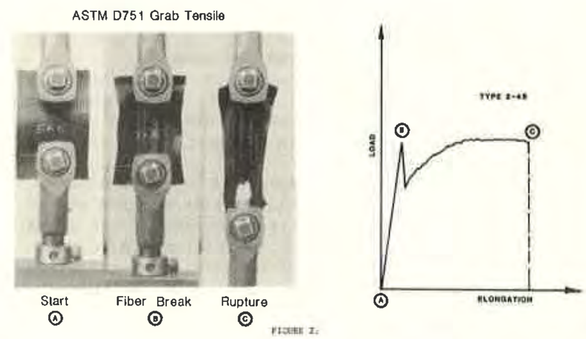

FIGURE 2.

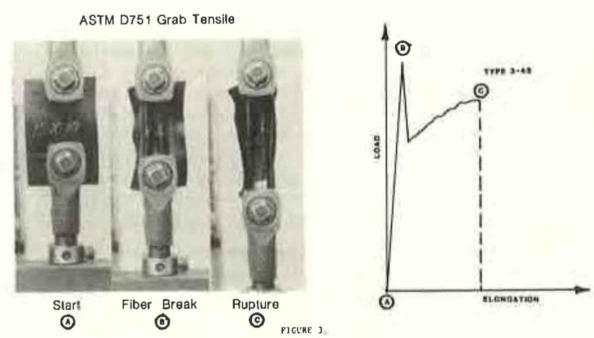

FIGURE 3.

	Type 1-45	Type 2-45	Type 3-45
B Fiber	145	185	280
B %	22	25	21
C Rupture	200	180	230
C %	245	160	115
W-T-R	1111	513	718

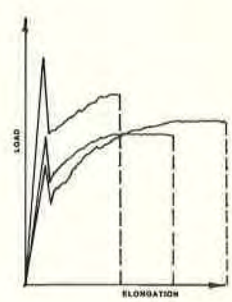

FIGURE 4.

International Conference on Geomembranes
Denver, U.S.A.

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Evaluation of Geomembrane Seams Exposed to Selected Environmental Conditions

In an attempt to learn more about the strength and durability of presently available seaming systems, the Municipal Environmental Research Laboratory of the United States Environmental Protection Agency (USEPA) has funded research with the U.S. Bureau of Reclamation (USBR) to evaluate geomembrane liner seams exposed to selected simulated environmental conditions. A total of 37 combinations of reinforced and nonreinforced polymeric sheet materials joined by various seaming methods are being subjected to chemical solutions, brine and water immersion, freeze/thaw cycling, wet/drying cycling, heat aging and accelerated outdoor aging. Evaluation of the effects of environmental exposure will include before and after mechanical testing of seams (dynamic load, in shear and peel, and static dead load) and Gas Chromatography (GC) or Gas Chromatography/Mass Spectrometry (GC/MS) analysis of the polymeric materials. Immersion tests were started in the fall of 1983, however, at the time of this printing no comparative data were available for publication.

INTRODUCTION

An important aspect of hazardous waste treatment, storage, and disposal is the prevention of surface- and ground-water contamination by fluids containing hazardous constituents. Over the last few years, polymeric geomembranes have increasingly been specified and used in the construction of hazardous waste containment facilities, such as landfills and surface impoundments. A considerable number of laboratory tests and pilot-scale studies are currently being conducted to assess the effects of chemical waste products on the strength and durability of polymeric membrane liners; however, little has been done to assess the performance of the various factory and field seams used in joining the manufactured roll goods in the factory and the fabricated panels in the field.

The objective of this study is to evaluate bonded seams of commonly used geomembrane materials and the bonding methods employed. Tests will be carried out in an attempt to detect physical changes in these seams when exposed to selected chemicals and simulated environmental conditions. It is anticipated that physical property testing and chemical analysis will provide the necessary data to assess the strength and durability of factory and field seams before, during, and after exposure. Such tests and the data gathered therefrom will eventually be used in preparing guidelines for engineers, specification writers, regulatory agencies, geomembrane fabricators, and installers to ensure that quality seams are produced for the intended use.

The project is now in progress and several tasks have already been completed. This paper describes the liner samples which were obtained, the immersion test setup, field seam fabrication, and ultraviolet (UV) exposure testing. Interim results are presented on several nondestructive test methods for evaluating the integrity of factory and field seam bonds. Also included are sections presenting future work in environmental cycling and heat aging of liner seam samples.

RESEARCH PROGRAM

The purpose of this study is to determine the chemical and physical effects of several types of simulated environmental exposure on the seams of various geomembrane materials. These exposures include: immersion of liner samples in six chemical solutions, tapwater and saturated brine, accelerated outdoor aging, freeze/thaw cycling, wet/dry cycling, and accelerated heat aging. After exposure of the samples, the effects will be analyzed by mechanical as well as analytical testing on the seam systems.

Materials Used in the Study

The geomembrane materials which are being tested were obtained from twelve industry representatives who manufacture and/or fabricate geomembranes commonly used in pollution and seepage control. Manufacturers were contacted and asked to participate in the study in an attempt to obtain a broad selection of recommended polymeric types (including reinforced and nonreinforced samples), scrim, and seaming techniques.

Polymer sheet materials submitted by industry for this project included the following:

High Density Polyethylene (HDPE)	30 mil, nonreinforced
	80 mil, nonreinforced
Linear Low Density Polyethylene (LLDPE)	30 mil, nonreinforced
Ethylene Propylene Rubber (EPDM)	40 mil, reinforced
Polyvinyl Chloride (PVC)	30 mil, nonreinforced
Chlorinated Polyethylene (CPE)	30 mil, nonreinforced 36 mil, reinforced

Chlorosulfonated Polyethylene (CSPE)	36 mil, reinforced
Ethylene Interpolymer Alloy (EIA)	30 mil, reinforced

In addition, two or more sources submitted additional samples of PVC, HDPE, CPE, and CSPE representing different manufacturing methods including varying scrim types and seaming methods.

The following factory and field seam types were submitted or field- fabricated for evaluation in the project:

Thermal - hot air
Thermal - dielectric
Thermal - hot wedge
Extrusion - fillet weld
Extrusion - lap weld
Bodied solvent adhesive
Solvent adhesive
Gumtape/cement
Vulcanized

A total of 37 combinations of sheet materials and seaming methods are being evaluated in the study.

Field Seam Fabrication

In addition to factory seams and polymer roll goods submitted by industry, field seams were constructed at the Bureau of Reclamation's Engineering and Research Center under representative field conditions.

The field seams were fabricated by a third party commercial installer in accordance with manufacturers' instructions and utilizing manufacturer-supplied equipment and products where required. Some field seams which could not be constructed at the Bureau were witnessed and collected at job sites by Bureau personnel. Figure 1 shows a typical section of seam being fabricated for this research.

Figure 1. Fabrication of a typical seam section

Approximately 475 meters (1,500 linear feet) of different field seams, including solvent adhesive, bodied solvent adhesive, thermal and extrusion welds were completed for this project.

All seam systems were cut into large samples for environmental exposure. The nonreinforced seams were cut into 300-mm-(12-in-) wide samples in order to later yield twelve 25-mm-(1-in-) wide specimens for mechanical testing and the reinforced seams were cut into 450-mm-(18-in-) wide samples to provide four 50-mm-(2-in-) and eight 25-mm-(1-in-) wide specimens. To allow for possible edge effects due to the scrim exposure, 25-mm (1-in) of each edge of the scrim-reinforced samples was cut off and thrown away. All samples were attached with polypropylene ties to fiberglass rods for exposure testing, as shown in Figure 2.

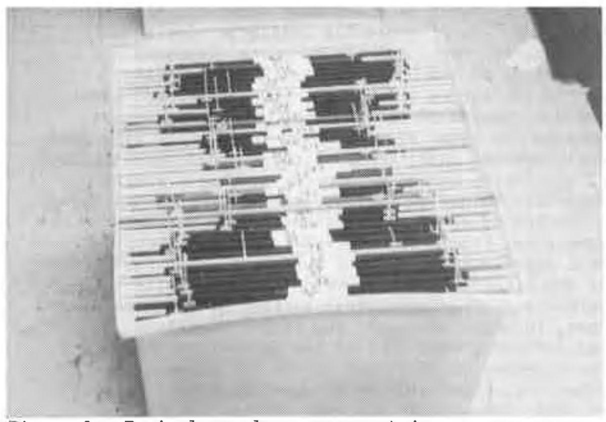

Figure 2. Typical sample arrangement in one 45-gallon tank

Chemical and Aqueous Solution Immersions

The six chemical solutions chosen for this research program include the following:

10% phenol,
10% hydrochloric acid,
10% sodium hydroxide,
10% methyl ethyl ketone,
5% furfural, and
100% methylene chloride.

These chemicals were chosen to represent a wide range of chemical groups, such as organic acid, inorganic acids and bases, halogenated hydrocarbons, ketones and aldehydes and should provide a range of effects on the liner materials. Methylene chloride is not soluble in water; therefore, pure solvent was used to prevent the problem of phase separation. Pure chemicals or aqueous chemical solutions were selected for testing over simulated or actual wastes from waste sites to simplify verification of testing procedures. The use of representative groups of chemicals also allows for reasonable interpretation of the data.

Chemical, brine, and water immersion of seam samples are being accomplished in covered 170-liter (45-gallon) polyethylene and polypropylene tanks as shown in Figure 2. At the end of specified exposure periods, samples will be retrieved for mechanical and analytical testing. In parallel with the tank immersions, smaller coupons of the liner materials are immersed in small clear glass jars for periodic weighings and thickness measurements. The smaller coupons allow for easier inspection of the materials for obvious excessive degradation, swelling, or change in color or surface texture. If any accelerated response is observed in the coupons, the seam samples will be removed from the larger tanks before they are destroyed completely. The ratio of the volume of liquid to the surface area for each coupon is 6.2 mL/cm^2 (40 mL/in^2).

Three tanks of room temperature tapwater and six tanks of saturated sodium chloride brine solution (three tanks at room temperature and three at 50°C (122°F)) are set up for immersing some of the seam samples. The remaining samples are either placed in running tapwater for 6-month saturation before begining 50 cycles of freeze-thaw and wet-dry testing or set aside for heat aging. Double-sided exposure of all samples is being used to accommodate the large number of samples in minimum space. The accelerated effect of double-sided exposure compared to one-sided exposure reduces the time it would take to see any effects the liquids may have on the samples.

Accelerated Outdoor Exposure

In addition to the above exposure conditions, the Desert Sunshine Exposure Test (DSET) Laboratories in Phoenix, Arizona, are performing accelerated outdoor sunlight exposure testing on representative seam samples. The exposures are being accomplished on the EMMAQUA(R) accelerated weathering test machine located at the test facility. Figure 3 shows one of the EMMAQUA machines which are capable of tracking the sun and focusing the suns rays on the specimens for optimum ultraviolet (UV) exposure.

The seam samples are 125-mm (5-in) in exposed seam length. After one year of actual sunlight exposure (approximately equal to seven to eight years of weathering) the samples will be returned to the Bureau and tested for peel strength retention and observed for any obvious deterioration.

Accelerated Environmental Cycling and Heat Aging

In an effort to determine the effects of cyclic stress and heat on the various seam systems, short term freeze/thaw and wet/dry cycling will be carried out on the seam samples that have been submerged in tapwater for six months and heat aging will be carried out on the samples which were set aside. Presently, there is little information available for determining the long-term effects on physical properties in subfreezing temperatures, especially for those membranes that have a tendency to absorb water. Seam samples will be subjected to alternating cycles of freezing for 24 hours and thawing for 24 hours. They will be tested after intervals of 10, 20 and 50 cycles. Wet/dry cycling will represent the fluctuating liquid/liner/air interface where severe weathering can take place. Other seam samples will be tested after 10, 20 and 50 wet/dry cycles. These cycles alternate 24 hours of water submersion with 24 hours of air drying at room temperature.

Many liner systems are installed in the warmer climates of the world where higher temperatures often may be destructive to the liner material. Seam samples will be subjected to 70°C (158°F) for periods of 4, 8 and 13 weeks of oven aging in an effort to provide an accelerated test of long-term heat effects on the seam systems.

Figure 3. EMMAQUA test machine for accelerated outdoor exposure

Physical/Mechanical Testing

At the end of 3-month, 6-month, and 12-month chemical, brine, and water exposures and after completion of the required environmental conditioning intervals, the following physical and mechanical tests will be performed to determine changes in physical properties:

1. Volume (thickness) and weight - Coupon samples were measured for volume and thickness before immersion. Exposed coupon samples are measured every week for the first month, monthly for the next 3 months, and quarterly thereafter.

2. Dynamic tensile shear testing - 50-mm-(2-in-) wide reinforced or 25-mm-(1-in-) wide nonreinforced seam specimens are tested in tensile shear using dynamic loading at a low strain rate of 0.8 mm/s (2 in/min). The reinforced samples are wider for tensile testing in an effort to eliminate edge effects due to the scrim reinforcement. Results will be in N/m (lb_f/in) width of seam.

3. Dynamic peel testing - 25-mm-(1-in-) wide specimens are subjected to 180° peel at a strain rate of 0.8 mm/s (2 in/min). Results will be in N/m (lb_f/in) width of seam.

4. Static dead load peel testing at elevated temperature - 25-mm-(1-in-) wide specimens are subjected to a 180° peel dead load equivalent to approximately five percent of the material's breaking strength per inch of width at a temperature of 50°C (122°F). Results will be expressed by pass/fail, time to failure, and mode of failure.

Test results of exposed seam samples will be compared to the test results of original unexposed seam samples for all tests. The mode of failure will be evaluated as well as the numerical results of shear, peel and dead load in N/m (lb_f/in) of seam width. Figures 4 and 5 show the test equipment used for static and dynamic tensile testing.

Figure 4. Typical dynamic peel test being performed on an Instron 1123 tensile test machine

Nondestructive Test (NDT) Methods

As a part of the EPA project, factory and field seams were evaluated using six known nondestructive test (NDT) techniques. The six techniques evaluated include the following:

- o Acoustic method - ultrasonic pulse echo (5- to 15- MHz)
- o Acoustic method - continuous wave resonant frequency (167 KHz)
- o Air lance- 345 kPa (50 lb/in^2)
- o Vacuum chamber
- o Double seam pressurization
- o Mechanical point stress

The two acoustic methods were performed at the Bureau of Reclamation with equipment and personnel supplied by Battelle Pacific Northwest Laboratories, Richland, Washington. The remaining techniques were performed and evaluated by Bureau personnel.

In general, air lance, vacuum chamber, and mechanical point stressing work well on most seam types. Air lance testing utilizes a jet of 345 kPa (50 psi) air directed at the seam edge through a 4.76 mm (3/16") nozzle. This method detects unbonds, however it was found to be unsatisfactory on the stiffer HDPE material and thicker reinforced sheets. Mechanical point stressing utilizes a blunt instrument, such as a dull screwdriver, to check the bonded edge. This method is not applicable to all geomembranes, especially the thinner ones susceptible to puncture. Vacuum chamber testing should be limited to 30 mil and thicker liners due to the deformation experienced by thin liners under relatively low vacuum. Double seam pressurization is presently limited to the dual hot wedge type of seam used on HDPE; however, the technique (which simply involves injecting a specified air pressure between parallel seam areas and examining for leaks or drop in pressure) could work on other types of dual seams as an integrity check.

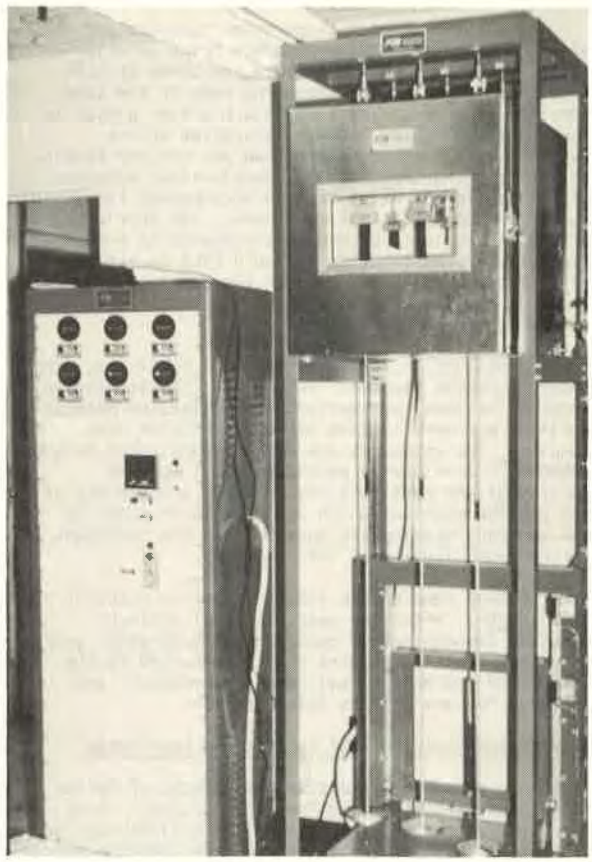

Figure 5. Static dead load test apparatus showing temperature control chamber and automated timing system

The continuous wave resonant frequency, also called ultrasonic impedance plane (UIP) technique testing was found useful in detecting unbonded areas in all types of bonded lining materials including the various scrim-reinforced sheets. As a check on electronic methods, specimens were cut out of areas detected on the UIP as being unbonded and these specimens were tested in mechanical peel. The results of this testing confirmed the accuracy of the UIP when used on reinforced lining seams. The ultrasonic pulse echo technique was found useful in detecting unbonded areas in the nonreinforced sheet materials but not in the scrim-reinforced sheet materials. Both electronic NDT methods need improvement in field application to provide faster test methods. Table 1 outlines the recommended NDT methods for the various lining material types and thicknesses based on this research.

It should be pointed out that these methods only check for bonding of lining material. No correlation between bonding and seam strength has been made from these nondestructive test methods alone.

Analytical Testing

After six months and one year of exposure to chemicals, tapwater, brine, and accelerated outdoor exposure, samples of each polymer sheet material and seam interface through the thickness of the seam, will be examined for changes in chemical composition using Gas Chromatography (GC) and Gas Chromatography/Mass Spectrometry (GC/MS). These analyses will hopefully detect any subtle changes in the chemical composition of the polymer sheet material and bonding system which may have occurred. This method could predict the potential for deterioration of the sheet material or seam. It is important to note very valuable potential use of GC/MS analyses in the early detection of changes in a given polymer with time. This may enable industry and end users to detect problems due to aging or specific environmental exposures.

RESEARCH COORDINATION

The EPA Municipal Environmental Research Laboratory is coordinating this research with other EPA-funded projects in an effort to provide meaningful and comparative test data. Some aspects of these projects are to study microscopic changes in polymer sheet samples after one-sided and double-sided exposure to chemicals. Another focus is on evaluating service life and developing prediction methods based on chemical and physical properties. On a wider scale, the EPA sponsored a meeting with other government agencies who are doing similar research on geomembranes for use in waste containment. Topics for on-going or planned geomembrane research include the following:

- Dead Load Testing - Seams
- Evaluation After Aging - Seams
- Nondestructive Seam Testing
- Field Seaming Procedures/Evaluation
- GC/MS Analysis of Geomembranes
- Geomembrane Failure Mechanisms
- Polymer Chemistry vs. Aging
- Chemical Compatibility/Resistance
- Accelerated Aging of Geomembranes
- Geomembrane/Soil Cover Interaction
- Impoundment Construction Techniques
- Prevention of H_2O Contamination
- Gas Desulpherization of Waste/Use of Geomembranes
- Leak Detection Techniques
- Hazardous Waste/Geomembrane Compatibility
- Evaluation of Old Installations
- Geomembrane Repair Procedures
- Geomembrane Covers - Landfills
- Subgrade Anomaly Conformance
- Restoration of Old Installations
- Utilities Waste Containment
- Soil Cover - Seepage Prevention
- Soil Cover - Infiltration Characteristics
- Prediction of Subsidence - Soil/Geomembrane Cover
- Low-Level Radioactive Waste Containment
- Geomembrane Cost Analysis/Cost Effectiveness
- Determination of Effective Life of Geomembranes

Meetings and interactions with other research groups help direct this project and future EPA projects so that results are comparable and complementary.

FORMULATION OF GUIDELINES

The goal of EPA's seam program is to examine all aspects of seaming and seam durability when using geomembranes for pollutant control in order to provide information on the performance, selection and installation of various lining materials. The information will assist the end users in determining what liner(s) would be effective in containing specific hazardous wastes and ensure proper construction and installation for maintaining the integrity of the liner during daily operations. Compiling the data from this project with others and making it available to the end users may reduce the potential for leachate migration through seam failures to ground-water and surface-water sources.

This paper has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

Table 1. Recommended NDT Methods Based on this Research

Geomembrane system	Thickness (mils)*	Ultrasonic pulse echo (5-15 MHz)	Continuous wave resonant Frequency (167 kHz)	Air lance	Vacuum chamber	Double seam Pressurization	Mechanical point stress
Reinforced	30		*	*	*		*
CSPE	36		*	*	*		*
and	45		*	*	*		*
CPE	60		*		*		*
Nonreinforced	20	*	*	*			
CPE	30	*	*	*	*		*
Nonreinforced	20	*	*	*			
PVC	30	*	*	*	*	*	
	45	*	*	*	*		*
HDPE	20	*	*		*	*	
HDPE-A	30	*	*		*	*	*
LLDPE	40	*	*		*	*	*
	60	*	*		*	*	*
	80	*	*		*	*	*
	100	*	*			*	*
Reinforced	30			*	*		*
EPDM	45			*	*		*
and	60				*		*
BUTYL							
EIA	38				*		*

*1 mil = .0254 mm

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Application of Mass Spectrometry to the Study of Geomembranes

The Bureau of Reclamation is responsible for the design and installation of various geomembrane systems for a number of field projects. These projects range from lining an irrigation canal to lining a water storage reservoir. With this responsibility, the Bureau must be able to test and monitor the performance of these membranes before, during, and after installation. Conventional evaluation of these materials has routinely involved physical and mechanical testing along with such tests as volatility of plasticizers in an effort to determine aging characteristics as well as conformance to specifications. The Bureau has recently expanded its testing capabilities to include an investigation into the chemical composition relating to the type and amount of plasticizer and other additives that are added to the base polymer. This paper describes in detail a procedure which incorporates fused silica capillary gas chromatography together with mass spectrometry. The analytical techniques used in determining the chemical composition of plasticizers in PVC will be discussed.

INTRODUCTION

Establishing the plasticizer composition in polymers has traditionally involved infrared spectrometry, gas chromatography, and determination of the percent extractables. Infrared spectrometry is suitable for analyses when the plasticizer is a single component and can be extracted from the membrane in a pure form. This technique is not suitable for the analysis of a mixture, nor is it adequate to determine the molecular structure or molecular weight of a plasticizer. Gas chromatography has been used by a number of investigators to determine the composition of plasticizers [1-5]. These investigators used packed chromatographic columns of various lengths, polarities, and operating conditions to achieve their analysis. Unfortunately, there is not a single column or set of operating conditions which are adequate to separate a complex mixture of plasticizers into its individual components or simultaneously separate a mixture of both high and low volatility in a single analysis. Previous chromatographic identification of plasticizer composition has been based upon chromatographic retention time or relative retention time data [1, 6]. This approach is not totally valid because this technique cannot distinguish or account for the simultaneous detection of two or more components. This technique does not generate any molecular structure or molecular weight data. The percent extractable determination was conducted to arrive at a quantitative determination of the total amount of plasticizer that is present in a sample. The major drawback with this method is that components other than plasticizers, if present in the extract, will be included in the final result. This will then alter the data to a higher value. Unfortunately none of these techniques, combined or individually, lend themselves to simultaneously separating, identifying, and quantifying a complex plasticizer composition in a sample.

The combination of GC/MS (gas chromatography/mass spectrometry) is used to separate, identify, and quantify the individual components of a sample in a single analysis. With the advent of high resolution gas chromatography using chemically bonded fused silica capillary columns, it is possible to separate a complex mixture into its individual components. For example, figure 1 is the RGC (reconstructed gas chromatographic) trace of sample No. B-6930. Each of the peaks on the trace correspond to a single component. This trace is the plasticizer extract from a PVC sample. Using a single column, this technique lends itself to the separation of components with varying degrees of polarity and volatility.

The mass spectrometer which is coupled directly to the gas chromatograph will generate data that allows one to establish the molecular weight and the molecular structure of the separated components from the gas chromatograph. The molecular weight and the molecular structure are determined by examining the mass spectrum of a separated component. For example, figure 2 is the mass spectrum of an isomer of heptyl nonyl phthalate. The various peaks or lines in the spectrum correspond to the m/z (mass to charge) ratio or weights of the breakdown products from this molecule. Examination of these peaks reflects the molecular weight and structure of the molecule. The peak at m/z 149 is characteristic of dialkyl phthalates and is always present with this class of compounds. The remaining peaks indicate what type of alkyl groups are attached to the molecule. The molecular weight of the compound in figure 2 is 390. This peak is observed in the mass spectrum. Figure 3 is the mass spectrum of di-undecyl phthalate. The molecular weight as indicated by the mass spectrum is 474. Comparing the mass spectra of these two figures we see some similarities and dissimilarities. The similarities (the peaks at 149 and 167) indicate that these components are dialkyl esters of phthalic acid. The dissimilarities in the respective mass spectra indicate that they are not the same molecule; they have different molecular weights and structures, and hence different physical properties. This examination was conducted on all peaks which were detected by the mass spectrometer.

The intensity of the response, of a separated component, from the mass spectrometer is directly proportional to the amount of material which is present in the sample. The quantitation is achieved by comparing the computer generated area counts of each individual component

Table II
GC/MS Operating Condition

I. Gas chromatograph operating conditions

Column:	30 m 0.32 mm i.d. fused silica DB-5 (J&W Scientific) capillary column.
Injection port temperature:	300 °C.
Column temperature:	50 °C for 4 minutes and then temperature programmed up to 300 °C at 8 °C/min. and held at 300 °C for 16 minutes.
Temperature of transfer line:	280 °C.
Carrier gas:	Helium.
Flow rate:	1 mL/min.
Injection mode:	Splitless.
Volume injected:	0.5 to 1.0 µL.

II. Mass spectrometer operating conditions

Ionization mode:	Electron-impact.
Electron energy:	70 eV.
Accelerating voltage:	4000 V.
Scan rate:	0.5 sec/decade.
Resolution	1000.
Electron multiplier:	1800 V.
Mass range:	600 to 20 amu (atomic mass unit).
Trap current:	200 µA.

detected with that of a standard of Bis-2-ethylhexyl phthalate. This procedure will generate two quantitative pieces of data. First, the amount that an individual component contributes to the total amount of plasticizer extracted will be determined. Second, by summing all of the individual amounts in the sample, one can accurately determine the total percent extractable plasticizer in the sample.

GC/MS is a technique which can generate qualitative and quantitative analyses on all individually separated components in a single sample. This technique can also be used to determine what other additives or degradation products are present in a sample extract. The only requirement for detection by the mass spectrometer is that the components in the extract will pass through the gas chromatograph.

PROCEDURE

Four samples of PVC (polyvinyl chloride) sheeting and two samples of CPE (chlorinated polyethylene) were subjected to GC/MS analyses. Table I lists the sample number, description, and manufacturer of the samples.

Table I

Sample No.	Sample Description	Manufacturer
B-7244	Original PVC	A
B-7244	56-day volatility test 20-mil PVC	A
B-6930	Original PVC	B
B-21050	7-year field sample 15-mil PVC	B
MBO	Original CPE	C
MBA	36-month field sample CPE	C

The following procedure is used for these anaylses:

1. Portions of each material are ground to a 20 mesh size.

2. A 1-gram sample of ground membrane is weighed out and placed into a 250-mL screw-cap centrifuge bottle.

3. The extraction solvent, 2:1 carbontetrachloride in methanol, is then added to the sample [7]. The bottle is sealed with a polyethylene lined cap and extracted on a wrist action at maximum speed for 1 hour.

4. After extraction, the sample is filtered through a glass wool plug into a clean 500-mL K-D (Kuderna-Danish) concentration unit.

5. A small boiling chip is added to the K-D unit to prevent bumping during concentration and a three ball Synder column is then attached to the K-D unit and the sample is concentrated over a hot water bath to a volume between 1 to 3-mL.

6. The concentrated extract is then quantitatively transferred to a 10-mL screw-cap culture tube.

7. The volume of the extract is adjusted to a volume of 7.0 mL by the addition of either the extraction solvent or chloroform.

8. The sample is now ready for GC/MS analysis.

The GC/MS analyses have been conducted using a VG-7035 double-focussing mass spectrometer that is interfaced to a Hewlett-Packard 5731 gas chromatograph. The mass spectrometer is interfaced to and controlled by a VG DS-2035 data system. The fused silica capillary column is interfaced directly to the ion source of the mass spectrometer. Table II lists the operating parameters for the GC/MS analyses.

The accepted ASTM procedure for the determination of plasticizer in PVC calls for a Soxhlet extraction [7]. This extraction procedure has been abandoned for the following reasons: (1) preextraction of the Soxhlet with the extraction solvent did not remove all contaminates from the thimble, (2) as is evident from table III, there is little difference in the extraction efficiency of the two techniques, and (3) the Soxhlet

Table IV (1)(2)
Results of GC/MS Analyses

	Heptyl	Nonyl	Undecyl	Heptyl nonyl	Heptyl undecyl	Nonyl decyl	Nonyl undecyl	Percent extractable GC/MS	Conventional
B-7244	73.11	7.73	18.60	68.94	68.30	0.48	44.64	28.2	29.3
B-7244A	14.14	3.43	19.61	48.83	55.41	0.46	38.64	18.1	22.9
% change	-80.71	-55.63	+ 5.43	-43.67	-18.87	- 4.17	-13.44		
B-6930	46.39	5.12	18.93	49.52	66.50	0.27	41.99	22.9	29.5
B-21050	27.66	2.45	10.13	39.60	40.30	0.20	24.64	14.5	15.3
% change	-40.38	-52.15	-46.49	-20.03	-39.40	-25.93	-41.32		

(1) Values are µg/g.
(2) Determinations were in triplicate.

extraction requires at least a day and a half as compared to 1 hour for the bottle extraction.

RESULTS AND DISCUSSION

The results of the GC/MS analyses for the plasticizer composition in PVC are listed in table IV. This table lists the amount and the type of components in the sample extract which have been detected by the mass spectrometer and also lists the percent extractables based on the GC/MS analyses and the conven tional determination. For simplicity, this table is broken down according to compound types. No attempt has been made to distinguish between the various isomers of a given phthalate. From table IV it can be seen that the extracted PVC samples are comprised of seven different n-alkyl phthalates. One can also observe that the concentrations are changing with time. For example, the concentration of di-heptyl phthalate has decreased from 73.11 µg/g in sample B-7244 to 14.14 µg/g after the 56-day volatility test. This trend is observed for all the field and test samples.

The reconstructed gas chromatographic traces of the samples from table I are shown in figures 4 through 7. These traces have been expanded to show a greater degree of detail in the regions which are directly related to plasticizer composition. These traces are profiles of the extracts and indicate the number and relative quantities of components that have been detected by the GC/MS.

Figures 4 and 5 are the RGC traces of an original PVC sample (sample No. B-7244), from manufacturer A and the same sample (sample No. B-7244A), after a 56-day volatility loss test. Upon examination of these traces, a change in the profile between the two traces is observed. This difference indicates that a change in the plasticizer composition of sample B-7244A has taken place. The data in table IV also reflects some loss of all plasticizers, but the greatest loss is observed for the di-heptyl phthalates. This is not suprising. Of all the phthalates detected, the di-heptyl phthalates have a lower molecular weight, and will volatilitize more rapidly [8].

Figures 6 and 7 are the RGC traces from an original PVC sample (sample B-6930) from manufacturer B and an identical lining that was installed in the field in 1977. A visual inspection of the field sample by the Corps of Engineers in 1983 indicated that the lining had deteriorated rather badly with time. This deterioration is reflected in approximately a 50-percent loss of extractable material when compared with the original sample. The traces (fig. 6 and 7) from the GC/MS analyses of these samples do not exhibit the same profile.

Table III
Comparison of Extraction Techniques (1)(2)

Sample Weight	Bottle Extraction	Soxlet Extraction
4 g	31.46	32.58
1 g	31.12	32.43

(1) Determinations were in triplicate.
(2) Results are based on percent extractable.

This is evidence that the plasticizer composition of the field sample has changed with time. This change is also reflected in the amounts of the various plasticizers that have been detected. The data from table IV indicates that the plasticizer losses are high and are rather evenly distributed among the seven different plasticizers detected. At this time, no attempt has been made to detect differences in other additives such as stabilizers or antioxidants which would possibly explain why the losses are so great.

Figures 8 and 9 are examples of a GC/MS analyses that have been conducted on the extract of an original CPE sample (sample No. MBO) and the same sample after a 36-month field exposure (sample No. MBA). The traces from the original and field samples generally exhibit the same profile, but there are some differences. The most noticeable change is the appearance of several additional compounds in the field sample. These compounds, based on their mass spectra, appear to be polymer degradation products. At the time of publication an exhaustive interpretation of the data has not been completed.

CONCLUSIONS

The analyses that have been conducted on geomembranes from three separate manufacturers have detected changes in the composition of the extract from these materials. The changes that have been detected demonstrate that gas chromatography/mass spectrometry can be used in testing and monitoring of geomembranes. This technique can be applied to establish a quality control program, monitor contract specifications, determine aging characteristics of geomembranes, and conduct investigations as to why a particular material has deteriorated. The basic analytical methodology for the analysis of geomembranes is established, but more investigations correlating chemical composition with physical and mechanical testing and aging characteristics need to be conducted.

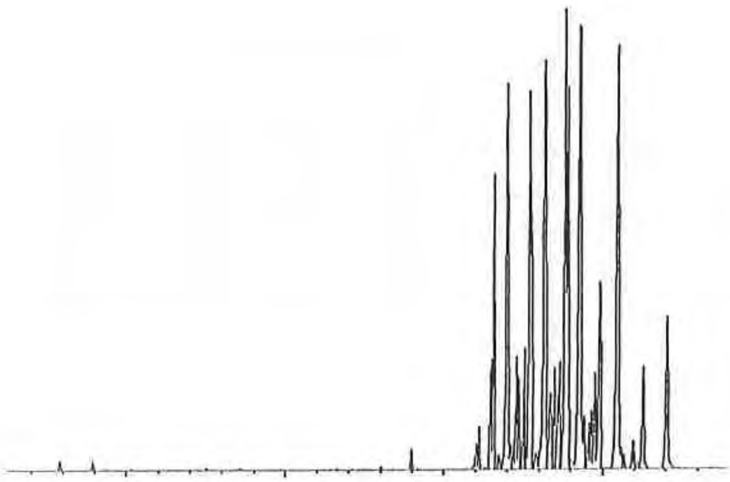

Fig I RGC Sample B-6930

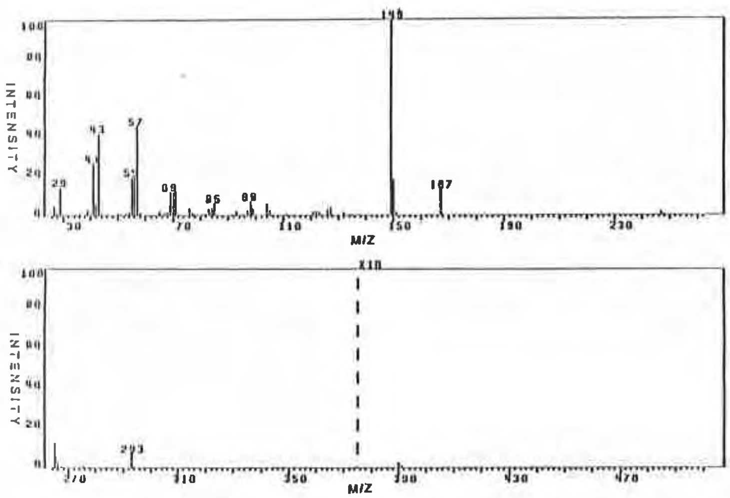

Fig 2 Mass Spectrum of Nonylheptyl Phthalate

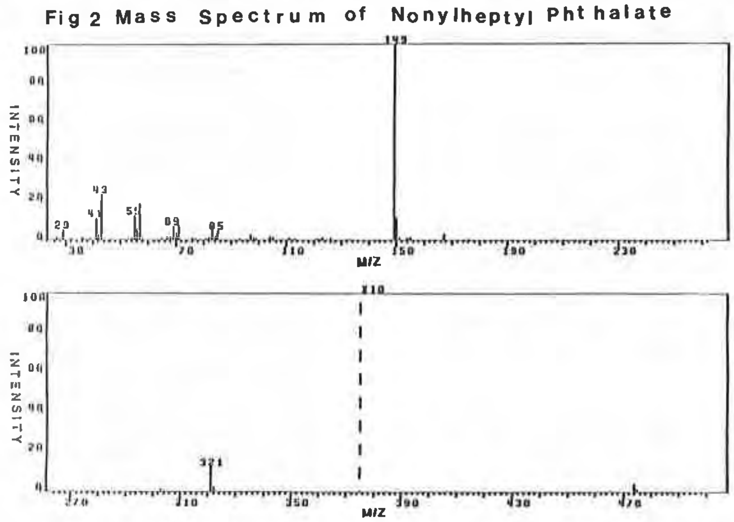

Fig 3 Mass Spectrum of Di-Undecyl Phthalate

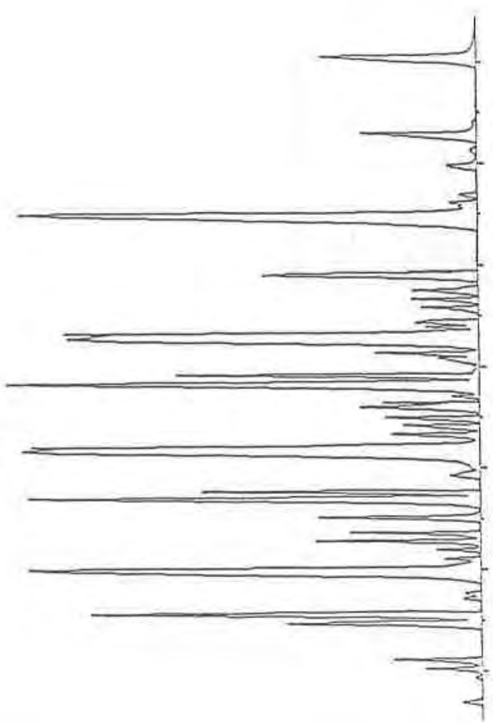

Fig 4 RGC Sample B-7244

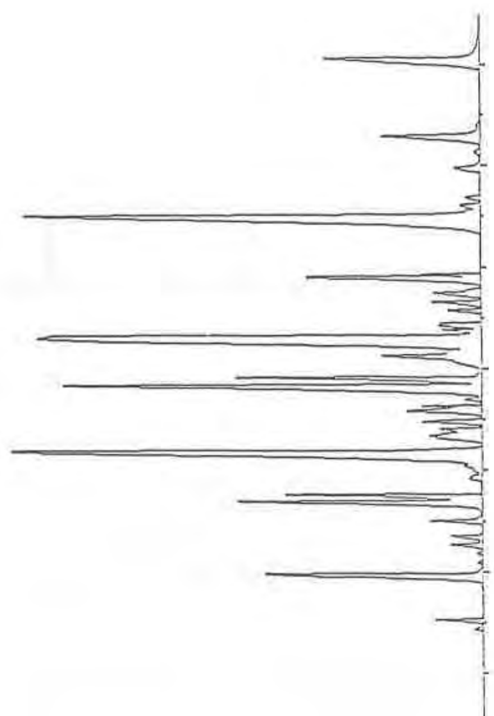

Fig 5 RGC Sample B-7244A

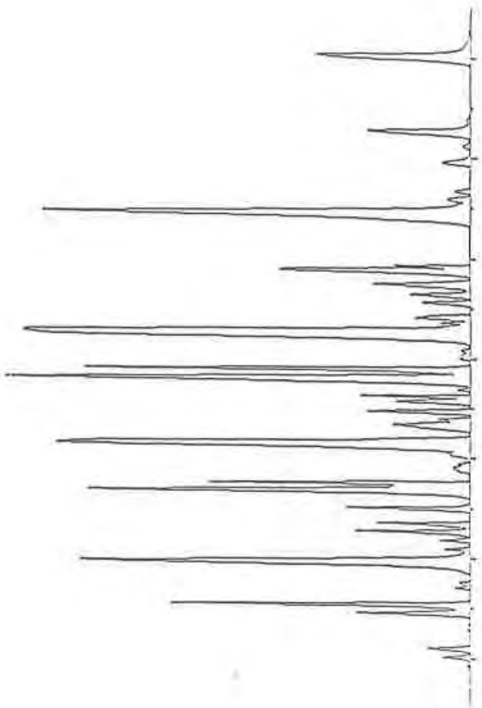

Fig 6 RGC Sample B-6930

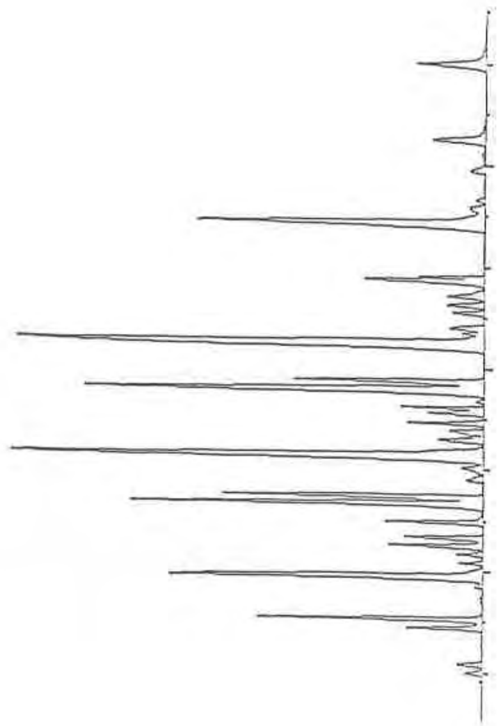

Fig 7 RGC Sample B-21050

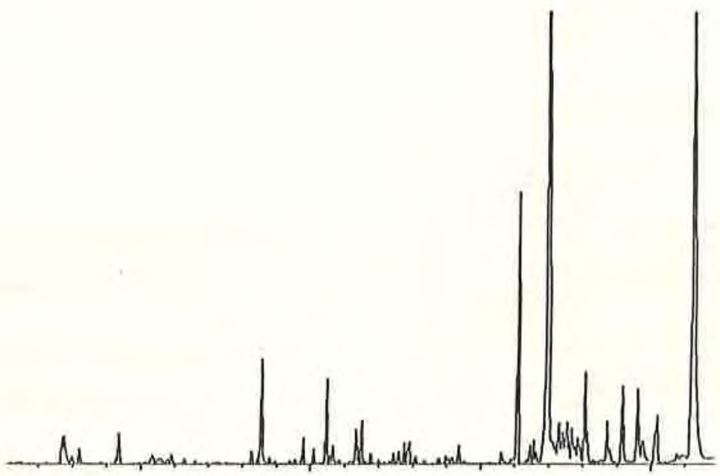

Fig 8 RGC Sample MBO

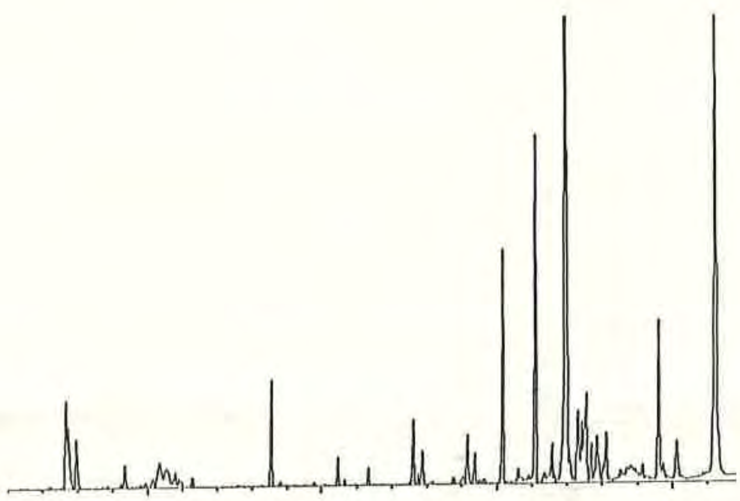

Fig 9 RGC Sample MBA

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Compression Creep of Geotextiles Used as Protective Geomembrane Underliners

Long-term geomembrane integrity is promoted by employing a thick, porous geotextile nonwoven to protect against mechanical stress and to provide drainage for trapped liquids and gases. The geotextile transmissivity is dependent upon fabric caliper changes under the compression load of the solution contained by the geomembrane. Geotextile caliper exhibited a 25% or 50% reduction as compression force increased to 40 kPa or 80 kPa, respectively. Geotextile compression creep resistance to 79 kPa normal load, 8 meter pond depth, was investigated by measuring caliper changes over a 23-month test period. After the 40-50% reduction of initial loading, only a small further caliper loss of less than 10% was noted as the fabric came to equilibrium during the first 9 months of testing. All geotextiles then exhibited resistance to further caliper loss. Thus, long-term geomembrane protection by the geotextile can be expected.

INTRODUCTION

Increased use of geomembranes to contain toxic or corrosive chemical wastes has heightened concern about the long-term stability of the impermeable lining. Incorporation of a thick, porous, nonwoven geotextile beneath the geomembrane is rapidly gaining acceptance as the method of choice to protect the geomembrane from mechanical stresses of puncture and abrasion and to provide drainage for liquids and gases trapped beneath the geomembrane (1,2). Puncture protection is particularly critical during the initial placement of the geomembrane to prevent damage from sharp rocks lying in the subgrade. Abrasion protection and conductivity of trapped gases or ground water insure the long-term integrity of the geomembrane.

Efforts in our laboratory to model geomembrane and geotextile-geomembrane puncture and abrasion resistance toward soil-aggregate environments have been recently reported (1). 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 type, geotextile caliper, and/or geotextile basis weight were important factors that determined resistance to damage. The cyclic compression loading method was also used to compare the relative 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/or geotextile caliper as important factors to preventing geomembrane failure.

Failure to properly drain fluids trapped beneath the geomembrane can lead to catastrophic failure (2). These liquids and gases may originate from solutions leaking through the liner, ground water, gases generated by organic or polluted soil, air trapped beneath the geomembrane during installation, or air displaced from the soil by a rising ground water table. Pressure from these accumulated gases or liquids can float or rupture the geomembrane. Excess trapped water may reduce strength of soil supporting the geomembrane, thus promoting bank deformation. Drainage can be promoted by the use of thick, nonwoven geotextiles.

DEFINITIONS AND FORMULAE

The ability to conduct fluid in the plane of the geotextile, transmissivity, determines its utility as a drain under the geomembrane. The transmissivity θ of a fabric is expressed as a volume rate of flow, q, per unit fabric width, B, and unit hydraulic gradient, i.

$$\theta = q/Bi \qquad (1)$$

The in-plane permeability coefficient K_p can be derived from the transmissivity if geotextile thickness, H, is known.

$$K_p = \theta/H = q/Bi\ H \qquad (2)$$

The porosity, n, of a geotextile is the volume fraction of air in the fabric and can be calculated by Equation 3 where μg is fabric area density, Hg is the fabric thickness, and Pf is the fiber volume density.

$$n = 1 - \mu g/Hg\ Pf \qquad (3)$$

Porosity is often expressed as a percentage. For thick, bulky, needled geotextiles in-plane permeability, Kp, has been shown to be directly porportional to a function of porosity as shown in Equation 4 (3).

$$K_p = A\ n^3/(1-n)^2 \qquad (4)$$

The constant A is the product of fiber and fluid constants.

Transmissivity can be expressed as the product of in-plane permeability and fabric thickness.

$$\theta = K_p\ H \qquad (5)$$

Equation 5 is useful because inclusion of thickness allows the transmissivity to be investigated as the fabric is loaded under compression. As an example, when the containment pond is filled with solution, the force compressing downward on the geomembrane and the geotextile under it will increase in direct proportion to water depth. Under compression the fabric thickness or caliper will be reduced with a corresponding loss in transmissivity.

GEOTEXTILE THICKNESS UNDER COMPRESSION

The change in geotextile caliper under increasing compression force was investigated for five thick, porous, nonwoven geotextiles of the type used to provide fluid drainage under geomembranes. These products are characterized in Table 1 as Fabrics 1-5. The dependence of fabric caliper upon compression force is illustrated by the plots in Figure 1. Caliper vs. compression was determined using an Instron tensile testing machine operating in the compression mode. Force was recorded as the geotextile was compressed at the constant rate of 1.27 mm/sec (0.05 inch/min.). Caliper was observed to fall approximately 25% as loading increased to 40 kPa (5.8 psi), a force equal to 4m (13 ft.) depth of water. A further loss to approximately 50% initial thickness was observed as compression force increased to 80 kPa (11.6 psi), equivalent to the weight of 8m (27 ft.) depth of water. Loss in caliper with increasing normal load then began to stabilize as the loading approached 125 kPa (18.1 psi), a force equal to 13m (42 ft.) depth of water.

TABLE 1

GEOTEXTILES USED AS PROTECTIVE GEOMEMBRANE UNDERLINERS, CHARACTERIZATION OF FABRICS USED IN CALIPER AND COMPRESSION CREEP STUDIES(1)

Fabric	Type	Basis Weight, g/m^2	Caliper Under Indicated Load, mm 0.0 kPa	79 kPa
1	Needlepunched, spunbonded polypropylene nonwoven fabric	361	4.62	2.21
2	Needlepunched, spunbonded polypropylene nonwoven fabric	394	4.93	2.34
3	Needlepunched, spunbonded polypropylene nonwoven fabric	546	4.78	3.04
4	Needlepunched, spunbonded polyester nonwoven fabric	569	4.62	2.72
5	Needlepunched, carded polyester nonwoven fabric	375	3.33	1.97
6	Needlepunched, carded polypropylene nonwoven fabric	295	--	1.92(2)

(1) Caliper coefficient of variation under 79 kPa force was typically 2-4%.
(2) Estimated.

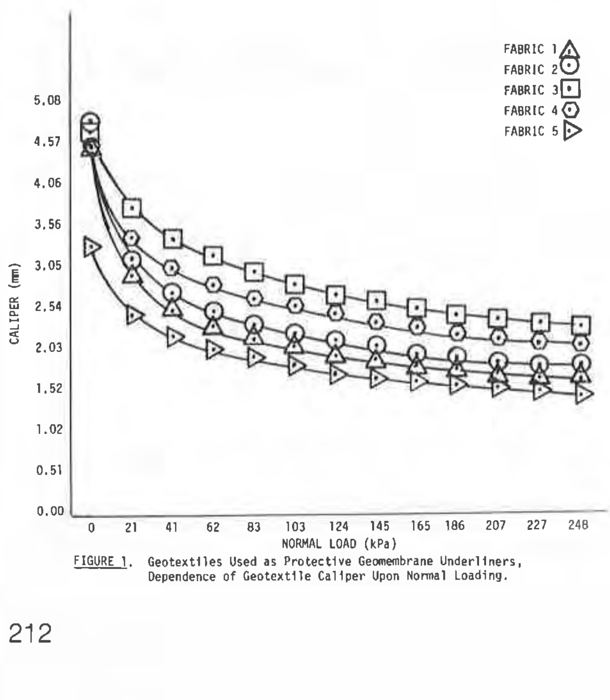

FIGURE 1. Geotextiles Used as Protective Geomembrane Underliners, Dependence of Geotextile Caliper Upon Normal Loading.

Recent investigations of the change in geotextile water transmissivity with increasing compression force generally found an initial exponential decrease in transmissivity followed by a region where this property holds nearly constant (3,4,5). Plots of transmissivity vs. normal load thus exhibit shapes similar to those seen in Figure 1.

COMPRESSION CREEP

Thus far, the loss in geotextile caliper and transmissivity has been described as the fabric is subjected to normal loading from the solution filling the liquid containment pond. Clearly it is critical for providing liquid drainage or gas release from under the geomembrane to use estimations of transmissivity corresponding to loading expected by this operating depth of the pond. Long-term protection of the geomembrane by the geotextile necessitates maintenance of that compression transmissivity with time. Loss of transmissivity might take place by infiltration of soil into the fabric. This could be a significant problem if the soil supporting the liquid containment pond had a significant clay content (4). Compression creep could be a second mechanism for loss of the geotextile transmissivity. Compression creep is the slow loss of fabric caliper or thickness with time under load. Poor compression creep resistance would mean a slow reduction in planer gas and liquid transport properties as caliper and thus porosity decreased.

Geotextile creep studies have generally focused on loss of tensile properties while the fabric was subjected to a static load (6,7,8). Since those studies do not relate directly to changes in porosity with time under normal load, a study of compression creep was initiated.

Six, thick, needlepunched geotextiles were evaluated for resistance to compression creep by measuring the change in caliper when subjected to a normal loading of 79 kPa, equivalent to a depth of 8.0 meters (26.4 feet). These geotextiles are characterized in Table 1 and represent the four generic fabric types commonly used in this application--polyester, polypropylene, needlepunched carded, and needlepunched spunbonded. Basis weight ranged from 295 g/m^2 up to 569 g/m^2.

A compression creep test stand, as shown in Figure 2, was employed to test four specimens of each fabric. In this drawing two of the specimens are visible; the other two would be positioned similarly on the back side. The four specimens, each 79 cm^2 (12.25 in^2) in area, were loaded with a total weight of 254 kg (560 lbs), provided by metal plate 2, plywood, and a 55-gallon drum filled with the calculated quantity of water.

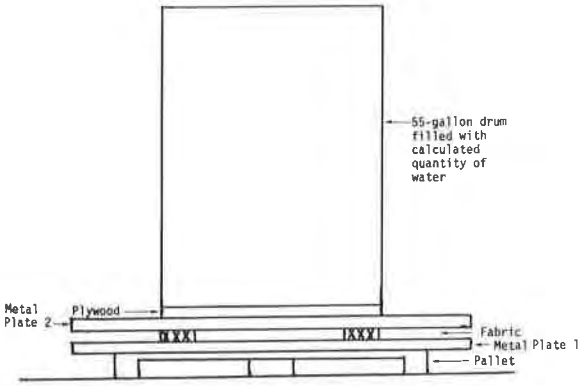

FIGURE 2. Compression Creep Test Stand.

The change in each fabric caliper with time after loading was determined by the following procedure. A micrometer was used to measure the distance between the bottom of metal plate 1 and the top of metal plate 2 at two sites near each of the four squares of fabric. A base value of these eight distances was obtained for each test stand during the first month of the compression creep experiment by repeating the above-described measurement procedure on six different days.

The displacement between the metal plates could not be directly equated to geotextile caliper under the specified load due to slight metal plate curvature. However, since the average force acting on the four fabric specimens equaled the specified force of 79 kPa, average changes in metal plate displacement could be equated to changes in caliper.

During each 3-month interval following test stand set-up, several measurements were made to determine the distance between metal plates 1 and 2 at each of the eight locations. The average of these distances, divided by the base value, multiplied by 100%, yielded the percent initial compression caliper retained for the fabric during each time period. Results are summarized in Table 2. Thus, as an example, Fabric 1 after 9 months showed 93.4% of the initial measured displacement. On the average the distance between metal plates 1 and 2 had been reduced 6.6% at each of the eight measuring sites in Test Stand 1.

TABLE 2

COMPRESSION CREEP--PERCENT INITIAL COMPRESSED CALIPER RETAINED WITH TIME FOR 0 TO 23 MONTHS LOADING AT NORMAL FORCE EQUAL TO 79 kPa OR 8.0 METERS (26.4 FEET) WATER DEPTH(1)

MONTH	FABRIC 1	FABRIC 2	FABRIC 3	FABRIC 4	FABRIC 5	FABRIC 6
0	100	100	100	100	100	100
3	99.2	98.4	97.2	98.9	100	--
6	94.2	95.2	91.8	96.7	96.7	99.1
9	93.4	95.1	91.5	95.7	96.1	98.8
12	94.4	96.2	90.8	99.0	101	100
15	96.2	99.9	91.6	102	106	104
18	96.0	98.2	92.2	100	105	101
21	96.2	96.2	90.2	100	103	100
23	97.0	96.2	90.6	101	105	102

(1) Coefficient of variation generally less than 5%.

Inspection of the percent initial compression caliper values in Table 2 shows a resistance to compression creep for all six geotextiles. In each case caliper is reduced during the first 6 to 9 months. Then after approximately 12 months, the percent initial compression caliper value remains constant or slightly increases. Thus, as an example, Fabric 1 shows approximately 5% loss of the initial compression caliper during the first 12 months. The observed compression caliper then remains nearly constant for the balance of the test. The difference between the six fabrics thus appears to be the time needed to reach an equilibrium compressed thickness.

The change in compressed caliper with time can be directly estimated by combining the measured geotextile calipers at 79 kPa shown in Figure 1 with the percent initial compressed caliper values summarized in Table 2. The results are shown as plots in Figure 3. As noted, caliper generally decreased during the first 9 months, then stabilized for the balance of the test period.

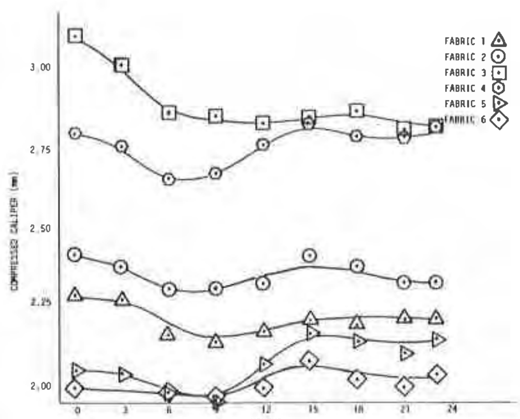

FIGURE 3. Compressed Creep Resistance Illustrated by Small Change in Compression Caliper With Time Under 79 kPa Normal Load.

TABLE 3

CALIPER AND POROSITY FOR GEOTEXTILES SUBJECTED TO COMPRESSION LOADING

Fabric	Caliper No Load mm	Caliper Under 79 kPa Load, mm Initial	After 12 mos.	After 23 mos.	Porosity, % No Load, mm	Porosity Under 79 kPa Load, % Initial	After 12 mos.	After 23 mos.
1	4.62	2.21	2.09	2.14	91.4	81.9	80.9	81.3
2	4.94	2.34	2.25	2.25	91.2	81.4	80.6	80.6
3	4.77	3.04	2.76	2.75	87.3	80.1	78.1	78.0
4	4.64	2.72	2.69	2.75	91.4	85.0	84.7	85.0
5	3.32	1.97	1.99	2.07	91.8	86.2	86.3	86.9
6	--	1.92	1.92	1.96	--	83.0	83.0	83.4

Fabric porosity before loading, after initial compression, and after 12 and 23 months of compression was calculated using caliper values from Table 1 and Figure 3. A summary of caliper and porosity for geotextiles subjected to compression loading is shown as Table 3. As discussed above, caliper is significantly reduced when the geotextile is loaded; 40-50% of the unloaded fabric thickness is lost as compression rises to 79 kPa. This causes a 6-10% reduction in porosity. As loading continues for 12 months and 23 months, further loss in either caliper or porosity is minimal.

Thick, porous, nonwoven geotextiles are increasingly being used to insure long-term integrity of geomembranes containing toxic or corrosive chemical wastes. Protection from liquid or gas buildup under the geomembrane depends upon maintaining geotextile transmissivity. Six geotextiles representing the types of fabric used in this application were exposed for 23 months to a compression force equal to that of 8 m (26.4 ft.) of water depth. All fabrics immediately lost 40-50% of their noncompressed caliper. A further slow loss of up to 5% noncompressed (10% compressed) caliper was noted as the geotextiles came to equilibrium under the load. Then after approximately 12 months, fabric thickness remained nearly constant. Thus, all geotextiles exhibited resistance to compression creep. Long-term protection of geomembranes by these types of geotextiles can be expected.

ACKNOWLEDGEMENTS

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Hazardous Waste Landfill Geomembrane: Design, Installation and Monitoring

The geomembrane selection process and installation monitoring for a hazardous waste disposal cell liner are highlighted in this paper. Concepts relating to development of quality control requirements for geomembrane installation are presented. Observations of the gemembrane installation and repair of defects are summarized. Observations during geomembrane installation indicated that defects in the geomembrane sheets were more prevalent than those typically detected in the welded seams. Underlying rock protuberances resulting from freeze/thaw cycles, pinholes that developed in wind-damaged sheets, and mechanically caused geomembrane penetrations from accidents were most common.

INTRODUCTION

This paper presents a case history of a geomembrane installation for a hazardous waste disposal cell. The 1 hectare (2.5 acre) cell was constructed in a Midwestern US State. Cell design and geomembrane selection were in conformance with the then current United States Environmental Protection Agency (USEPA) Regulations (26 July 1982) and Guidelines for Hazardous Waste Disposal under the Resource Conservation and Recovery Act (RCRA) Permitting Requirements (3, 4).

GEOMEMBRANE SELECTION

Selection of a suitable geomembrane for hazardous waste disposal followed a step by step process. Steps in the process included:

- definition of the waste leachates,
- geomembrane material and thickness selection, and
- compatiblity testing with simulated leachate.

Definition of the Waste Leachates

The first step in the geomembrane selection process was to identify the waste materials to be disposed in the new cell and to estimate the leachates which may result from infiltration contact. The new cell was to contain stabilized wastes resulting from addition of lime and/or flyash to sludges existing in several on-site evaporation and treatment ponds.

Each pond had been used for different containment/neutralization functions. The evaporation ponds had been used for aeration of volatile organic containing fluids from industrial waste solvents. Neutralization of industrial acids and bases had been performed in various treatment ponds, resulting in pond sludges each respectively high in chemical concentrations of chromium, phosphorous, sulfates, and lead. Pond sludges were highly acidic, resulting in the need to add lime and flyash to achieve a neutral pH and stabilized consistency.

The concentrations of hazardous chemicals (represented by USEPA priority pollutants) in leachate that would be generated by contact of infiltration with the stabilized sludges were estimated by the project chemical engineer. The concentrations of USEPA priority pollutant metals in the leachate were estimated to be 2 to 3 orders of magnitude less than concentrations measured in raw sludge. The estimated concentrations depended upon the specific metal and mobility characteristics in water. The concentrations of USEPA priority pollutant volatile organics in the leachate were estimated to be 1 to 2 orders of magnitude less than the raw sludge concentrations. Subsequent testing of simulated stabilized sludge leachates indicated that these estimates were reasonably accurate.

Preliminary Geomembrane Selection

A review of manufacturers' literature regarding compatibility with the anticipated waste leachate concentrations was performed. Candidate materials were selected and letters requesting interest, verification of chemical compatiblity to the estimated leachate concentrations, and guarantees were sent to five known manufacturers of candidate geomembranes. The geomembrane material, high density polyethylene (HDPE), and Manufacturer, Gundle Lining Systems, Inc. of Houston, Texas, were selected based upon the responses. Gundline HD was specified.

A 60-mil geomembrane thickness was chosen. This decision was initially considered to be a conservative judgment based on the desire to exceed minimum 30-mil USEPA thickness requirements (4), and based upon experience relative to material durability and the desire to minimize the effects of damage during installation. Subsequent observations during geomembrane installation indicated that this judgment was reasonable and not overly conservative.

The geomembrane availability and cost (compared to representative price quotes from other suppliers for similar projects), although not the primary decision factors, were considered in the geomembrane selection.

Compatibility Tests

The Manufacturer's claim of geomembrane chemical compatibility was verified by laboratory testing. Compatibility testing was performed in general conformance with USEPA Guidelines, Method 9082 (superseded by Method 9090) (5, 6) and National Sanitation Foundation (NSF) Final Draft Standards,Oct 1982 (2).

Some defects could not be attributed to specific accidents. These defects included punctures and deep scratches, probably from: 1) stones caught in boot soles (Figure 7), 2) scratches caused by movement of ice collecting in the disposal cell during the winter, and 3) scratches from winter-visiting animals. Where judged by visual inspection to penetrate 20 mils or greater, a weld bead was applied to the punctures or scratched area.

The winter exposure prior to leachate collection system sand layer placement resulted in several problems related to geomembrane liner sheets and also within some welded seams. Freezing and thawing cycles apparently opened pinhole sized penetrations in: 1) thinned geomembrane sections caused from previously described wind-crimping of some geomembrane sheets, and 2) some welded seams which originally passed vacuum testing. The pinholes in welded seams were typically at the intersection of two seams. Weld beads were applied to wind-crimped pinhole penetrations. Previously welded seams were smoothed with a grinder (Figure 8), rewelded and vacuum tested.

Summary of Remedial Measures for Defects Resulting from Geomembrane Liner Installation

Table 1 summarizes geomembrane liner penetrating defects both after initial and final liner inspections. Defects per 100 lineal m (305 lineal ft) of seam and 1000 m^2 (10,765 ft^2) of surface area were calculated and are presented in Table 1.

Fig. 8 Grinding a Patch Weld for Rewelding

TABLE 1 - SUMMARY OF DEFECTS WHICH PENETRATED THE GEOMEMBRANE LINER

Type of Defect/ Method of Repair	Total No. of Defects	Total Defect Length or No. of Defects per Unit Length or Area[a]
Initial Inspection		
. Seam Pinhole Leaks/ Rewelded	140	6.4/100 m of Seam[b]
. Mechanically (accidentally) caused cuts and punctures/ patched or welded	10	0.9/1000 m^2 of Sheets[b]
. Wind Damage Crimps and Pinholes/ Patched (13) and Welded (26)	39	122/1000 m^2 of Wind Damaged Sheets[b]
. Wind Damage Folds/ Cut and Welded	3	15 Lineal m - Total
Final Inspection		
. Pinholes that Developed at Previously Acceptable Vacuum Tested Seams/ Rewelded	45	2/100 m of Seam
. Mechanically (accidentally) caused cuts and punctures/ patched or welded	34	3.0/1000 m^2 of Sheets
. Pinholes that Developed at Previously Acceptable Vacuum Tested Wind-Crimp Locations/ Welded	7	22/1000 m^2 of Wind Damaged Sheets
. Miscellaneous Non-Cause Specific Cuts and Punctures/ Patched	26	2.3/1000 m^2 of Sheets
. Rock Protuberances/ Cut and Patched	305	27/1000 m^2 of Sheets
. Wrinkles Removed/ Cut and Welded	9	104 Lineal m - Total

Notes:
a. Based on the total seam length of 2175 m; total geomembrane area of 11,200 m^2; and wind damaged geomembrane area of 320 m^2.
b. 100 m = 328 ft; 1000 m^2 = 10,765 ft^2

ACKNOWLEDGEMENTS

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REMEDIAL MEASURES FOR DEFECTS RESULTING FROM GEOMEMBRANE LINER INSTALLATION

The Monitoring Engineer observed and documented all aspects of geomembrane liner sheet placement, seam welding procedures and quality, defects and repairs.

Problems Associated with Geomembrane Liner Sheet Placement

Several problems resulted from the methods and procedures used to place the geomembrane sheets in the disposal cell, including:

- wind damage, and
- wrinkles from thermal expansion.

The sandbagging process used to weigh down the geomembrane sheets was successful, except in one instance, when gusty winds caught several sheets during placement (Figure 1). The sheets flapped violently in the wind resulting in severe damage, comprised of tears, folds and crimps. The torn and severely damaged portions of the sheets were discarded. All evident folds and crimps in the remaining portions of the sheets were vacuum tested after each sheet was placed and welded into final position. Detected pinholes along folds were repaired by application of a weld bead to the geomembrane surface. Weld beads were also applied to fold lines which, by evidence of their light gray color (as compared to the black original surface color) exhibited stretching and thinning.

The final geomembrane liner inspection was performed approximately 8 months after essential completion of the liner installation. This 8 month time period included typical freezing winter temperatures and several freeze/thaw cycles. Vacuum testing during final inspection detected pinholes at the location of several wind crimps. It is judged that these pinholes may have developed during the winter exposure of the geomembrane.

The thermal expansion characteristics of the HDPE geomembrane were reversible; that is, when thermal expansion occurred during daytime heating from a hot sun, the contraction upon cooling at night resulted in essentially full recovery to the original shape. However, since sheet seaming operations were performed at various times of the day when the geomembrane sheets were in various stages of expansion, alignment of all sheets at the same ambient temperature and degree of expansion was impossible. Consequently, several areas of the liner developed excessive wrinkles.

Wrinkles which were judged to significantly inhibit drainage of the leachate collection system sand layer (placed on the geomembrane) were removed. A rule-of-thumb developed for the specific project was that if the wrinkle height at the estimated final in-situ temperature of approximately 12°C (55°F) was greater than 25 percent of the 0.3 m (1.0 ft) thick leachate collection system sand layer, the wrinkle would be removed.

Wrinkles were removed by cutting the geomembrane at the top of the wrinkle, overlapping the cut sections and applying a weld bead to seam the overlapped sections.

Placement of the leachate collection system sand layer was performed in the cooler summer morning hours so that the geomembrane was in a condition most closely representing final in-situ temperature conditions. The sand was spread so that wrinkle waves were not pushed ahead of the sand. Wrinkle waves could result in the accumulation of large wrinkles at the edges of the geomembrane. Where possible, the sand was spread perpendicular to the wrinkle axis.

Problems Associated with Seam Welding Procedures

Several problems developed during seam welding procedures, including:

- the hand pulling method for field startup seam tests,
- startup seam test timing, and
- welding in light rainfall.

Field fabricated startup seam tests were required as a part of the quality control program to verify proper performance of the seam welding units. A minimum 0.6 m (2 ft) long section of test seam was welded before in-situ liner welding was allowed (Figure 2). A section of the test seam was cut into two portions, each 1.3 cm (0.5 in.) wide. One portion was hand pulled by the Geomembrane Installer until failure in peel or shear

Fig. 1 Gust of Wind During Installation

Fig. 2 Startup Seam Test Weld

The most readily apparent problem related to protuberances in the liner which were caused by underlying rocks. Project Specifications required removal of rocks greater than a 2.5 cm (1 in.) size from the silty clay soil liner constructed directly under the geomembrane (a geotextile was not specified for installation under the geomembrane). The silty clay soil liner was constructed in an engineered controlled manner and the final surface smoothed with a steel wheeled roller to the satisfaction of appropriate parties prior to geomembrane liner installation. The freezing conditions during the winter interlude apparently caused underlying rocks to frost-heave in a manner similar to rock heaves that trouble northern-climate farmers in plowed fields each spring.

The Monitoring Engineer identified rock protuberances as potential defects and the Geomembrane Installer made the final decision as to whether the potential defect was a problem requiring corrective action. Corrective action was comprised of cutting a 3-sided hole in the geomembrane, removing the rock, and welding the cut or applying a patch. Figure 4 shows protuberances caused by underlying rocks.

Mechanically caused penetrations of the 60 mil thick geomembrane resulted from accidents during cleaning and final inspection. For example, a semicircular cut resulting from accidental dropping of the metal end of hose used for washing the geomembrane is shown in Figure 5. Patches were applied to the cut areas.

Several holes resulted when the carefully and conscientiously operated motor grader and dozer cut too deep when removing sand from the anchor trench area and when spreading the leachate collection system sand layer, as shown in Figure 6. Patches were applied to these gouged areas.

Poor workmanship during cutting of fitted geomembrane pieces for the leachate collection system sump areas (cut without a backup cutting board) resulted in deep cuts, some penetrating through the primary geomembrane. These defects were corrected by applying a weld bead to the cut followed by vacuum testing to verify weld appropriateness.

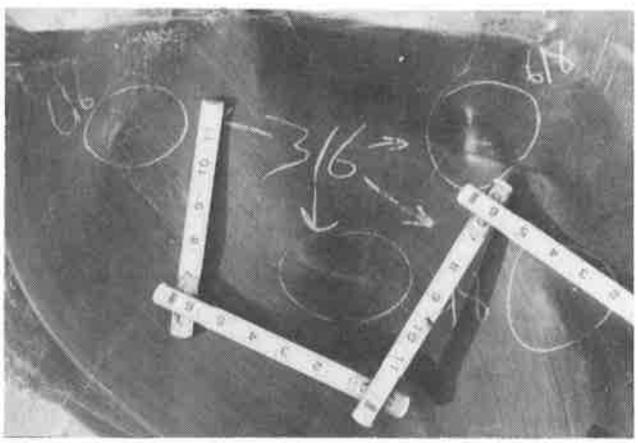

Fig. 4 Proturberances Caused by Underlying Rocks

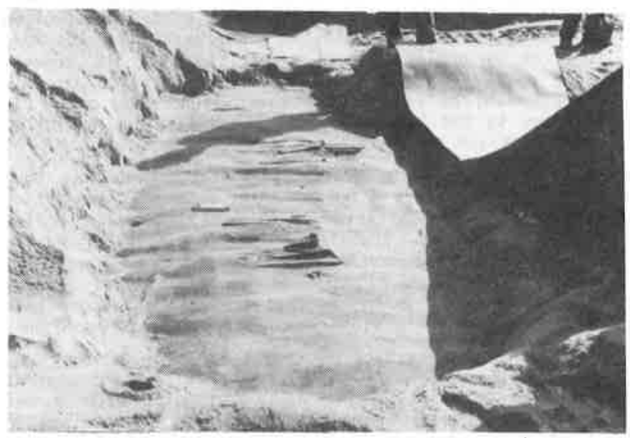

Fig. 6 Accidental Gouges Caused by Dozer During Leachate Collection System Sand Spreading

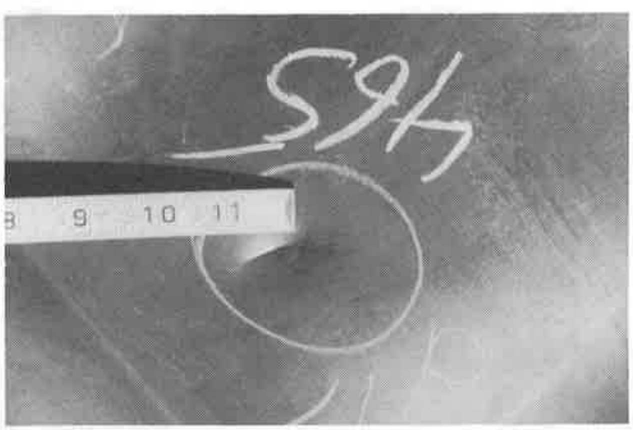

Fig. 5 Cut from Accidentally Dropped Hose

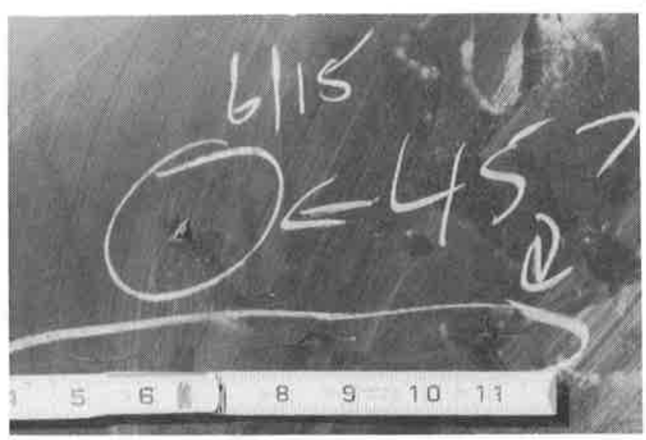

Fig. 7 Punctures Likely Caused by Stone Caught in a Boot Sole

occurred. If failure of the test seam weld joint did not occur, the welding unit was judged to be operating properly and in-situ liner welding proceeded.

Samples of the second portion of the startup seams were tested by the Monitoring Engineer. A problem developed when failure of the welded joint of the second portion of a few test seams occurred after the initially tested portion had not failed. The inconsistency was apparently due to acceptable testing of the initial portion by an older small-framed installer versus failure of the second portion of the test seam when pulled by a young, strong Monitoring Engineer employee. Subsequent laboratory testing indicated that the tensile force applied by the young, strong person exceeded the Project Specification laboratory tensile force maximum requirement by 1.5 times. Thus, although failure in the weld joint occurred during the crude field test, the failure forces grossly exceeded the applicable laboratory failure criteria.

Startup seam tests were required at the beginning and end of each day to verify welding unit performance. A problem developed when several end-of-the-day tests failed. A failed test at the end of the day raised questions about the test unit performance after welding, rather than before. Failed end-of-the day tests required reassessment of all afternoon (after-lunch break) in-situ liner seams. This resulted in extra vacuum testing and general uncertainty regarding the prformance of each welding unit. Subsequent startup seam testing was required at the beginning of each work period, rather than at the end of the day.

Extrusion welding requires dry conditions. Welding is typically not performed until the liner dries after the morning dew lifts. Welding can not be performed in rain. In several instances, the completion of welding for a liner section was within a few minutes of being finished when a light rain began. Workers constructed a makeshift canopy of plastic sheet above the welder to keep rain off of the geomembrane surface and allow welding to proceed.

Problems Associated with Quality of Welded Seams

All seams were vacuum tested. In addition to repairs required for typical defects of pinholes in improperly welded seams, several problems developed which related to seam quality. Problems included:

- time delays between field seaming and laboratory testing, and
- Regulatory Agency acceptance of the Geomembrane Manufacturer's laboratory testing of their own products and performance.

Seams were judged to be adequate by virtue of passing vacuum tests. When field startup seam tests failed at the end of the day, laboratory tests of retained portions of the startup seams were required to verify seam adequacy. Laboratory testing was performed at the Geomembrane Manufacturer's facility which resulted in several days of delay. In a few instances, the leachate collection system sand placement over the geomembrane liner proceeded more rapidly than the laboratory testing could proceed. This timing problem was resolved by placing a cap strip on the seams in question, prior to and irrespective of the outcome of laboratory testing. Figure 3 shows cap strips being placed on a seam resulting from wrinkle removal.

Testing methods and responsible parties were discussed with the appropriate Regulatory Agency prior to geomembrane installation. However, a problem arose when changes in agency personnel occurred and the new personnel questioned previously approved policies. In particular, one policy that was questioned was that of

Fig. 3 Cap Strips Being Applied to a Wrinkle Removal Seam

the Geomembrane Manufacturer providing laboratory compatibility tests and startup seam tests for their own products and performance. The Regulatory Agency concern was irrespective of the fact that the Monitoring Engineer directed all laboratory tests and witnessed critical testing procedures. The problem was resolved by performing independent check testing by a nationally respected laboratory selected from two firms recommended by the Regulatory Agency. Correlation of test results from this independent laboratory (to previously obtained tests on a portion of the same in-situ test coupons) was taken as confirmation of the Geomembrane Manufacturer's test results.

Problems Associated with the Quality of In-Place Geomembrane Sheets

The most significant problems relating to geomembrane installation were not associated with typical defects of inadequate seams, but rather with defects associated with the geomembrane sheets. The problems were aggravated by the 8 month delay from liner installation until final inspection and placement of leachate collection system sand layer.

Final inspection was performed after hand washing the entire liner surface with a fire hose, squeegees and sponges to remove wind blown silts to allow for visual inspection.

Problems identified during this final detailed inspection were associated with:

- underlying rock protuberances,
- mechanically caused cuts and punctures,
- miscellaneous cuts and punctures, and
- pinholes that developed during winter exposure.

The selected geomembrane was tested for compatibility with three fluids: 1) raw sludge, 2) simulated leachate generated from raw sludge, and 3) simulated leachate generated from stabilized sludge. The fluid generated from the stabilized sludge was judged to be most representative of leachate which may result from the in-situ wastes disposed in the landfill.

The leachate generated from raw sludge was considered to be representative of unlikely but possible in-situ conditions where full stabilization may not have been achieved. Short-term (7 day) testing of the geomembrane with raw sludge was performed to indicate the maximum possible effect of the sludge; this testing was used only to verify compatibility at the time of geomembrane purchase. Purchase was required prior to completion of long term compatibility testing because of construction scheduling.

Simulated leachates from representative raw and stabilized sludges were generated by mixing four parts of distilled water (by weight) with one part of raw or stabilized sludge for 24 hours in a flexible polyethylene container (air was expelled from the flexible container to minimize organic volatilzation). The stabilized sludge was broken into pieces which passed a 0.95 cm (3/8 in.) sieve prior to testing. Representative sludges used for leachate generation contained high concentrations of metals and volatile organics. The pH of each leachate was maintained at the level resulting from the distilled water contact (not adjusted to a pH of 5 as suggested in the USEPA toxicity test) to simulate final in-situ conditions. Raw sludge leachate pH ranged from 3.8 to 8.1. Leachate from stabilized sludges ranged from pH 9.0 to 12.0.

The results of compatibility testing with simulated leachate from 4-month long contact with both stabilized sludge leachate and raw sludge leachate indicated that "no significant loss of properties" had occurred during testing. Slight variations were noted, but when evaluated versus data in the literature (1) it was concluded that significant variations had not resulted.

DEFINITION OF QUALITY CONTROL PROCEDURES DURING GEOMEMBRANE INSTALLATION

The Geomembrane Manufacturer was responsible for product quality control and quality control during installation. The Monitoring Engineer was responsible to verify and document installation quality control procedures. Project Specifications relating to quality control included definition of:

- responsibilities of various parties,
- geomembrane installation procedures,
- testing requirements,
- reporting responsibilities, and
- repair procedures.

A pre-installation meeting of all parties was held to discuss the Project Specifications and responsibilities of each party involved with the geomembrane installation. An installation plan, required in the Project Specifications, was reviewed to be sure all parties agreed with the step-by-step location and seam patterns for geomembrane placement.

OBSERVATIONS DURING INSTALLATION

The Monitoring Engineer was responsible for observing and documenting all aspects of geomembrane installation. Key aspects of installation which were monitored included:

- placement of geomembrane sheets,
- seam welding procedures,
- quality of seams, and
- damage of geomembrane sheets.

Placement of Geomembrane Sheets

The Monitoring Engineer observed all operations relative to the placement of the 6.75 m (22.5 ft) wide geomembrane sheets. Monitoring included:

- checking delivery tickets and Geomembrane Manufacturer's quality control reports to verify that the geomembrane rolls received on-site met the Project Specifications,
- inspecting the clay subgrade to verify the Geomembrane Installer's acceptance of the smooth, firm condition on which the geomembrane was to be placed,
- verifying that the geomembrane placement plan was being followed,
- observing that geomembrane anchor trench construction met the design drawing details,
- checking that the required overlap of adjacent geomembrane sheets was being achieved, and
- checking to verify that the details of sump area grades and construction were properly achieved.

Seam Welding Procedures

The Monitoring Engineer was reponsible to verify that Project Sepcifications relating to seam welding procedures were followed. Monitoring included:

- checking that each thermal bonded extrusion seam welding unit had achieved the proper extrudite temperature, and that welding was performed at the proper speed, and
- performing periodic field testing of initial test seams (startup seams) to further verify the condition of the welding unit and welder.

Quality of Seams

The quality of each seam was verified by the Monitoring Engineer. Monitoring included:

- observing the weld material quantity, seam straightness, and seam uniformity,
- observing the Geomembrane Installer's vacuum testing of all seams,
- observing the Geomembrane Installer's vacuum testing of all seam repairs, and
- reviewing results of laboratory testing of startup seam coupons and of field cut coupons of the in-place liner.

Damage of Geomembrane Sheets

After completion of the geomembrane installation, the entire liner surface area was washed of wind blown soil. The Monitoring Engineer was responsible for:

- observing the geomembrane washing procedures, and
- locating defects caused by accidents and weather related actions.

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The Use of Geomembranes for Phosphogypsum Impoundments in South Africa

Geomembrane liners and geotextiles have been used at four phosphogypsum tailings impoundments designed and constructed by the authors at fertilizer plants in South Africa. This paper describes the legal and engineering factors and requirements for preventing pollution of the regional groundwater by seepage for these impoundments.

The four case histories of phosphogypsum impoundments are presented to illustrate various facets of the principles of geomembrane use, design of impoundments using geomembranes and their performance.

In the first case history, a liner was not utilized as it was considered originally that sub-site material was relatively impermeable and in the second case history a clay liner only was used. In both cases, a geomembrane liner was installed on subsequent extensions to these impoundments after seepage had been detected from the disposal facility. In the latter case, this was due to chemical attack by hydrofluoric acid on the clay liner. The third case history describes disposal of neutralized phosphogypsum and the fourth case history is for an impoundment where a geomembrane was exclusively used.

INTRODUCTION

Calcium sulphate or phosphogypsum is a byproduct of the wet process manufacture of phosphoric acid for use in the fertilizer industry. Over four million tonnes of phosphogypsum tailings are produced annually in South Africa; they are deposited at impoundments constructed by upstream methods.

Hydrofluoric acid and other waste products in the phosphogypsum, if they leave the impoundment in water seeping from the facility, may pollute surface water and groundwater. South African laws governing waste discharge and water quality make it necessary to neutralize the phosphogypsum before deposition, or site the impoundment on a natural, low permeability layer, or place a geomembrane liner beneath the impoundment.

This paper discusses:

- regulations governing the disposal of phosphogypsum
- the chemistry of phosphogypsum and the implications to disposal
- the availability and properties of geomembranes
- four case histories at which phosphogypsum has been deposited either in a neutralized state or on geomembrane liners.

CHEMISTRY OF THE MANUFACTURING PROCESS

The wet process manufacture of phosphoric acid involves crushing the phosphatic ore and treating with sulphuric acid. This yields orthophosphoric acid, calcium sulphate and hydrofluoric acid as follows:

$$Ca_5F\ (PO_4)_3 + 5H_2SO_4 + X\ H_2O \rightarrow 3H_3PO_4 + 5(CaSO_4.X/5\ H_2O) + HF$$

The value of X is a function of the manufacturing process (the temperature and concentration of the feedstock used). This results in one of three forms:

Dihydrate Product	X = 10
Hemihydrate Product	X = 2.5
Anhydrite Product	X = 0

Over ninety-five percent by volume of the product is phosphogypsum, which is washed to remove most of the H_3PO_4. The hydrofluoric acid renders the phosphogypsum tailings highly acidic, with a pH<2.

IMPLICATIONS FOR WASTE DISPOSAL

Any strongly acidic waste material presents potential disposal problems, both as regards the disposal facility design and operation, and as regards the secondary or resultants effects -- seepage or runoff impact. Phosphogypsum is no exception and is, in many respects, a particularly difficult waste in that it contains an extremely aggressive acid, hydrofluoric acid.

Two factors to be considered in the design and operation of phosphogypsum tailings impoundments are:

- the pollution potential of the waste
- chemical attack of the waste on natural construction or liner materials.

Each of these is discussed below.

POLLUTION POTENTIAL

In addition to their low pH, acid wastes have high levels of other dissolved chemical species in their liquid phase. In phosphogypsum, these are derived from unreacted feedstock, unreacted reagents, and the products of the chemical reactions. As many species are highly soluble, they appear in the liquid phase of the phosphogypsum slurry and are chemically mobile when the tailings are deposited at the impoundment. Seepage from the impoundment is liable, therefore, to have a significant pollution potential.

The actual pollution load to surface water or groundwater is the product of the pollutant concentration and the seepage volume; nevertheless the loss of even relatively small volumes of phosphogypsum slurry liquid may have a major impact on the receiving waters.

If the tailings are neutralized, the concentration of the dissolved species in the neutralized product is much lower. This indicates the effectiveness of neutralization in reducing potential contaminant levels and hence the pollution potential. Thus neutralization is a possible way to control environmental pollution from a phosphogypsum waste disposal facility which may negate the need to line the impoundment.

CHEMICAL ATTACK ON NATURAL MATERIALS

Weathering and degradation of natural materials in the environment is generally very slow. Most natural materials are relatively resistant to normal weathering by the acid attack of dissolved carbon dioxide carbonic acid. Natural materials are normally primarily silicates or aluminosilicates and resist the attack of strong acids, such as hydrochloric or sulphuric acid for a long time. Hydrofluoric acid, however, has the unique capability to dissolve silicates as shown by the following reaction:

$$4\ HF + SiO_2 \rightarrow SiF_4 + 2H_2O$$

which hydrolyses to

$$3\ SiF_4 + 2H_2O \rightarrow 2H_2SiF_6 + SiO_2$$

The main product is hydrofluorisilicic acid, an aqueous phase that often appears as a semi-gelatinous, colloid material in attacked silicates.

The environmental consequences of hydrofluorisilicic acid attack can be serious. Natural materials used to line impoundments to prevent pollution are attacked. Their physical properties change: a clay that is relatively impermeable, cohesive, and structurally strong becomes permeable, cohesionless, and weak. The result is:

- seepage from the impoundment of a highly polluting liquid through the degraded liner
- physical failure of starter dikes, embankments, and solution trenches.

REGULATIONS GOVERNING PHOSPHOGYPSUM DISPOSAL

The engineering requirements for a phosphogypsum impoundment are generally dictated by the need to satisfy current regulations in terms of effluent seepage quality from the impoundment and its potential impact on water sources (abstractions) or resources.

In the former case, effluent discharges are controlled by regulations of the Water Act No. 56, 1954, as amended (R553, 5 April 1962). These define "general" effluent quality limits for the majority of South Africa, but where surface waters are of particularly high quality, a stricter set of limits, the "special" standards, are applied. If drinking water supplies are liable to be prejudiced by phosphogypsum disposal, the SABS-241 (1971) standards are used.

The implications of these standards for a range of typical parameters problematic in phosphogypsum wastes are illustrated in Table 1. This includes values for pH, total dissolved solids, sulphate, phosphate, and fluoride for both neutralized and unneutralized wastes compared with the current standards. As can be seen, the concentrations in the wastes themselves are many hundreds or thousands of times greater than these standards and hence seepage control measures, such as geomembranes, are essentially for impoundments.

Table 1
Comparison of dissolved species in phosphogypsum wastes with current water quality regulations in South Africa

Parameter	Values in ppm		
	Normal "Acid" gypsum	"Neutralized" gypsum	Effluent discharge standards[1] or drinking water standards[2]
pH value	1,2 - 2,0	6,0 - 9,0	5,5 - 9,5
Total dissolved solids	20 000 - 30 000	3 000 - 4 000	Intake + 500
Sulphate	4 000 - 13 000	500 - 1 500	250/400[2]
Phosphate	500 - 12 000	15 - 100	(2-10)[3]
Fluoride	300 - 1 500	10 - 30	1,0

1 Typical values for South Africa.
2 Drinking water standards used where there is no generally accepted effluent standards for that parameter; both recommended limits and maximum allowable values given.
3 No overall standards.

AVAILABILITY AND PROPERTIES OF GEOMEMBRANES

It is apparent, from the preceding discussion, that geomembranes may be necessary to limit or eliminate seepage from waste impoundments in order to meet current environmental, water quality regulations. Compared with North America, there is a limited range of geomembranes suitable for lining waste impoundments available in South Africa. These include:

- low density polyethylenes
- elastomeric polyolefin alloys (based on HDPE)
- polyvinylchlorides
- high density polyethylenes
- butyl rubbers.

A summary of the geomembranes currently in use in South Africa is given in Table 2, with approximate costs in rands (1 U.S. dollar being equivalent to R1.20). Each of these materials has advantages and disadvantages in terms of their properties and costs. A comparison of geomembranes potentially suitable for phosphogypsum impoundments is given in Table 3, where such advantages and disadvantages are noted. Comments on the suitability and performance of materials is based on practical experience of the materials and not necessarily on the manufacturers' specifications of their products. In this sense, therefore, some of the comments may appear to be arbitrary rather than based on exact laboratory test data but reflect real problems or benefits in the practical uses of the materials.

CASE HISTORIES

The four case histories chosen illustrate a number of features related to phosphogypsum disposal and the use of geomembranes in seepage control:

- acid tailings discharge and the associated water pollution control measures
- chemical attack by hydrofluoric acid on natural materials
- the disposal of neutralized tailings over a natural clay and the low pollution potential of neutralized tailings
- effect of liner strength on the stability of a phosphogypsum embankment
- environmental control of acidic wastes using geomembrane liners.

CASE HISTORY 1

The phosphogypsum disposal facility is on highly weathered igneous rocks. No liner was installed as an initial assessment had concluded that the weathered rock was sufficiently impermeable to prevent groundwater contamination. Regular environmental monitoring disclosed discolouration of vegetation downstream of the impoundment; a detailed hydrological and hydrogeochemical investigation indicated that a near-surface, perched groundwater table was contaminated by seepage from the impoundment. The water displayed elevated levels of sulphate, phosphate, fluoride, low pH, and high TDS values.

Possible methods considered to control pollution (by intercepting seepage in order to eliminate near surface aquifer contamination) included: a passive cut off curtain; a well point system; and a permeable interceptor trench linked to a pump sump. The design criteria established for the system included the following requirements:

- effective interception of seepage
- installation of the system should not cause secondary pollution
- materials used should be resistant to acid attack and corrosion
- low operating costs
- minimal long term maintenance.

Figure 1 shows the system adopted. The trench incorporates an acid resistant geofabric and inert 19 to 40 mm stone. The geofabric prevents ingress of fine material to the stone. The trenches lead to a sump from which seepage is returned either to the tailings impoundment or the plant to reslurry the phosphogypsum waste.

Subsequent extensions of the disposal facility incorporate a geomembrane liner. This liner will result in a significant reduction of the contaminant load to the near-surface aquifer. The discussion in a previous section on liner properties is based on the evaluations undertaken to select the liner actually used at this site.

CASE HISTORY 2

After the summer rains, seepage was observed along the toe dike of the phosphogypsum disposal facility; seepage continued with the onset of the dry season. In order to determine if the seepage was due to blocked drains, the drains were exposed. During this examination the clay liner in the area of the drains was found to have deteriorated.

In some places the clay liner was hard and brittle as a result of loss of plasticity. In other places, the liner had degraded to a red-brown gelatinous material. The whole impoundment is lined with the same clay; concern was expressed that seepage had chemically attacked the liner, caused it to degrade, and hence increasing the seepage causing groundwater contamination.

The hydrogeology and hydrogeochemistry of the groundwater beneath and in the vicinity of the impoundment was investigated by drilling a series of boreholes, installing piezometers, and taking water quality samples. This investigation showed that the permeability of the natural materials beneath the impoundment decreases with depth. Some contamination was found in the upper piezometers, but the deeper groundwater was uncontaminated: seepage from the impoundment was reaching near-surface groundwater, however, and was causing moderate contamination as a result of the failure of the natural clay liner.

Table 2
Summary of Geomembranes Used in South African Impoundments

	Low Density Polyethylene	Elastomeric Polyolefin Alloys				P.V.C.		HDPE		Butyl Rubber	
Thickness (μm)	500	500	750	1000	1500	500	750	1500	2000	1000	1600
Method of Jointing	Mechanical Clamping	Continous Extrusion				Adhesive		Continuous Extrusion		Vulcanizing	
Specific Gravity	0.94	0.95	0.95	0.95	0.95	1.32	1.32	0.95	0.95	1.2	1.2
Tensile Strength at Break (MPa)	14.5	15	15	15	15	19	19	21	21	9	9
Breaking Strength (N/25mm)	175	316	410	520	880	202	303	800	966	245	391
Elongation at Break (%)	550	600	600	600	600	320	320	600	600	470	470
Puncture Resistance (N)	19	24	38	48	71	23	N.A.	81	105	25	40
Tear Strength (N)	43	53	84	107	178	23	30	250	330	40	65
Abrasion Resistance (Taber Abrader/1000 Cycles)	36	12	12	12	12	46	46	11	11	86	86
Widths (m)	9	6.8	6.8	6.8	6.8	1.37	1.37	6.8	6.8	1.2	1.2
Lengths (m)	100-250	250	200	150	90	50	N.A.	90	70	Modules of 20 m x 30 m	
Estimated comparative price*(R/m²)	4.50	5.50	6.50	8.00	11.00	4.00	N.A.	10.00	N.A.	10.00	16.00

*Based on areas of 10 000 m²

Table 3
Comparison of Geomembranes for Phosphogypsum Impoundments

MATERIAL	ADVANTAGES	DISADVANTAGES
Low density polyethylene 500 μm	1. Low relative cost (R4.50/m^2) 2. Lightweight (0.47 kg/m^2) 3. Good weathering and chemical resistance properties. 4. No plasticizer required for flexibility. 5. Low permeability. 6. Wide widths. 7. Free from pinholes.	1. Vulnerable to mechanical damage - low strength properties. 2. Joints have to be buried. 3. Cross joints not possible. 4. Low abrasion resistance. 5. Mechanical joints (stripseal method).
Elastomeric polyolefin alloy 500 and 750 μm	1. Lightweight (0.48 & 0.71 kg/m^2) 2. Good seam strength with extrusion weld (shear = 95% tensile). 3. Good weathering and chemical resistance properties. 4. No plasticizer required for flexibility. 5. Low permeability. 6. Wide widths. 7. Free from pinholes. 8. Easily repaired. 9. Good puncture resistance 10. Few joints, quick laying rate.*	1. Medium relative cost (R5.50 to 6.50/m^2)+ 2. Joints have to be sandpapered prior to extrusion welding to clean and roughen joint surfaces. 3. Low abrasion resistance.
Elastomeric polyolefin alloy 1000 and 1500 μm	1. Good seam strength with extrusion weld. 2. Good weathering and chemical resistance properties. 3. No plasticizer required for flexibility. 4. Low permeability. 5. Wide widths. 6. Free from pinholes. 7. Easily repaired. 8. Good puncture resistance.	1. High relative cost (R8.00 to R11.00/m^2). 2. Slow laying rate. 3. Joints have to be sandpapered prior to extrusion welding. 4. Preheating of joint in front of welder is necessary. 5. Low abrasion resistance. 6. Moderate weight (0.95 and 1.43 kg/m^2).
Polyvinylchloride (PVC) 500 μm	1. Lowest relative cost (R4.00/m^2). 2. Lighweight (0.6 kg/m^2). 3. Highest specific gravity, i.e. will not float. 4. High abrasion resistance. 5. Joint surfaces can be cleaned with chemical solvent. 6. Quick laying rate with factory welded 30 m wide strips.*	1. Poor seam strength (shear = 37% tensile). 2. Moderate weathering properties - vulnerable to hail damage and no guarantee against U/V light with local material. 3. Plasticizer required for flexibility. 4. Loss of plasticizer with time, even when buried. 5. Difficult to repair (?)
High density polyethylene (HDPE) 1500 μm	1. Good weathering and chemical resistance properties. 2. No plasticizer required for flexibility. 3. Low permeability. 4. Free from pinholes. 5. Excellent puncture resistance.	1. High relative cost (R10.00/m^2). 2. Low flexibility. 3. Difficulties with extrusion welding. 4. Joints have to be sandpapered prior to extrusion welding. 5. Preheating of joint in front of welder is necessary. 6. Low abrasion resistance. 7. Moderate weight (1.44 kg/m^2).
Butyl Rubber 1000 and 1600 μm	1. Good seams with hot bonding. 2. Joints can be cold vulcanized. 3. Excellent weathering and chemical resistance properties. 4. No plasticizer required for flexibility. 5. Low permeability.	1. Highest relative cost (R10.00 to R16.00/m^2). 2. Moderate/heavy weight (1.2 to 1.92 kg/m^2). 3. Moderate puncture resistance. 4. Small panels - slow laying rate.

*Approximately similar laying rates.

To minimize the contaminant load to the groundwater, deposition of phosphogypsum was stopped on those areas where contamination was found to be worst. A new facility was constructed, and the new facility incorporated a geomembrane liner in order to eliminate seepage to the groundwater. The previous discussion on liner properties is based also on the evaluation undertaken to select the new liner for this facility.

CASE HISTORY 3

The site selected for this phosphogypsum disposal facility is a low lying alluvial marsh area, underlain by soft silts and clays. The owners of the site were eager to use the phosphogypsum as a lightweight landfill to reclaim the area; natural fill material is in short supply and the reclaimed land will be valuable for light industrial development.

In order to reclaim the area in the long term and to preclude adverse consequences of an accidental discharge of effluent to the surrounding marsh, the tailings are neutralized before deposition.

Free silica in the phosphate rock reacts with the hydrofluoric acid to form stable silica compounds and hydrofluorsilicic acid. Fluorine is removed from the reaction by the precipitation of calcium fluoride and chuckhrovite, both of which have a low solubility and are stable. Neutralization is controlled at a pH between 7.0 and 8.0; at this pH the fluorine content of the supernatant liquor at the impoundment is about 30 ppm.

As noted above, the impoundment is underlain by thick alluvial clays of high plasticity. The silty clay has a clay content varying from 10 to 30 percent. The plastic limit varies from 20 to 30, the liquid limit from 30 to 58 and the PI from 10 to 30. The in situ moisture content is generally higher than the liquid limit. The soil plots as a clay of low to high plasticity.

The upper 5 m of the profile are unconsolidated; below this the average undrained, unconsolidated strength varies from 10 to 15 kPa. The clays are thixotropic and have a sensitivity of about 5. From triaxial tests the average measured C_v is 0.25 mm^2/min. Constant lead permeability tests give an hydraulic conductivity of 8×10^{-10} m/sec.

The silty clay is underlain by a medium dense poorly graded silty sand (SP). This layer is a confined aquifer, with an effective water pressure about 2 m above the ground level.

The low permeability of the foundation subsoils and the confined aquifer effectively limit downward seepage to the groundwater. Seepage through the embankments is collected in drains, a solution trench, and a return water pond from which water is returned for reuse in the plant.

Regular monitoring around the impoundment shows no detectable pollution.

CASE HISTORY 4

Originally the impoundment was built simply from the sketch of a foreman. The impoundment was constructed on sloping ground (at about 7.5° to the horizontal) and the area was lined with PVC.

The authors became involved when requested to recommend continued deposition procedures. Potential instability of the downslope embankment by sliding of the gypsum along the PVC was soon identified as a possible problem.

Values from laboratory strength tests for gypsum gave C = 0kPa, $\emptyset = 23^{\circ}$ for a saturated remoulded sample and C = 13kPa, $\emptyset = 30^{\circ}$ for a saturated sample, perpendicular to the bedding plane of the gypsum. Values for the shear resistance at the interface of the gypsum and the plastic were also determined. The test involved the use of a standard shearbox, the lower half of which was filled with a metal block. On the surface of the block, the plastic was secured so that the shearing plane coincided with the plastic to gypsum interface. Depending on test condition, average values of: C = 0kPa, $\emptyset = 22^{\circ}$; C = 12kPa, $\emptyset = 32\frac{1}{2}^{\circ}$; and C = 17, $\emptyset = 30^{\circ}$ were obtained.

The original foreman-designer had recognized that sliding of the embankment along the surface of the plastic might occur. To prevent this, he constructed a series of "keys" -- long rows of earth piled up as shown in Figure 1. Those ground asperities, provided they do not shear, increase the resistance to sliding along a potential failure plane.

The increase in sliding resistance is quantified by $\emptyset e$ which is the effective angle of friction along the failure plane. $\emptyset e$ is given by:

$\emptyset e = \emptyset b + \lambda$ where $\emptyset b$ is the basic angle of friction on the plane (i.e. of the plastic gypsum interface)
λ is the wave angle.

The wave angle is given by:

$\lambda = \tan^{-1} \frac{2V}{\bar{L}}$ where V and L are as defined in Figure 1.

Where a plane profile is not one of true waves but consists of "asperities" at regular intervals, as occurs at the impoundment, it may be shown that:

$$\lambda = \bar{\lambda} + \lambda^{1} (\bar{L}/L^{1})$$

where $\bar{\lambda} = \tan^{-1} \frac{2\bar{V}}{\bar{L}}$

$\lambda^{1} = \tan^{-1} \frac{2V^{1}}{L^{1}}$ as defined in Figure 1.

For the potential sliding surface an average, and hence effective, value of λ of 3° was calculated.

To analyze the factor of safety of the gypsum embankment against sliding, Tergaghi Wedge type analyses and Morgenstern and Price analyses were performed.

Figure 2 shows that if construction of the embankment had proceeded at its existing angle (that is 20°), a maximum height of about 14 m could be achieved before the factor of safety fell below an acceptable minimum of 1.3. As shown on Figure 2, a drain had been established back from the toe. If the slope of the embankment were simply flattened, the phreatic line, which occurs when the pool is in its normal operating position, would emerge at the face. The resulting saturated conditions and seepage would be unacceptable.

Accordingly, after many iterations the solution shown in Figure 3 was recommended and has been implemented. New drains were installed, as shown, on the tailings surface at the elevation as it existed at the time, and also on a divider wall. For this geometry, the factor of safety is above 2.0 for potential failure surfaces involving the full height of the embankment.

An intermediate step back of 20 m was effected. The second step back was effected when the embankment height had increased a further 13 m. Deposition and construction to the final height continues. Movement monitoring beacons on the embankment are surveyed regularly, and, to date, indicate no unacceptable movement on the geomembrane.

CONCLUSIONS

It is apparent from the foregoing discussion that unneutralized phosphogypsum tailings have a high potential for surface water and groundwater contamination. Should it prove impossible to site a disposal facility on natural impermeable materials, a geomembrane is often required to minimize seepage losses and hence comply with current water quality standards in South Africa.

The comparison in the specifications and costs of geomembrane materials available in South Africa reflects the inevitable trade-off between application and costs, and the degree of environmental protection required. In practice, the latter is essentially site specific and hence no one geomembrane can be recommended for all applications.

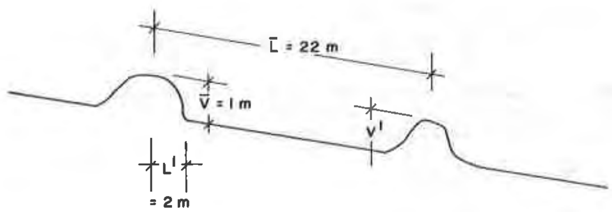

FIGURE 1 GEOMETRY OF 'KEYS' TO PREVENT SLIDING

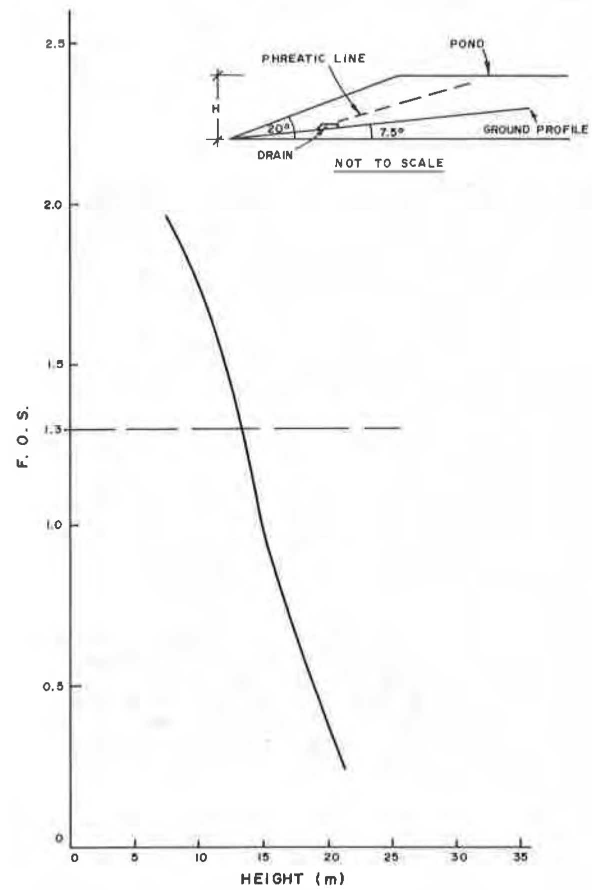

FIGURE 2 HEIGHT vs F.O.S. FOR IMPOUNDMENT

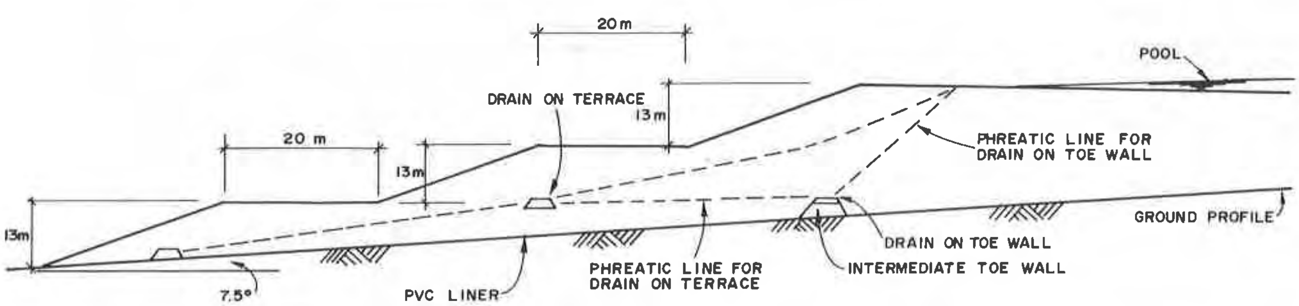

FIGURE 3 GEOMETRY OF EXTENDED PHOSPHOGYPSUM EMBANKMENT ON PVC LINER

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Guidelines for the Installation of Geomembranes for Ground Water Protection in West Germany

Lining membranes have been employed in West Germany in the field of ground water protection for more than ten years. The legislative background for mandated ground water protection is the Water Resources Act, which over time has been augmented by documents issued by various German institutions and which detail specific requirements for geomembranes in specific applications. This paper discusses those "Guidelines" which have served to bring lining technology to a level of sophistication that assures strict compliance with the intention of the law.

I INTRODUCTION

Geomembranes have been employed in ground water protection projects in West Germany for more than ten years. During the first five years, from about 1973 to 1978, roughly one million square meters in more than 50 projects in Germany were lined, mainly in the field of domestic and industrial waste disposal facilities (1). Two-thirds of these installed liners are high density polyethylenes, and the rest are modified polyolefins with other thermoplastic materials accounting for a very small percentage. During the next five years the use of geomembranes multiplied until by 1983, four to five million square meters had been installed. In 1983 alone, one million square meters were lined, with HDPE or modified polyethylene being employed almost exclusively.

Experience from those projects already carried out, as well as results from long-term tests, show that a reasonable level of ground water protection has been and will be effected with geomembranes, if the material and dimensioning are chosen appropriately, and if careful quality assurance is guaranteed during production and installation. Specialized geomembrane suppliers have worked hand in hand with various German institutions in developing technical guidelines which have contributed to the presently achieved level of technology.

II LEGAL BACKGROUND

The Water Resources Act (WHG) (articles 1, 19 and 34) constitutes the legal background for ground water protection measures in West Germany. The noteworthy sections of the Act are:

- Article 1a, paragraph 2, of the Water Resources Act requires anyone who undertakes any act which could impair water resources to exercise reasonable skill and care to prevent any contamination of the water or any other detrimental change to its properties.
- Article 19 g (1 and 2) of the Act requires that pollutants be so treated that ground water contamination and any other detrimental change are avoided.
- Article 34, paragraph 2, mandates that all materials be stored and held in such a manner that ground water contamination and any other detrimental change in its properties are avoided.

Ground water contamination has, however, occurred, sometimes as the result of willful disregard of the law, but more often from honest errors in handling, storing and transporting polluting media, as well as from inappropriate storage of domestic and industrial wastes. This situation shows that the technology for preventing ground water contamination, and thus achieving compliance with the legal provisions, was either not available or was little understood.(2)

Based on these experiences the responsible authorities started to work out more precise standards with stricter requirements for waste disposal facilities related to their location, the total liner system and the lining material.

III WASTE DISPOSAL FACILITIES

Two documents on the subject of waste disposal are discussed in this paper. They show the path of development to the present, more stringent requirements. The published guidelines are:

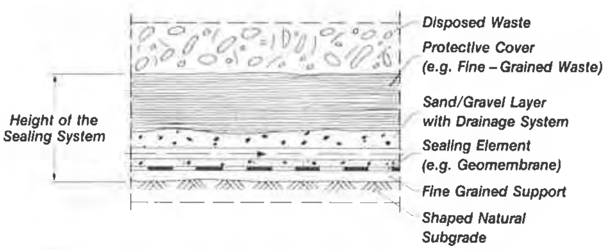
- "Waste Disposal Instructions" of the LAGA (3)
- Guideline of the State of North Rhine Westphalia NW (4) "Disposal Base Linings Made of Liner Membranes".

1. "Waste Disposal Instructions"

LAGA is a committee whose members represent water authorities from all West German states. This group has worked on general guidelines and instructions for waste disposal for years. The most recent edition of the "Waste Disposal Instructions" was issued in 1980, although this instruction deals only generally with the design of a base liner system for a disposal facility.

The 1980 Instructions require special lining measures to prevent or minimize seepage of leachate into the subgrade. Decisions about specific lining requirements are made on a case by case basis by the responsible authorities: the state institutions of water and waste, and/or the water subauthorities.

A geomembrane is one of the four elements of a lining system, as set out in the Instructions. The other three components are fine-grained support for the geomembrane (laid over a shaped subgrade), a sand and gravel layer over the geomembrane, and a protective cover. A diagram of a lining system structure is shown in Figure 1.



Normal lining system for disposal base sealing with geomembranes (3)

FIGURE 1

The Disposal Instructions recommend that any geomembrane, depending on the particular project, undergo special tests for the material and its jointing technique. The document also stipulates that a lining membrane must maintain operational effectiveness until such time as there is no more leachate to be treated.

2. Base Lining Systems Guidelines

The need for more specific liner requirements than are found in the general Instructions led to a special guideline for "Base Lining Systems with Liner Membranes for Disposals", developed by the State of North Rhine Westphalia and issued in 1982 as a draft document. It will soon be issued in final form by the Ministry of Food, Agriculture and Forestry in Dusseldorf, State of North Rhine Westphalia (5). Detailed requirements for liner performance during installation and operation of a disposal facility are set out in this guideline.

The document deals in its main sections with:

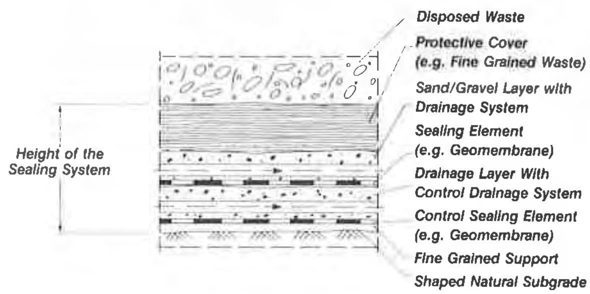
- general technical principles;
- possible loadings of liners;
- requirements for liner membranes and their testing and quality assurance; and
- planning of lining systems and execution of installation.

The guideline also offers a summary of the present level of technology and discusses technical possibilities for systems lined with geomembranes. It discusses, for the first time, the entire issue and will be used by system designers and permitting authorities to assist in thorough and professional decision-making.

A. General Technical Principles

In addition to the standard lining system described in the LAGA "Instructions" and shown in Figure 1, this guideline describes a maximum security lining system which might be necessary under certain conditions, such as when the immediate location of leak points is essential. In comparison with the standard lining system, this maximum security lining system contains, as shown in Figure 2, two additional elements:

- a second geomembrane, and
- a draining layer with a control drainage between the liners.



Maximum lining system at disposal base sealing with geomembranes (4)

FIGURE 2

The entire lining system is required to provide a long lasting operation. This means that any leachate must be intercepted safely and drained properly during the expected period of service. The material and the jointing technique thus must be resistant against any harmful substances, and their quality must be continuously tested during production and installation.

B. Possible Loadings of the Liner

A distinction regarding loadings must be made between physical, chemical and biological influences, although normally all three of them could occur together. Concerning physical stresses, loads during the installation as well as during the operation of the disposal facility have to be taken into consideration when the material is selected. During installation the geomembrane has to survive short-term loadings. Long-term loadings are to be expected later, during operation of the facility. For instance, long-term settling of the ground under the liner and/or the contents of the disposal facility can exert forces on the liner. Additionally, there are so-called general physical loadings such as UV radiation, temperature changes, and sustained high or low temperatures.

The chemical loadings depend on the substances to be disposed in each particular project, which can be highly concentrated or diluted aggressive media and generally, leachate or gases from the disposal. These influences on the lining material selected also have to be tested specifically for the project.

Microorganisms, vegetable matter such as certain types of roots, and higher animal organisms like rodents also may influence the performance of geomembranes.

The specification regarding all types of loadings is still limited to a qualitative standard. An exact quantitative specification for the degree(s) of loading is, in most cases, not yet possible.

C. Liner Requirements

The NW guidelines correlate liner requirements with possible loadings. Because these loadings cannot be determined exactly, however, the quantification of the degree of stress is not always resolved. Nevertheless, a clear summary of requirements has been compiled; it offers assistance in the decision making and material selection process of those authorities engaged in the planning of disposal facilities. Besides the special requirements mentioned in the guideline, liner materials must, of course, meet the respective material standards. Some of the physical and chemical requirements for geomembranes are shown in Table I. The characteristics of "minimum thickness" and "tensile strain behavior" are given special attention here.

TABLE I
Some Selected Requirements For Geomembranes (5)

Characteristic Value	Testing Value	Requirements	Test Methods
General Physical Requirements			
Thickness	Minimum Thickness Average Single Value	≥ 2 0 mm Minimum Thickness ≥ – 10% + 10% Referred to Average	DIN 53 370
Water Absorption	Change in Weight	≤ 1 0 weight – %	DIN 53 495
Special Physical Mechanical Requirements			
Tensile Stress Behavior	Uniaxial: Tensile Force at 5% Elongation	≥ 400 N/50 mm	DIN 53 455
	Biaxial: Elongation at Break	≥ 10%	Diameter of Test Specimen = 1 00 m Burst Test
Tear Resistance	Tearing Force	≥ 200 N	DIN 53 515 DIN 53 363
Puncture	Critical Drop Height	≥ 750 mm at 500 gr	DIN E 16 726
Seam Strength	Welding Factor	≥ 0 9 for semi-crystalline high polymers ≥ 0 6 for amorphous high polymers	
Chemical Requirements			
Resistance Against Highly Concentrated Chem Agents	Change in Weight	≤ 15%	DIN 53 521
	Change of Mechan Properties	≤ 25%	Din 53 455
Resistance Against Diluted Fluid Agents	Change in Weight	≤ 10%	DIN 53 521
	Change of Mechan Properties	≤ 20%	DIN 53 455
Resistance Against Seepage Water of Domestic Waste Disposals	Change in Weight	≤ 5%	DIN 53 521
	Change of Mechan Properties	≤ 20%	DIN 53 455

The minimum thickness of 2.0mm - independent of the liner membrane material - is specified, based primarily on the loadings during installation which cannot be determined in the laboratory, and thus can only be approximated for a particular site. The specification requires that the liner retain its impermeability -- no mechanical damage can exist when the installation is finished. This minimum thickness is seen as a basic requirement and may even have to be increased if the material has relatively low mechanical properties. It cannot be decreased (as for instance, when the liner is reinforced) because under certain loading conditions, such as point penetration, reinforced liners do not necessarily exhibit better mechanical behavior.

The liner thickness is also important to behavior under tensile load as could occur, for instance, during settlements inside and under the body of a disposal facility.

Two minimum requirements have been determined concerning liner behavior under a tensile load:

- The liner must be able to absorb 400 N at an elongation of 5% and a width of 5 cm during the uniaxial tensile test, and
- The deformation at break must range above 10% during the short-term bursting-pressure test (biaxial tensile load).

These properties mark a new development in the evaluation of liner membranes. A flexible geomembrane with a relatively high elongation at tear was once considered particularly suitable, for instance, in water construction projects. Today, however, the opinion prevails that geomembranes must be able to distribute locally-occurring deformation to a wide area, even under extreme loads. This is possible only if the liner has an adequate thickness and corresponding high modulus of elasticity, together with sufficient deformation reserve (6). These minimum requirements are guaranteed by the values required for the uni- and biaxial tensile loads.

IV CATCHMENT BASINS

The application of geomembranes for catchment basins requires approval by the subordinate water authorities (StAWA).

Unlike the application of geomembranes at the base lining of disposals, a liner is employed in this application area as an additional safety measure only. It ensures that contamination of the ground water does not occur as the result of accidents or unforeseen incidents at facilities where water-polluting fluids are handled. The loading period is limited to a maximum of three months, and thus generally lower requirements than those for disposal linings are justified.

The Institute für Bautechnik (IfBt) (Institute for Construction Engineering), Berlin, acting for all German states, has been engaged in the authorization of suitable materials for handling, storing and transporting water-polluting fluids. These materials are approved and installed according to "Construction and Testing Principles for Liner Membranes" (7).

Geomembranes used as liners for catchment basins need to have a test certificate which is granted on the basis of discussions and recommendations by an experts' committee. These recommendations are guided by the standard regulations, i.e. the "Construction and Testing Principles".

Tests for proving the suitability of geomembranes relate also to their general physical properties, the reaction to media with which the liner could come in contact, and reaction to biological attacks. In addition, the flammability and the electrostatic discharge capabilities must be proved.

It is the purpose of these tests to specify the liner for requirements (8):

1. General physical properties
   Special material standards (DIN) are relevant regarding the mechanical properties of the material. Special tests may be required, such as the resistance to environmental stress cracking for polyethylene.

   Concerning the specific material requirements for geomembranes, only those properties are applicable which are tested according to the presently valid material standards. Four material groups are considered:

   - Elastomers
   - Polyethylene
   - Polyisobutylene
   - PVC non-rigid

   Some uncertainties shall be avoided by determining a relatively low minimum thickness of 1.0 mm. However, only HDPE liner membranes have been approved to date.

2. Chemical Resistance

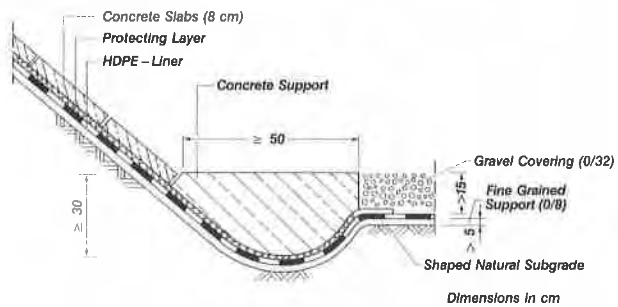
   Normally, loading conditions on geomembranes do not last longer than three months, which explains the relatively high admissible change in mechanical properties of up to 25%. This value can be compared with the geomembrane requirement for disposal base linings against concentrated media.

3. Other Properties

   Geomembranes are required to be resistant against attacks by rodents, microorganisms, roots and plant growth.

   Here also there are lower requirements than for disposal base linings. For example, changes in the modulus of elasticity of up to 50% are admissible regarding resistance to microorganisms.

   To fulfill the requirements regarding flammability, the lining membranes must be covered with mineral materials. Figure 3 shows a cross-section of a liner in the transitional area between bottom and embankment. For storing combustible fluids, the volume resistance must not exceed $10^8 \Omega$, and the surface resistance must not exceed $10^9 \Omega$. If this is not possible, changes in design have to be made.



Lining system for catchment basins with geomembranes (7)
FIGURE 3

4. Quality Assurance

   Intensive testing concerning the quality assurance has to be carried out by the producers of approved geomembranes. These internal controls must be matched to the particular material. Table II shows the prescribed tests for geomembranes made of HDPE.

TABLE II
Internal Controls For Polyethylene Geomembranes (1)

1	Thickness	2 × each shift
2	General Conditions	2 × each shift
3	Stress at yield	1 × each shift
4	Behavior after heat storage	1 × each shift
5	Density of the raw material	1 × each week
6	Melt index of the raw material	1 × each week
7	Melt index of the lining membrane	1 × each week

These internal control programs are meant to prove quality consistency and keep the processing influences within predetermined tolerances.

Internal control is corroborated by the external control of an authorized testing institute. This third-party control is part of the test certificate and is agreed upon by contract.

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The Use of Geomembranes in Basins for the Storage of Materials Contaminated by Dioxin at Seveso

The paper deals with the use of geomembranes for the lining of two basins, built between 1981 and 1983 at Seveso (Northern Italy), for the safe storage of materials contaminated by TCDD (Dioxin). First of all, the paper describes the basic design of the basins, which were built in contaminated areas that had previously been cleaned up. The geomembranes were employed in the basins both for lining the bottom and the slopes and for protecting the part above ground, thus ensuring appropriate containment of the contaminated material. In particular, as regards the part below ground, an almost impermeable layer ($k=5x10^{-10}$ m/sec), composed of a mixture of aggregates and bentonite, was also laid underneath the synthetic sheets. The geomembranes used (HDPE for both the basins) were supplied and placed by two industrial firms. A description is given of the two different laying technologies, with reference to the practical problems involved in sheets welding and in its control.

1. FOREWORD

The Icmesa Company was a medium-dimension chemical plant, manufacturing intermediate materials for cosmetic and pharmaceutical industry, belonging to Hoffmann La Roche group. The plant was placed in the area of Meda Municipality, at about 20 km from Milano.

On July 10th 1976, owing to an uncontrolled exothermal process in the reactor for trichlorophenol synthesis, it occurred an increase of internal pressure which caused the explosion of the breaking disc and the spreading in the atmosphere of mixed chemical substances contained in the reactor among which the 2,3,7,8 TCDD. The toxic cloud under the wing action spread in the south-south east direction and contaminated a very large area, about 18.000.000 m^2.

This area has been divided into three zones, according to the poisonous substance concentration: A zone, B zone and R zone. In the A zone, where the TCDD concentration was higher (Fig. 1), the population was evacuated.

2. EXAMINED POLLUTION CONTROL SYSTEMS

The first hypothesis, formulated on the basis of 1976 scientific knowledge, foresaw TCDD destruction by incineration only for highly contaminated materials, while a natural decay was foreseen for the soil. But this second assumption about the soil did not prove to be true and the quantity of material to be incinerated increased enormously. Therefore the project of an incinerator was rejected for technical - economical reasons.

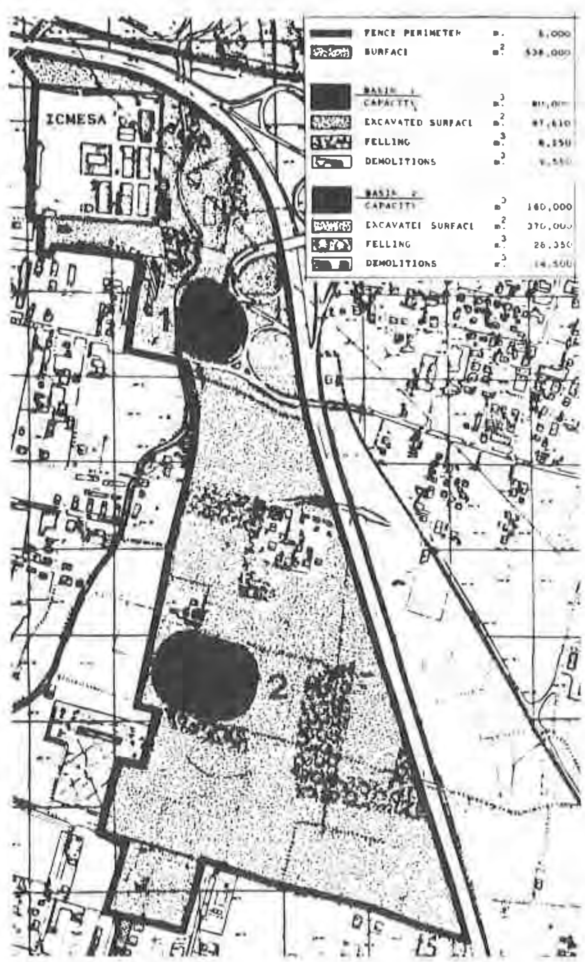

Fig. 1 - Fence perimeter of the A zone and position of the two basins

Other systems (such as TCDD extraction by means of an extractant, microorganisms TCDD destruction, ultra-violet rays use, etc.) were unsuccessful. At last, it was decided to collect the contaminated materials in basins for the safe storage.

3. BASINS CHARACTERISTICS AND DESIGN PRINCIPLES

According to the soil contamination distribution, the A zone was subdivided in order to differentiate the digging operations. In fact, the contaminated soil was removed in layers 0.20÷0.30 m thick: this operation was repeated once or more times, depending on initial distribution. This made it possible to evaluate the soil quantity to be transferred to the basins. In addition, there was the volume of buildings to be pulled down, trees, waste and draining materials stocked after previous decontaminations.
Owing to various political reasons two basins were built up. As a matter of fact, in the A zone were involved two Municipalities: Meda and Seveso, each of them had to look after its part of contaminated material. The position of the two basins is shown in Fig. 1.
The capacity of the first basin, placed in Meda Municipality, has been estimated of 80,000 m^3, whereas the second in Seveso of 160,000 m^3.
Essential planivolumetric data were as follows:

- First basin

top surface	m^2	9,300
bottom surface	m^2	4,457
maximum depth	m	9
maximum height	m	5
slope		1/2

- Second basin

top surface	m^2	21,875
bottom surface	m^2	4,480
maximum depth	m	10
maximum height	m	6
slope		1/4

According to Kays (1) and Piepoli (2), the principles followed for the basins design were as follows:
- study of geological and geotechnical conditions of the selected zones, in order to ascertain embankment feasibility;
- creation of various lining barriers, so as to prevent the poisonous substance from spreading in the environment;
- construction of a drainage system which could be easily checked;
- development of a basin monitoring system for periodic controls.

4. IN SITU INVESTIGATIONS AND GEOTECHNICAL ANALYSIS

A study on local hydrology was carried out in the area under examination (radius of about two kilometres); in this area wells, springs and streams flooding zones are determined.
In total, eight borings were carried out at different depths, in any case more than 20 m below the basin bottom. Fig. 2 shows as example the geological map and the geological cross-section in the second basin area.
The in situ investigations found the existence of a continous clay layer at 25-30 m from the basins bottom.

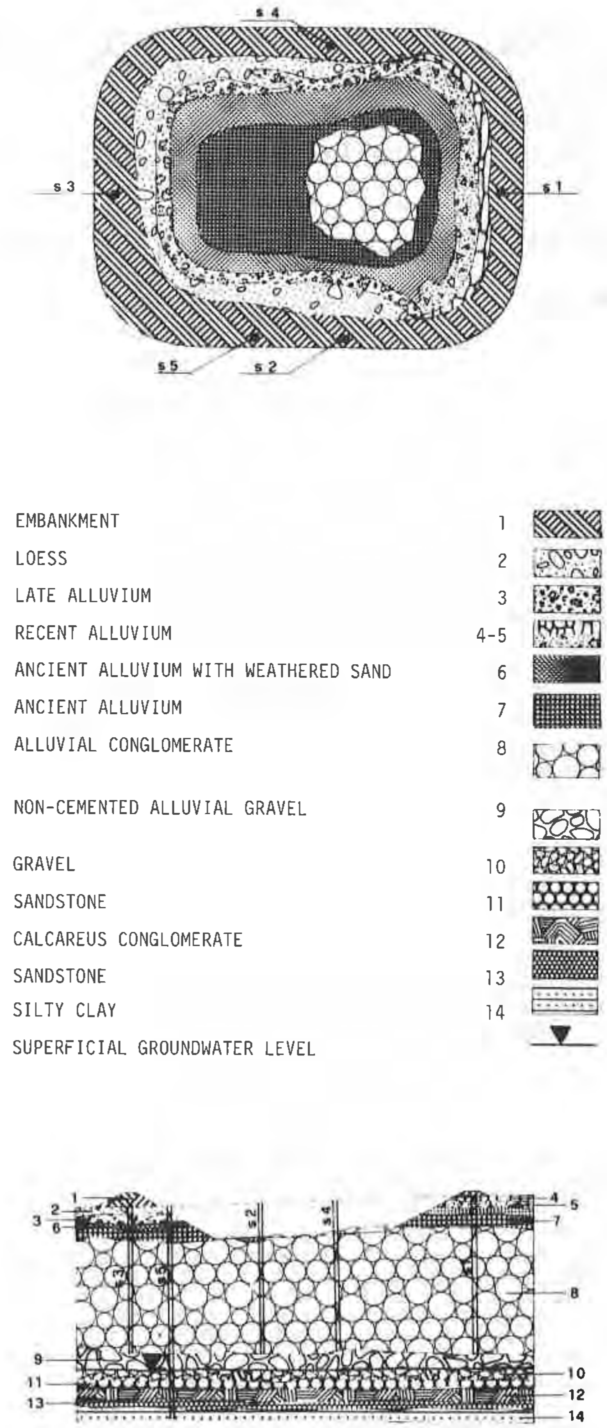

Fig. 2 - Geological map and geological cross - section of the second basin, with the positions of the five borings

The investigations showed too the presence of two ground water levels: the superficial one was at about 20 m from the basins bottom.
The height of the clay layer was sufficient to constitute a separation bed from the underlying groundwater level. This almost impermeable clay layer makes it possible, in case of need, to restrict controls to superficial groundwater level.
The in situ investigations pointed out too that the basins area mostly consists of alluvial deposits and rests on an alluvial conglomerate layer, as shown in Fig.2. It was also checked the bearing capacity, according to the zone lithology, the groundwater positions and their maximum reachable level.
With regard to soil settlements, it is possible to consider the alluvial conglomerate layer as a rigid foundation.
Therefore the basins bottom is not subjected to settlements caused in the subsoil by debris load.

5. LINING METHOD

Following the philosophy of nuclear materials storage, for the basins lining design, various barriers have been interposed to prevent the contaminating agent spread. Four kinds of barriers were considered.

5.1 The Chemical-Physical Bond between TCDD and Soil

Analyses performed six years after showed that TCDD was in a surface soil layer, 0.20-0.30 m deep, and its concentration values were identical to those found in 1976. This makes it possible to state that the contaminating agent was not mobilized by atmospheric agents.

5.2 The Filling Methods

The material has been subdivided according to the contaminating agent concentration rate, placing the highly contaminated part in the middle of the basin and the less contaminated part along the outer surface.

5.3 The First Lining Layer (0,20 m thick)

It was made of a suitably studied mixture, composed of aggregate (sand and gravel) and bentonite.
The work of mixing, laying and compacting was extremely difficult and required the elaboration of particular working methods. During the laying, a geotechnical laboratory tested daily in situ the laid materials, in order to verify their design requirements.
The results obtained by this placing method were finally satisfactory: in fact Modified Proctor density values higher than 98% and permeability values higher than 10^{-10} m/sec were recorded.
The layer of aggregate and bentonite was covered with a bitumen emulsion, as protection from dehydration or washing due to meteoric agents.

5.4 The Second Lining Layer (2,5 mm thick)

It was made of HDPE geomembranes, thermowelded along the edges.

6. GEOMEMBRANES USE

The choice of HDPE geomembranes was made in view of their high resistance to the action of chemical-physical and environmental factors, after an accurate comparison of the various synthetic products characteristics.
The testing results showed also the higher abrasion resistance of HDPE among the various kinds of lining materials, according to Cazzuffi, Puccio, Venesia (3).
The thickness of HDPE sheets was established at 2.5 mm, on the basis of technical requirements for this kind of waste basins, in order to obtain a good in situ behaviour.
The geomembranes used in the two basins were supplied and placed by two different industrial firms, using different laying and welding systems.

6.1 First basin laying and welding system

The material was supplied by Schlegel and placed by an associate German Company (GFA). The cross-section of the first basin and the geomembrane position are shown in Fig. 3.
The sheets were made in rolls 10 m high and 150 m long: they were precut according to a particular placing plan and laid first on the slopes, then on the bottom surface. Sheets were overlapped for about 0.20 m; surfaces were roughed, using abrasive grinding wheels, then fastened down with Laster, so as to prevent relative movements of the two sheets.
These operations required a certain lenght of time, but allowed the two sheets to have the same elongations.
Afterwards, welding by extrusion of a 40 mm colourless, pure HDPE, was made. Sheets were heated by means of two halogen lamps placed in front of the extruder (Fig. 6).
Welding visual control was easy: in fact, impurities, steam bubbles or not properly welded spots were clearly detectable for material transparency. Welds were also submitted to ultrasonic inspection. A welding sample was taken every 500 m and subjected to tensile test on the site.

6.2 Second basin laying and welding system

Geomembranes were supplied by SARNA ITALIA, made under licence of the homonymous Swiss Company. The cross-section of the second basin and the position of the geomembrane are shown in Fig. 4.
The sheets 2.5 m high were supplied with a maximum length of 185 m. Geomembranes were placed from edge to edge following the basin shorter side. There was some difficulty in laying the sheets on the top edges.
The welds were executed by a radiant-wedge system (Fig.7) performing a double parallel weld, leaving inside a groove for pneumatic control: the groove was subjected to a pressure of 150 kPa, which had to remain unchanged for about 10 minutes. In this basin too, in situ tensile tests on the welds were performed every 500 m.

6.3 Comparison between the two laying and welding systems used

Sheets height and welding methods were the principal differences between the geomembranes used in the two basins.

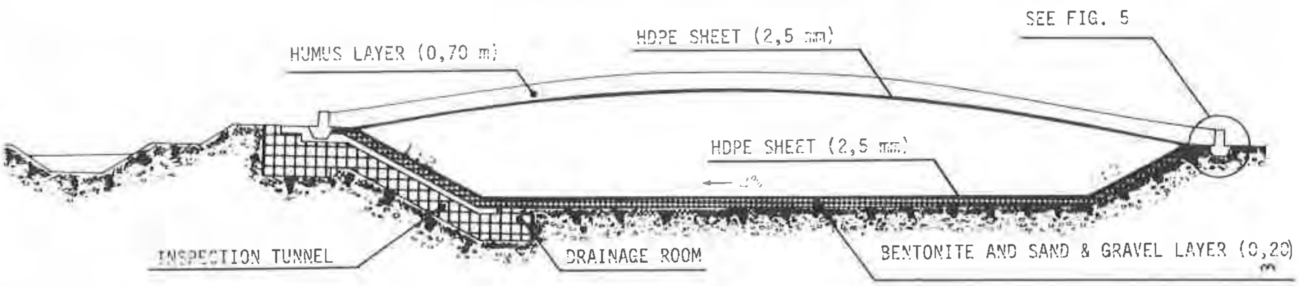

Fig. 3 - Longitudinal cross-section of the first basin

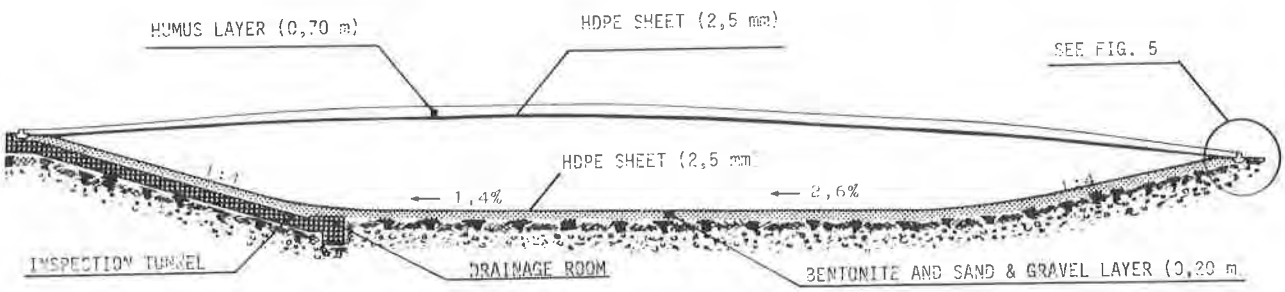

Fig. 4 - Longitudinal cross-section of the second basin

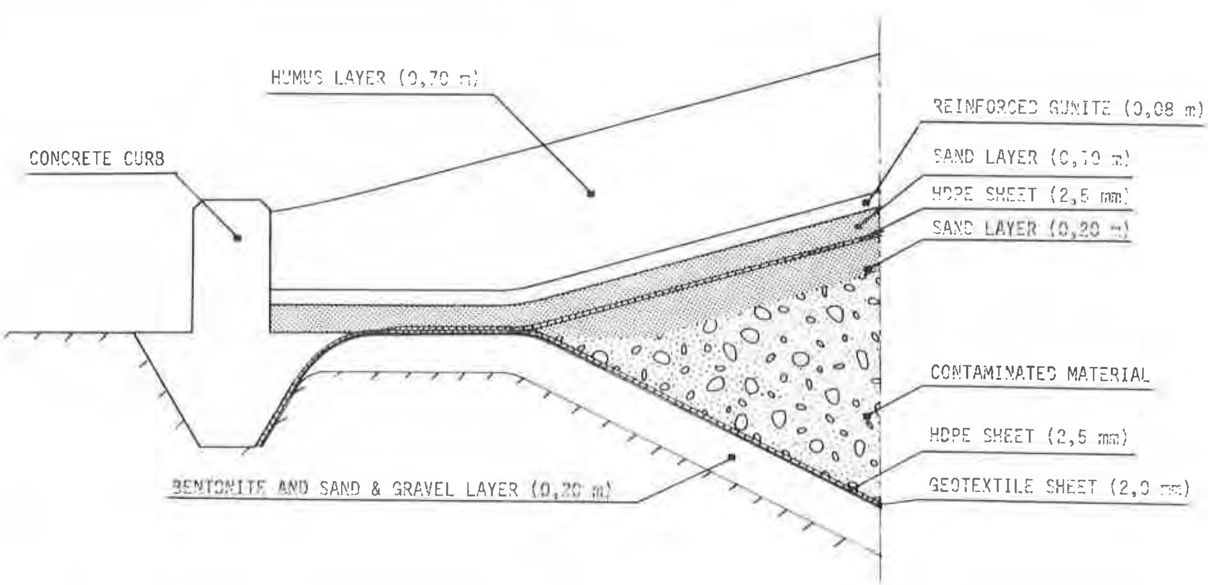

Fig. 5 - Detail of the HDPE sheets anchorage to the slope edge

The sheet 10 m high (first basin) gave the best suggestions for the following reasons:
- higher placing speed;
- shorter welding length;
- less technical assistance.

Moreover the welding system of the first basin proved to be the best, since possible defects were easily found during the control.
On the contrary, the second basin sheets required a greater assistance during their laying, an extremely careful cleaning of the surfaces to be welded and some difficulties in detecting defective welding spots.

6.4 Laboratory tests

On the arrival at the yard, the geomembranes was checked to ascertain their conformity to the established requirements. A sample was taken every 3000 m^2 and submitted to the following physical and mechanical laboratory tests:
- thickness;
- mass per unit volume;
- tensile test.

The tests were carried out in Milano, at the Plastic Materials Laboratory of the Technical University (Politecnico) and at the Special Materials Laboratory of the Research Center for Hydraulics and Structures (ENEL).
Laboratory tests results both for geomembranes and geomembranes welds are listed in the Tables 1 and 2.
The tests gave similar good results for the two different HDPE geomembranes.

PROPERTIES		TEST METHODS	SCHLEGEL (1)	SARNA (2)
Thickness	(mm)	ASTM D 374	2.62	2.55
Mass per unit volume	(kg/m^3)	ASTM D 792	943.7	946.7
Stress at yield	(MPa)	ASTM D 638	19.1	18.5
Stress at failure	(MPa)	ASTM D 638	34.0	34.6
Strain at failure	(%)	ASTM D 638	935	980
Tangent modulus	(MPa)	DIN 53457	1027	-
Secant modulus	(MPa)	ASTM D 882	-	725

Tab. 1 - Physical and mechanical properties of the two geomembranes used at Seveso.

(1) in the first basin, from the tests carried out at the Plastic Materials Laboratory of the Technical University in Milano (average value from 80 specimens)

(2) in the second basin, from the tests carried out at the Special Materials Laboratory of the Research Center for Hydraulics and Structures (ENEL) in Milano (average value from 80 specimens)

PROPERTIES		TEST METHODS	SCHLEGEL (1)	SARNA (2)
Stress at yield	(MPa)	ASTM D 638	21.2	19.2
Stress at failure	(MPa)	ASTM D 638	16.9	14.6

Tab. 2 - Mechanical properties of the two different welds systems from the tests carried out at ENEL-CRIS

(1) average value from 4 specimens
(2) average value from 24 specimens

7. BASINS CONSTRUCTION DETAILS

The anchorage of the HDPE sheets to the slope edge is shown in Fig. 5.
The other basins construction details were as follows.

7.1 Drainage system and inspection tunnel

Basins slopes converge below the lining layers to a point, in which a reinforced concrete room has been built, in order to collect waters inside the waste basin.
Apart, another pipeline collects-for control-waters between HDPE sheet and a bentonite mixture. The drainage room is accessible from the embankment through an inspection tunnel at the beginning of which a forewell made it possible to instal treatment plants and to place a dressing-room for technicians employed on control. Plants inside the tunnel are explosion-proof for the presence of biogenetic gas. Before placing contaminated material in the basins, a sand layer was spread out on the bottom for both draining and protective purposes of geomembranes.

7.2 Filling methods

Access ramp to the first basin was built up with some difficulty using contaminated material. For the second basin the ramp was foreseen during construction, so that filling works were made easier.
An adequate protection for HDPE sheets was arranged along the down ramp, so that damages caused by heavy vehicles transit were avoided. The contaminated material was placed on the basin bottom and separated from the draining layer by a geotextile sheet.
The material placed in a layer, 0.80-1.00 m thick, was carefully rolled before proceeding to the next.
Vertical draining walls were built inside the contaminated material: the walls consisted of coarse materials remaining after buildings demolition of the A zone.

7.3 Basins covering

When the filling operation was at the end, the material was levelled, than a geotextile sheet was stretched out and fastened down using a bitumen emulsion. The HDPE geomembrane was placed and welded to the one of the bottom. Then a 0.20-0.30 m thick layer of mixed quarry material was layed, on which a reinforced-concrete covering casting with adequate joints was placed, so that differential settlements were possible.
On the top a layer of cultivable soil, 0.70 m thick was placed, with the aim of hill grass regeneration (Fig.3,4,5).

8. BASINS AND GEOMEMBRANES CONTROLS

For these basins different periodic controls were planned, among which the geomembranes control is very important (4). The principal controls were:
- topographic control: geodetic survey stations were positioned every 40 m along the sheets anchorage. During the filling operations every 50,000 m^3 a control was made to evaluate structure settlements;
- HDPE sheet permeability control: the sand layer placed between the first lining layer and the HDPE geomembrane were connected through pipes with the drainage room.

Therefore it is possible to collect waters coming from these hollow spaces and ascertain whether they are caused by HDPE sheet tearings;

- groundwater control: using the piezometers, installed during geologic investigations, it is possible to check constantly the groundwater level.
- geoelectric control: the usefulness of this kind of control in detecting sheet tearings is now under study, according to Peters and Al. (5). In fact, computer simulations gave excellent results and model tests are now in progress to ascertain its real feasibility.

The views of the two basins before the filling with contamined materials are shown in Fig. 8 and 9.

Fig. 6 - The geomembrane welding in the first basin (by extrusion system)

Fig. 8 - A general view of the first basin

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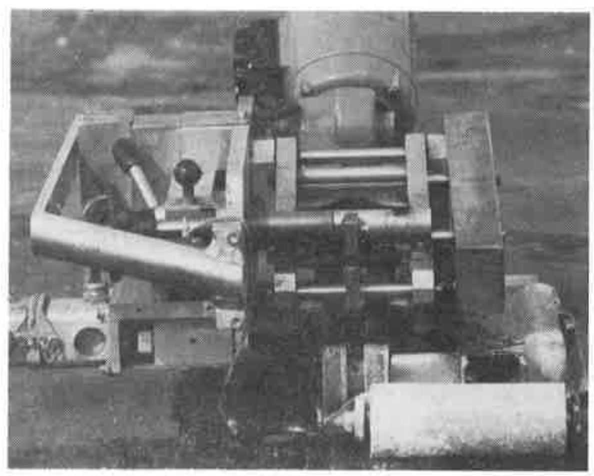

Fig. 7 - The geomembrane welding in the second basin (by radiant-wedge system)

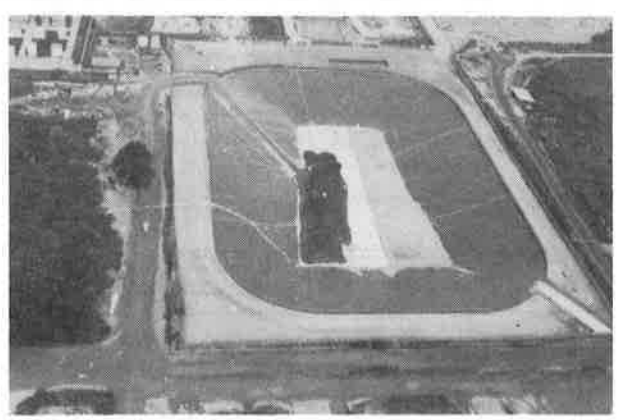

Fig. 9 - A general view of the second basin

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Use of HDPE Liners in Italy for Compliance with Solid Waste Landfill Regulations

The use of an HDPE geomembrane to contain dioxin contaminated earth at Seveso marked the first major application of the material in Italy. Since then, newly enacted regional and national laws governing domestic landfills have called for geomembranes a minimum thickness of 2.0mm. An independent study, conducted at the time of this legislative activity, confirmed the retention of tensile properties by HDPE in this thickness after exposure to sanitary landfill leachate. In three years, the use of HDPE geomembranes in Italy to line sanitary landfills has grown to more than a million square meters installed.

The Seveso Case

A safety valve that exploded during a chemical reaction at Seveso in 1976 resulted in the spilling of dioxin and extensive contamination of the area. This incident, widely reported by the international press, provoked a great public outcry, a number of scientific studies and clean-up proposals, and of course, political reactions.

At about the same time, it was discovered that urban solid wastes being burned in incinerators were giving off fumes containing trace amounts of dioxin.

The events led to a moratorium on the dumping of both industrial and domestic refuse, pending regional and national regulations. Even though Italy is, like many industrial nations, sometimes slow to anticipate dangerous situations, issuing appropriate acts and regulations only after a grave incident has occurred, these two events sensitized the authorities to the unresolved health and safety risks posed by increasing quantities of domestic and industrial wastes.

The immediate problem of whether to destroy or confine the contaminated earth at Seveso was studied at length by the scientific community, under the direction of an organization known as The Special Office for Seveso of the Regione Lombardia. Proposals for decontamination of the area ranged from incineration, to biological transformation with micro-organisms, to simple solar photodegradation. It was finally decided that confinement of the contaminated soil in a landfill lined with a geomembrane was the most feasible solution. High density polyethylene in a 2.5mm thickness was selected by the authorities because, in their opinion, it offered the greatest safety for the longest period of time. 20,000 m^2 were installed, and the landfill completed in 1982. This installation was the first major application of an HDPE lining system in Italy.

Comparative Research of Materials

Meanwhile, in 1980, a research program into the behavior of various polymeric liners was started under the direction of Professor Adriano Vanni, of Turin University's "Istituto Analisi Chimica Applicata", and in cooperation with the Turin Town Solid Wastes Department (A.M.R.R.). This research was funded by C.N.R. (Centro Nazionale Ricerche).

The objectives and methodologies of the research program were:

1. To define the life of the materials to be tested by laying them in a pilot-scale landfill sited near the main landfill "Basse di Stura" in Turin and then covering the material samples with urban solid wastes;
2. To compare these results with those obtained in a laboratory test in which samples were immersed in equivalent landfill leachate at various temperatures and checked for retention of material properties at regular intervals; and
3. To compare the resistance of various materials and their joining systems against bio-chemical attack, in order to forecast their service lives and the validity of the materials' warranties.

A discussion of the full work is beyond the scope of this paper, which will focus only on the laboratory immersion tests.

The materials that were tested were PVC (polyvinyl chloride), HDPE (high density polyethylene), EPDM (ethylene propylene diene monomer), CPE (chlorinated polyethylene) and BR (butyl rubber). Sample sizes were 21 x 29.7 cm, per DIN A4. Three sets of samples for each material were immersed in the leachate at temperatures of 21°C, 37°C, and 65°C. One set of samples for each material was retained for control purposes. The total time of immersion was 180 days, with samples taken from the leachate every 30 days and evaluated. The values for each material at the 37°C temperature and averaged from the samples are shown below in Table I.

It is important to note that the thickness of all materials tested was 1.2mm, with the exception of HDPE which was 2.5mm. These thicknesses were selected because they are the thickest gauges available on the market for the respective materials.

TABLE I

Results of Immersion Testing in Landfill Leachate

Swelling at 37° C (%) (absorption H_2O)	Control (averaged)	After Immersion (averaged)
PVC		2.9
HDPE		0.077
EPDM		3.73
CPE		4.83
BR		0.5
Stress at Break (kg/cm^2)		
PVC	175	173
HDPE	351	358
EPDM	120	119
BR	123	118
Elongation at Break (%)		
PVC	317	302-331
HDPE	880	888
EPDM	500	520
CPE	350	468
BR	520	507
Tear Resistance (kg/mm)		
PVC	7.7	7.06
HDPE	17	16.8
EPDM	4.2	3.5
CPE	—	2.7
BR	4.4	3.9
Residual Strain after Tensile Stress (%)		
PVC	11.5	9.7-17.3
HDPE	84.7	87.9
EPDM	3.2	3.8
BR	1.5	4.05
Puncture Resistance (kg/mm^2)		
PVC	40-51	39-46
HDPE	55.7	50.8
EPDM	48	47.5
BR	22.4	20.4

Although the period of immersion testing was short, the results generally confirm results obtained from the landfill trial exposures. EPDM, PVC, AND CPE showed the most swelling and were the materials most effected by temperature; HDPE was least affected. The variations in physical properties generally correlate with the amount of swelling; those materials showing minimum swelling also showed minimum variations in retention of physical properties. These authors note also that HDPE, after immersion in landfill leachate, exhibited the highest values for mechanical properties and a high rigidity in the trials for residual strain after stress.

Legislation - Regional

By the late 1970's four out of the 20 Italian Regiones had issued laws regulating the disposal of solid wastes. An out-of-date State Law from 1941 was still in force for the country as a whole, but it was inadequate in coping with the wastes of a country that has been completely transformed in 40 years, from an agricultural to an industrialized economy.

Regione Lombardia, the site of the Seveso incident, is the most densely populated area in Italy, with nine million people out of Italy's population of 56 million. It is also the most heavily industrialized region of the country. The disaster at Seveso illuminated the danger to which such a dense population was exposed. Accordingly, on June 7, 1980 Regione Lombardia issued Regional Law n. 94, followed by two sets of technical regulations, one for urban solid wastes and the other for toxic hazardous wastes. A plan for 16 new landfills in the Regione's territory was also distributed.

Section III of the law deals with second category landfills for domestic solid wastes or assimilated wastes. These detailed regulations set out specific requirements in the areas of: the necessary documentation to be submitted for authorization; features of second category landfills; the collection of surface water runoff; drainage and purification of leachate; recovery of biogas; lay-out of refuse; covering of refuse; and the landfilling of special refuse.

Article 16, which details the features of second category landfills, implies that a synthetic liner must be used, because it sets out a formula for permeability that a natural material may not meet. The Article requires no leakage of leachate for at least 100 years, which is calculated from the thickness of the liner divided by the permeability of the material. If the thickness and permeability characteristics of a natural material, such as clay, cannot meet the 100 year standard, a synthetic liner must be used.

In order to avoid any misinterpretation of the 100 year requirement, Regione Lombardia later clarified its regulation by requiring that all new landfills must have a synthetic liner in a minimum thickness of 2.0mm (80 mil). The reasoning behind this directive was a recognition at the regulatory decision making level that soil is never, in the real world, quite so impermeable as is specified, and that the conditions at a landfill construction site, including wind, mud, fluctuating temperatures and earth moving equipment, might well puncture or tear a thinner liner, even though it will have tested well in the laboratory.

Legislation - National

In September 1982 the State Government issued Act D.P.R. 915 that governs the disposal of solid wastes and fulfills the directives of the European Economic Community (EEC). This piece of legislation requires that all Italian Regiones adopt a plan for the disposal of solid refuse by mid-1985; and that all Towns, whether single or associated consortiums, use such disposal facilities by the end of 1986. Site planning is to be conducted by the Regiones in consultation with the Towns. Additionally, the State Government pledges to promulgate technical directives for the construction and management of such disposal facilities.

Liner Thickness

As this is being written (December 1983), draft technical specifications have been issued by the State Committee responsible for the promulgation of technical standards for domestic landfills, as described above as part of D.P.R. 915. These draft standards require that all controlled landfills must be sealed in some fashion in order to prevent groundwater pollution; and that natural and synthetic liners must meet specific requirements.

Liners of naturally occurring materials must have a permeability of less than 10^{-6}cm/sec and must be at least three meters thick.

Synthetic liners are required to have a minimum thickness and permeability so as to prevent any leakage for at least 50 years. This requirement for synthetic liners is similar to that of Regional Law n.94 for Regione Lombardia, except that the Regione is more strict with its requirement for a 100 year period of no leakage. The technical directive also mandates that a synthetic liner be laid over a cushion of one meter of soil with a permeability of 10^{-6}cm/sec; this measure is to provide an extra margin of safety.

Technical Standards Setting Organizations

Two organizations in Italy have issued specifications for high density polyethylene liners (as well as liners of other materials). U.N.I., the organization for standardization, specifies a minimum thickness of 2.0mm for HDPE. This is in line with Regione Lombardia's requirement.

Assogomma is an association of Italian polymeric liner manufacturers, similar in scope to the United States' National Sanitation Foundation (NSF). Technical specifications issued by Assogomma are used by landfill designers. Assogomma has prepared and published a document entitled "Technical Specifications for the Supply and Installation of a Synthetic Liner in High Density Polyethylene (HDPE) For Lining Controlled Domestic and Special Wastes Landfills." (There are similar publications for butyl rubber and EPDM). These specifications for HDPE directly address the thickness question by calling for a minimum thickness of 2.0mm when the liner is used in a landfill containing domestic or industrial refuse. Other properties required of an HDPE liner are detailed below in Table II.

TABLE II

1. COMPOSITION

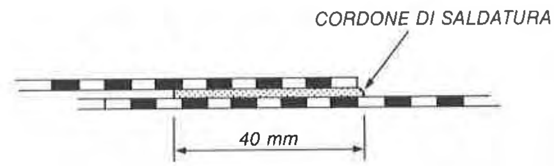
–High density polyethylene (HDPE):	•basic polymer •carbon black	≥97% (virgin) ab. 2%

2. PHYSICAL PROPERTIES

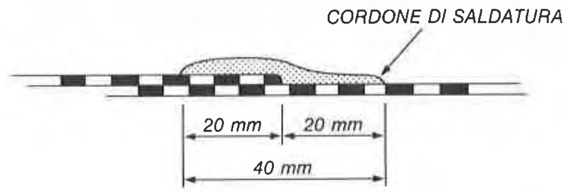
	VALUE	STANDARD
1. Specific weight	0,935 – 0,954 g/cm³	DIN 53479, ISO R1183
2. Coefficient of thermal expansion	$1{,}6-2{,}0\times10^{-4}/°C^{-1}$	ASTM D 696
3. Shore D hardness	56 – 65	DIN 53505
4. Impact resistance notched	No break	DIN 53453, ISO R179
5. Tensile stress at yield	>17 N/mm²	
6. Tensile stress at break	>24 N/mm²	DIN 53455
7. Percentage elongation at yield	>9%	v=50 mm/min ISO R527 Speed C
8. Percentage elongation at break	>600	
9. Tensile modulus of elasticity	>750 N/mm²	DIN 53457, ISO R527 speed = 1 mm/min.
10. Tearing	>145 N/mm	DIN 53515
11. UV resistance	$\Delta\sigma s<\pm10\%$ (10.000h) (50.000h)	DIN 53387 DIN 53460
12. Fragility at temperature	>–70°	DIN 53387
13. Cold bending without crack	>–20°	DIN 53361
14. Resistance to puncture	>1350 mm	SIA 280/14
15. Stress cracking	>2000h	ASTM 1693/60T

Assogomma further recognizes only two systems for seam joining for high density polyethylene liners: extrusion welding or double seam welding. It defines extrusion welding as "injecting the same (HDPE) polymer between overlapped sheets that are preheated with hot air." Extruded seams must be ≥ 40mm wide. (Figure 1)

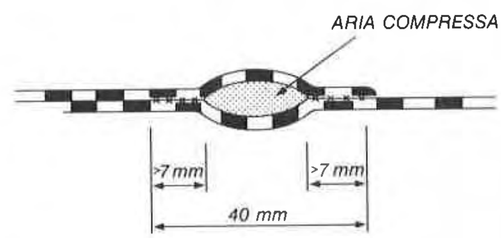
Brief lengths of seaming, when making repairs or when lining walls and steep slopes, may employ a variation of the extrusion welding process, as illustrated in Figure 2. In this case, the extrudate covers overlapped liner panels that are temporarily fixed with hot air treatment. Double seam welding creates two seam areas for the overlapped panels, leaving an intermediate canal that is used for air pressure testing. (Figure 3) The total area of overlap, and thus seam width, must be ≥ 40mm; the width of each seam path must be ≥ 7mm.



Overlap Extrusion Welding of HDPE
Figure 1



Extrusion Surface Welding of HDPE
Figure 2



Double Seam Welding of HDPE
Figure 3

Seams joined by extrusion welding or double seam welding are required to be tested with ultrasonic test methods or by air pressure tests, as well as by sight.

An acceptance certificate for a landfill lined with HDPE is issued by the Direzione Lavori, who is also present during testing and any subsequent repair work. Repairs are registered on the "as built" drawing.

The Economics of Using HDPE in Domestic Landfills

Typical costs for a medium-sized domestic landfill serving 200,000 inhabitants are listed below.

It is interesting to note that the cost for the installed HDPE liner in a 2.0mm thickness is less than 30 percent of the total investment, and that a reduction of liner thickness to 1.0mm (40 mil) cuts the liner cost by less than five percent. It is the opinion of these authors, which seems to be confirmed in the legislation and by standards setting bodies, that a reduction in liner thickness, whether for economic reasons or other considerations, is not worth the risk to human health and safety.

For this example, assume an average per capita production of waste of 0.9 kg/day; a projected service life for the landfill of six years; and a capacity require-

requirement of 500,000m^3 from the following:

200,000 population x 0.9 kg x 365 days x 6 years = 400,000 tons

400,000 tons - 0.8 (specific gravity) = 500,000m^3

Assume too that the landfill is an exhausted quarry, as such sites are typically converted into landfills in Italy, and that 150,000m^2 of material must be further excavated in order to obtain a bottom grade of no more than 0.5 $\pm$ 1% and embankments with a slope of 1.5:1.

The figures in $US have been converted from Italian Lire, based on 1983 market conditions. The qualification must be made that the costs listed represent typical costs for these items in Italy, and may or may not be representative for the same materials and services if purchased elsewhere.

1.	Cost of the land (exhausted quarry with surface area of approximately 80,000m^2)	$US 50,000
2.	Excavation (The excavated earth will be heaped beside the landfill and used later to cover the layers of refuse, at 20cm of inerts for every 2m of waste until the landfill is closed.) Subgrade must be compact and free of any protrusions; it may require a sand layer of 5cm.	187,500
3.	HDPE liner, 2.0mm thick - installed	437,500
4.	Protective layer of 30cm over the liner on the floor of the landfill, and discarded tires on the embankments	37,500
5.	Seepage drain of HDPE pipe laid over the liner	6,250
6.	Catch basin lined with HDPE	18,750
7.	Biogas uptake with 15 vertical steel chimneys and horizontal collection pipes and biogas burner	31,250
8.	Groundwater monitoring wells	25,000
9.	HDPE basins, pump, compressor, piping, for leachate aeration and recycling	31,250
10.	Boundary fencing and gate	18,750
11.	Office building, garage and motor vehicles	31,250
12.	Scales	31,250
13.	Service connections for electricity, telephone, water, etc.	31,250
14.	Landfill boundary vegetation	125,000
15.	Waste compaction of 20 t, unloading and loading shovel and truck to transport earth	312,500
16.	Design of the landfill, testing, instrumentation, construction superintendent	187,500
		$US $1,562,500

Landfills in Italy Using HDPE Liners

In only three years, since enactment of the model landfill legislation by Regione Lombardia, nearly all new landfills in Italy have used the 2.0mm HDPE liner. More than a million square meters are now in service. The major landfills where HDPE is now in use include:

MONTEDISON-Mantova	15.000m^2	industrial wastes
ECODECO-Corteolona	15.000m^2	industrial wastes
OSIO SOTTO (Bg)	35.000m^2	domestic solid wastes
SEVESO (Mi)	20.000m^2	industrial wastes
MOZZATE (Co)	100.000m^2	domestic solid wastes
CODIGORO (Fe)	15.000m^2	domestic solid wastes
MARIANO COMENSE (Co)	50.000m^2	domestic solid wastes
ZANICA (Bg)	50.000m^2	domestic solid wastes
CASATISMA (PV)	35.000m^2	domestic solid wastes
TAVAZZANO (Mi)	10.000m^2	industrial wastes
MALEO (Mi)	55.000m^2	domestic solid wastes
GERENZANO (Mi)	65.000m^2	domestic solid wastes
TORINO	65.000m^2	domestic solid wastes
BASSANO (Vi)	50.000m^2	domestic solid and industrial wastes
CHIURO (So)	50.000m^2	domestic solid wastes
AGIZGNANO (Vi)	35.000m^2	industrial wastes

Another 130,000m^2 have been installed in smaller landfills not listed above.

FIGURE 4 - The Osio Sotto landfill was lined with $35,000m^2$ of HDPE geomembrane.

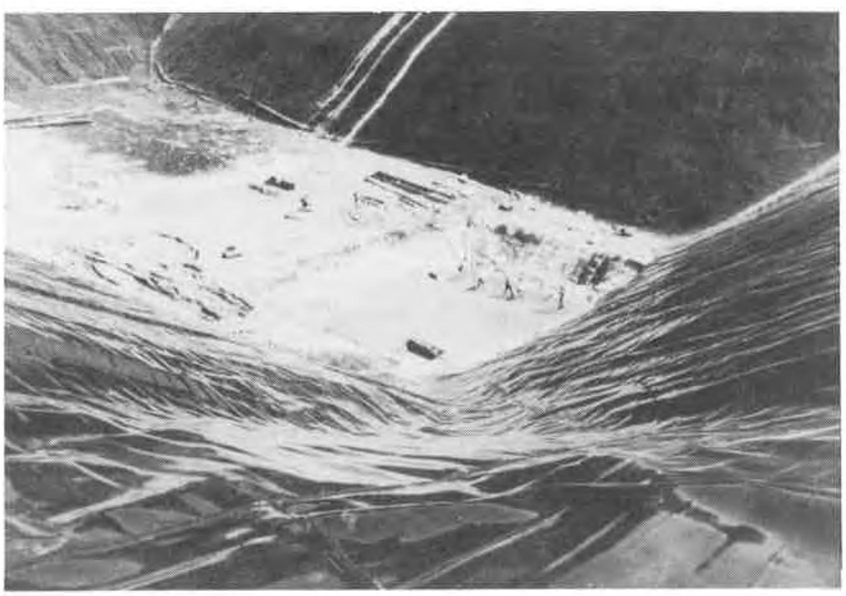


FIGURE 5 - The Mozzate landfill.

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Design and Construction of a Membrane Lined Flue Gas Desulfurization Pond

This paper describes the background, the design considerations, and the monitoring of the lining installation during the construction of the Naughton Flue Gas Desulfurization (FGD) Pond.

The Naughton Unit 3, FGD pond was designed to contain 414,000m^3 of waste liquor consisting of a concentrated solution of sodium sulfate, sodium sulfite, sodium bi sulfite and fly ash. These waste products are generated from the flue gas scrubber and are pumped through one of the two underground high density polyethylene pipes.

The pond was constructed by stripping an area to an acceptable elevation and compacting the subsoil to receive a 30-mil PVC liner. The liner area was 93,000m^2; also, the liner extended over a 518m long embankment and was covered by a 0.3m earth cover.

Seepage collection systems were designed and installed under the liner and in the embankment. The collected seepage drains into two secondary 2,025m^2 ponds with facilities to pump back the collected liquid. In addition, groundwater sampling wells were installed around the pond to monitor any leakage from the lined pond.

The pond has been in operation for three years and is performing in a satisfactory manner.

INTRODUCTION.

The Naughton Power Plant is located approximately 16km southwest of Kemmerer, Wyoming. It consists of a 3-unit coal fired station generating about 710 Mw. of electrical power. The plant obtains its water for operation from Hamsfork Reservoir. Fresh water is used initially for cooling tower makeup and the waste water is recirculated several times in the plant. In its last stage, some of the waste water is used in the flue gas desulfurization (FGD) scrubber in Unit 3. The scrubber's desulfurization process results in production of FGD waste liquor containing sodium sulfate, sodium sulfite, sodium bi sulfite and fly ash. The FGD waste is stored in an evaporation pond. This paper describes the design considerations used and the precautions taken during the construction of the project. The cooperation between the owners, state regulating agencies, the contractor, design engineers and their consultants has brought about an efficient, economical and a practical method for storage of FGD waste.

PROJECT DESCRIPTION.

The unit-3 FGD waste liquor evaporation pond has 414,000m^3 capacity. A 93,000m^2 area located between two natural ridges (Fig. 1) is lined with a membrane liner. A U-shaped embankment was designed and constructed to develop the required storage capacity. (Fig. 1.)

The waste liquor generated by the scrubber is transported into the pond through one of the two High Density Polyethylene pipes. The design discharge rate is approximately 3.1 L/s. The low rainfall in the area and a high evaporation rate provide ideal conditions for the use of an evaporation pond. The mining activities on the upstream side of the pond have reduced the expected runoff by reducing the rainfall catchment area. Even the minimal runoff resulting from the drainage area has been diverted into the neighboring valley. As a result of the controlled inflow into the pond and diversion of the storm water, spillway was considered unnecessary.

The project consisted of designing and constructing the embankment, diversion dikes and providing an acceptable liner system to contain the FGD waste.

DESIGN CONSIDERATIONS.

The pond design was carried out under three independent considerations:

A. Providing pond volume by taking into account the inflow from the plant, direct rainfall, runoff from the limited drainage area, evaporation, wave heights and wave run up during high wind. By these computations the crest elevation of the dike and the elevation of the maximum water surface were established;

B. A stable embankment with internal drains was designed by using compacted borrow materials obtained primarily from the pond area. A local borrow provided materials for processing the drainage and filter materials for use in the embankment; and

C. Designing and installing a liner system to minimize the seepage of FGD waste into the groundwater and to monitor the seepage losses, should seepage occur.

POND VOLUME.

The volume requirement for storage of the waste liquor was established in consultation with the owners based on the estimated flow rate and expected pond life. The evaporation rate was estimated from the available records. The runoff rate due to precipitation was based on a minimum of two consecutive 100-year storms. Diversion works were designed for diverting storm water generated by 0.5 times the probable maximum precipitation plus the 100-year storm. Wave heights and wave run up were also considered. By these considerations the crest elevation of the embankment was set at 2,115.9m (6,940 ft.) and the freeboard was set as 1.5m.

EMBANKMENT.

The embankment was designed as a zoned compacted earthfill provided with internal drains of sand and gravel. The internal drains consist of chimney drain and a horizontal drainage blanket. The longitudinal slope in the drainage blanket facilitated the collection of possible seepage at the low point in the valley. Provisions were made to lead the water into a smaller lined 2,025m^2 pond. (Fig. 2.)

Stability analyses were carried out to verify the adequacy of the safety factors for both the upstream and the downstream slopes under various seepage and earthquake loading conditions. The upstream slope was made flatter than required in order to facilitate the installation of liner. (Fig. 3.)

THE LINER.

A) Liner Selection.

The most difficult part of the design of liner was the selection of the type of liner. With many types of liners that are commercially available at competitive costs, it was quite confusing for the engineers to select a cost effective liner. A consultant's help was sought by the design engineers. Four basic types of liners were considered. These consisted of a) clay liner; b) asphalt liner; c) membrane liners such as reinforced Hypalon and High Density Polyethylene; and d) thin membranes such as PVC. After studying these alternatives, thin membrane was selected for the FGD Pond. Clay liner was considered unsuitable because of the slow rate of water build up and the dry months at Naughton which tend to develop cracks in clay liner which may be filled with blown material. Asphalt was in short supply because of the progress of work on a road-way project near Kemmerer. The presence of animals and the high winds in the area precluded the use of surface membranes. Therefore, the placement of thin buried membrane was considered. The following ground rules were used to select the type of liner:

1. Any liner will be covered by 1-foot thick soil cover.

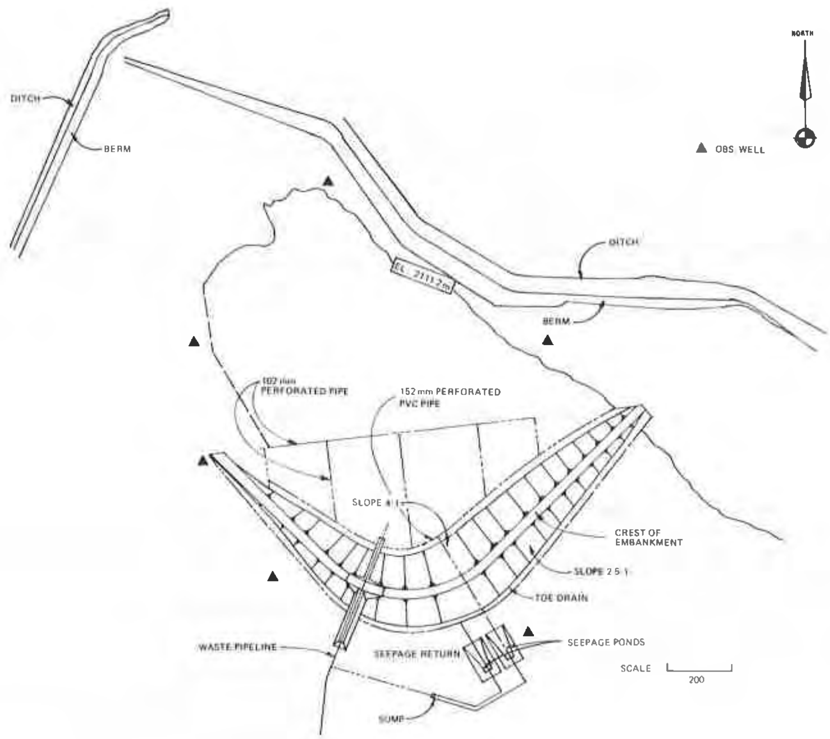

FIGURE 2: PLAN OF FGD POND
AND UNDER DRAIN SYSTEM

2. The chemicals in the liquor should not react with the liner material.
3. The chemicals should not react with the soil. If there is a chemical reaction and if gases are generated the membrane would tend to lift, if leakage occurs. Therefore, the adequacy of membrane selected depended on the soil chemistry.
4. The membrane should be puncture resistant during placement of soil cover.
5. The membrane should have adequate resistance to sustain the soil load and the waste liquor load and must be able to stretch because of the uneven settlement of soil underneath the liner.
6. Constructing field seams using local labor must be simple.
7. An underdrain system should be provided to monitor leakage of the waste liquor. Should such a leakage occur, some provision should be made to collect and pump back the collected fluid into the pond. The design of an acceptable underdrain system should be a part of the liner design.
8. Groundwater sampling wells should be installed to monitor the quality of groundwater. These wells should be monitored periodically to detect any leakage of the waste into the groundwater.

B) Liner Design.

Considering the chemical constituents of the FGD waste, it was concluded that the chemical reaction between the site soils and the FGD waste will not be significant and the thin membrane liner could be selected. Further study of the gradations of the site soils showed that the material is fine grained and hence it was possible to prepare a smooth surface for receiving the membrane and to use the on-site soil to cover the membrane. With a proper design of underdrain system and installation of observation wells, all the ground rules for the selection of thin liner could be met. PVC liner was selected although several other thin liners would be equally acceptable. The selected liner was first tested for puncture resistance by clearing a 6m by 6m area over which the liner was placed and the soil cover was placed over the membrane. Heavy construction equipment was moved over the soil. After several passes the membrane was removed and examined and no punctures were observed. With this test the selected membrane thickness of 30 mil was considered adequate.

An underdrain system was designed and set up in a grid pattern as shown in Figure 2. The system installation was restricted to areas where the depth of the liquor is maximum. The perforated underdrains were encased in sand and gravel. The main drain line was carried through the foundation of the embankment in a concrete encased pipe into one of the two 2,025m^2 lined ponds. (See Fig. 2.)

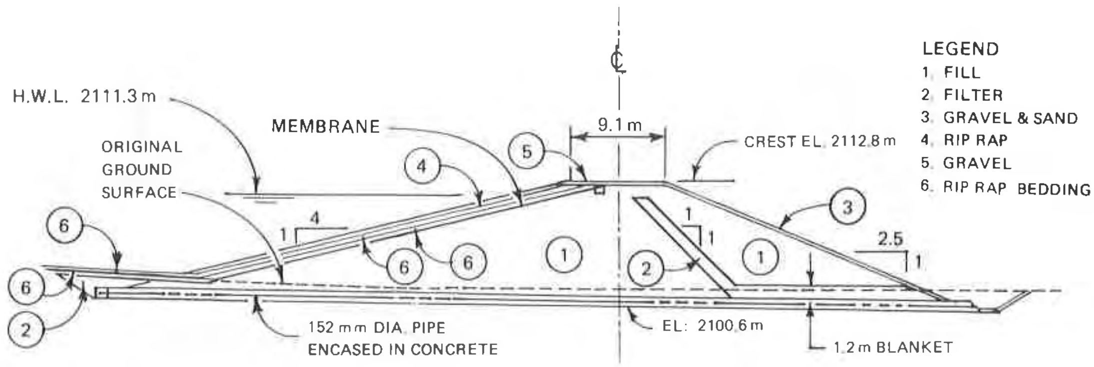

FIGURE 3: EMBANKMENT SECTION

Anchoring of the membrane was accomplished by excavating trenches in the rim of the pond at least 1m above the high water level. The membrane was extended through the trench and the trench was backfilled with soil. On the embankment slope the membrane was placed 1m below the surface of the slope. This soil cover provided sufficient weight to minimize the potential of soil slippage during the wave action. Riprap was provided as a part of the embankment design.

LINER INSTALLATION.

In pond area liner installation was preceded by stringent inspection of the proposed prepared area to determine whether the area was free of spillage of petroleum products; and free of ravelling, ruts, pockets, abrupt changes in grade, sharp objects such as rocks, and roots. The liner was spread on the prepared surface and again the liner surface was inspected visually to detect manufacturing defects and installation damage. Probes were used to inspect factory seams. Sandbags were used to retain the edges of materials in position. All factory seams and field seams were checked by an air lancing test using a compressed air jet at 550kPa, pressure.

On each day all equipment including shoes of workmen were checked for sharp objects to minimize membrane damage. Temperature conditions were monitored so that the adhesive and the sheet were in correct temperature range for forming an efficient joint seal. As an additional precaution, tarpaulins were used at work areas to minimize membrane damage potential. No vehicles were permitted on any membrane without soil cover nor any diesel or hydrocarbon of any type was permitted in the work area. Joints were lapped a minimum of 0.1m. Adhesive was applied in the seam area over incremental lengths of 0.5m. Where fish mouth type openings were detected in the seams during an air lancing test, the seam was separated and a patch was placed over the separated seam thus fully sealing the area from potential leakage. Patches were placed in any suspected area of cut, tear or abrasion punctures.

Equipment operation during backfilling was controlled. Particular care was taken during fill placement, finishing the surface of compacted soil and for any oil or gasoline leaks from the equipment. The operating equipment were not permitted to spin the wheels in place and only light equipment was used on soil cover for the entire operation.

ON THE MAIN EMBANKMENT.

The liner was extended on a 4:1 slope inside the body of the embankment. Soil cover was placed on the top of the liner. Riprap was carefully lowered into place by special equipment.

IN SECONDARY PONDS.

Two secondary ponds, each of 2,025 m^2 in area, are lined with the same type of membrane. A buried return pipe line with a pumpwell has been installed to pumpback any seepage that might collect in these ponds. Anchor reglets and rubber rods were used to anchor the membrane to the concrete structures in the secondary pond area.

PERFORMANCE OF THE LINER SYSTEM.

At present there are no indications of leakage through the membrane. Monitoring of the observation wells (Fig. 2) are being continued by the owner and the observation wells do not indicate any FGD chemicals.

The completed pond and embankment are shown in Fig. 4.

CONCLUSIONS.

It can be safely stated that with a proper study of the disposal system an economical and effective liner system has been developed. With a careful inspection program, a satisfactory liner system can be installed to satisfy design requirements. Liner installation and liner selection requires expert knowledge and thus the help of consultants is invaluable in choosing the correct liner for each specific case.

ACKNOWLEDGEMENTS.

The author wishes to thank Utah Power and Light Co., for permitting the publication of the paper. Mr. D. T. Teshner, Project Manager, and Mr. H. H. Burke, Manager of Soils Engineering at Bechtel, have reviewed and provided very valuable comments during design and construction, and in the preparation of this paper. Many engineers, specialists from government agencies and Bechtel consultants helped in the successful completion of this project. Their help in making this project a success is gratefully acknowledged.

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Case Study: Key Lake Mine

This paper describes the installation of a high density polyethylene (HDPE) liner for Key Lake Mining Corporation (KLMC) at Key Lake, Saskatchewan, 640 kilometers (400 miles) north of Saskatoon. The liner was installed in two water reservoirs and five monitoring ponds during August 1983. Key Lake Mine began full operations in October 1983 and will eventually be the largest uranium mining/milling complex in the world.

An invitation to tender for the liner was made in June 1983, when it became apparent that an already installed liner would have to be replaced before September/October 1983 in order to adhere to the startup schedule for the mine/mill.

It is hoped that this report will serve as an example of what can be accomplished quickly and well when an owner has carefully and thoroughly prepared his specifications, and when a liner manufacturer/installer responds with alacrity throughout the process of quotation and installation.

KEY LAKE MINE

The Key Lake region is located in the Athabasca sandstone region in the northern half of the Province of Saskatchewan, in western Canada. (Figure 1)

General Location of Key Lake
FIGURE 1

The project is jointly owned by the Saskatchewan Mining Development Corporation, Uranerz Exploration and Mining Limited, and Eldor Resources Limited. Key Lake Mining Corporation is wholly-owned by the joint venture partners and is the operator of the project.

There are three sources of ore at Key Lake: the Gaertner Pit, the Deilmann Pit and deposits of cobble ore. Although not large in area, they contain about 180 million pounds of U_3O_8. Ore is extracted by open pit mining methods, with separate pit development for each of the two discrete deposits. The mill is designed to process 780 tons ore per day and employs conventional separation and extraction techniques to process the high-grade ore. It is expected that overall uranium recovery will be greater than 97 percent, with an annual production capacity of 12 million pounds U_3O_8.

The two water reservoirs and five monitoring ponds which were lined with HDPE are part of the process plant complex, and are sited adjacent to the mill and tailings deposition area.

REGULATORY BACKGROUND

The regulatory scheme governing operations at Key Lake, as well as at other Saskatchewan uranium mining and milling facilities, is a complex, though somewhat informal, overlay.

Until the mid 1970's, the Atomic Energy Control Board (AECB) of Canada had conceded de facto jurisdiction over uranium mining to provincial authorities, because mining in general has traditionally been regarded as a provincial activity. By 1976, however, AECB had assumed a more direct role in the regulation of uranium mining.

AECB licenses an entire uranium mining and milling complex as a single entity, even though each component of the entity may fall under the regulatory authority of various federal and provincial departments. The mineral surface lease agreement at Key Lake, for example, is between KLMC and the Department of Northern Saskatchewan. This lease provides for, among other things, environmental protection measures that must be carried out at the facility. Approval of systems for the protection of ground and surface waters, of which the two lined reservoirs and five monitoring ponds are integral components, falls under the authority of Environment Saskatchewan. Absolute water quality standards are set by Environment Saskatchewan's Metal Mining Liquid Effluent Regulations; they are invoked at the point of discharge. Surface water quality standards are applied downstream from the point of discharge, at the monitoring station(s). For radionuclides, the Saskatchewan standard for surface waters of three pCi/L is the most stringent provincial standard in Canada.

THE RESERVOIRS AND MONITORING PONDS

Key Lake Mining Corporation filed a three-volume Environmental Impact Statement with Saskatchewan Environment in October 1979. That document was reviewed and officially accepted by the Deputy Minister of the Department in 1981. Included in the EIS are provisions for lining the water reservoirs and monitoring ponds with "an impervious liner."

It should be noted here that the tailings deposition area, a common area of application for geomembranes, was not lined with a synthetic liner but with a bentonite/sand mixture. The Key Lake tailings will generally be deposited using the sub-aerial technique, which results in a laminar structure with vertical segregation of sands and slimes, and thus a very low coefficient of vertical permeability. The tailings mass has considerable structural integrity and little potential for liquifaction. Any runoff from the tailings is collected and recycled to one of the water reservoirs for treatment before discharge.

The reservoirs and monitoring ponds are sited on substantially different strata than is the tailings impoundment, and their content, of course, is entirely liquid. The reservoirs and ponds sit on surficial materials of glacial till, sand and gravel.

The purpose of the reservoirs is to store temporarily all contaminated water from the open pit area and ore stockpile areas. This function began when the pit overburden excavation reached ore-bearing boulders.

The first reservoir, with a capacity of 240,000 m^3, holds mine water and site and stockpile runoff. All runoff from the ore stockpiles, the open pits and the mill is considered contaminated, and is collected and stored in this reservoir. This reservoir also serves as a source of process water for the mill, as no free water is ordinarily introduced into the milling process, other than what is collected by dewatering. This reservoir was lined with 37,000 m^2 (400,000 ft^2) of HDPE material.

The second reservoir holds supernatant water from the tailings area, as well as any other water requiring treatment before it is recycled or discharged to the environment through the monitoring ponds. The capacity of this reservoir is 120,000 m^3; it was lined with 20,800 m^2 (224,000 ft^2) of HDPE liner.

The five monitoring ponds, with individual capacities of 5,000 m^3, hold treated supernatant water and are used to test treated water before it is discharged; unacceptable water is recycled for further treatment. Each monitoring pond was lined with 3,100 m^2 (33,600 ft^2) of HDPE.

Analyses of the contents of the ponds as printed in the invitation to tender are reproduced in Table I.

THE THIRTY-HOUR QUOTATION

When it became apparent that KLMC would have to replace the liner that had originally been installed in the reservoirs and monitoring ponds, a consultant recommended the high density polyethylene material. On June 27, 1983, executives from the company flew to Houston, Texas, invitation to tender in hand, to meet with two Texas-based manufacturers of the lining material. After an initial consultation with KLMC personnel, both firms prepared their quotations and delivered them the next day, June 28. Agreement to one of the quotations was reached on June 29, with confirmation received on July 7.

TABLE I

SPECIFICATIONS—TECHNICAL CONDITIONS

The typical analyses of these liquids will be as follows:

Reservoir #1

pH	5.5	(5-7 range)
U_3O_8	4.2	mg/L
Ni	606	mg/L
As	16	mg/L
Co	457	mg/L
Mg	251	mg/L
Ra. 226	3000	pCi/L

Plus possible traces of organic solvent s.g. < 1.0
Maximum water temperature 50° C

Reservoir #2

pH	9	(8-11 range)
U_3O_8	0.5	mg/L
As	3-4	p.p.m.
Soluble with $CaSO_4$ Heavy Metals	0-0.05	p.p.m.

Monitoring Ponds
Clear water – pH 7

While this author does not subscribe to a Federal Express style of next-day delivery of quotations as a standard business practice, the urgency in contracting for a liner in this case arose from some unusual circumstances. Facility startup having already been delayed, officials of the company were understandably concerned that it not be stalled further. The climate of northern Saskatchewan also dictated a July or August 1983 schedule for installation; freezing temperatures in early September are not uncommon, and the frost-free period lasts only until mid-September. Snow cover in the region generally lasts from October until mid-April at the earliest and sometimes early May.

If the July/August timetable for liner installation had not been met, the liner could not have been installed until the spring of 1984 at the earliest, with a consequent delay in production from October 1983 to June 1984.

The original invitation to tender had been thoroughly prepared, both in a contractual sense and in its technical detail. Only minor changes to the document were necessary to reflect the change in specifications to HDPE in thicknesses of 1.5 mm (60 mil) or 2.5 mm (100 mil).

TECHNICAL CONDITIONS: CLIMATE AND ICING

According to KLMC officials, one of the chief performance concerns, as articulated by the change in specification to HDPE, centered on the liner's ability to withstand the generally severe climate and the icing conditions that will form in the reservoirs and ponds. The invitation to tender reads:

> "The HDPE liner shall be capable of withstanding the climatic conditions as outlined.
>
> During regular operation of these facilities the liquid levels will vary continually over a range from full to empty throughout the year.
>
> Thus parts or all of the liners may be exposed to the full range of climatic conditions for extended periods of time and must be capable of withstanding the effects of sun, wind, rain, hail, snow, etc."

Thus, not only would ice form on the reservoirs and ponds, but it would continually ride up and down the slopes as liquid entered the impoundments and as treated effluent was discharged. Or, as one engineer put it, "Those ponds are an environmental stress test in all its living glory."

Two forces can be exerted by the formation of ice in a lined basin. The first is lateral pressure that is brought to bear on the side slopes of a basin when the entire surface freezes. The second occurs, usually in springtime, when large chunks of ice that break off from the main body are driven by wind and wave action into the side slopes, creating localized point source forces on the liner. Portions of the ice cap may also literally "hang" on the liner and tear it easily if enough mass has accumulated.

A basin designer can mitigate the destructive potential of ice loading by employing a relatively mild slope, by avoiding materials with a tendency to "stick" to ice, and by specifying a high strength material in an increased thickness for better puncture and tear resistance.

A mild slope, such as 3:1, will ease the lateral pressure of a solid ice cap and permit it to ride up the slope rather than impinge on the liner in a direction perpendicular to it. Adherence of ice to a lining material occurs more frequently with roughly textured liners. The slopes of the ponds at Key Lake are 2.5:1 (horizontal to vertical). The HDPE liner that was used has a low coefficient of friction.

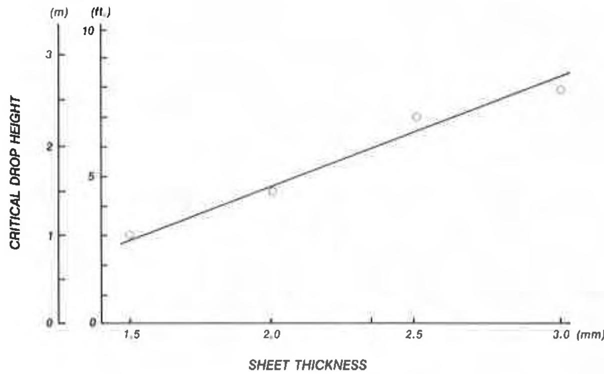
KLMC was persuaded to accept an HDPE liner in a 2.5 mm thickness, rather than the allowable alternate thickness of 1.5 mm, on the basis of data showing that any plastic material's resistance to puncture and tear is a linear function of its thickness. The incremental cost of the increased thickness was relatively low.

Schlegel Lining Technology has performed puncture and tear resistance testing on various geomembranes. The purpose of this testing was to determine the relationship between puncture resistance, tear resistance and liner thickness.

The test procedure used for the puncture resistance testing was the Swiss Standard SIA 280/14. In this test a steel bolt of specified dimensions and weight is allowed to fall onto a liner specimen. The height is varied until the critical drop height is found, i.e., the greatest height from which the steel bolt is dropped with no liner puncture for five individual trials.

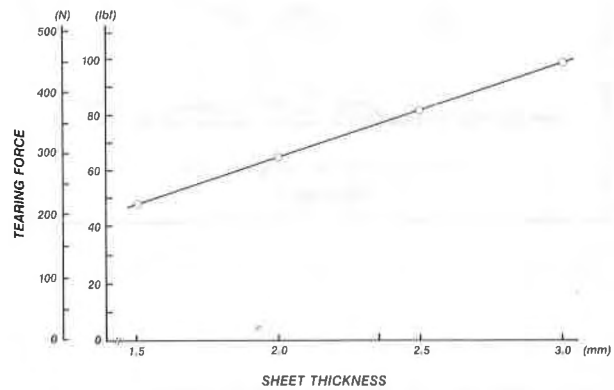
Tear resistance testing was conducted in accordance with the German Standard DIN 53 515. In this test a Graves angle test specimen is subjected to tensile stress, at a testing speed of 500 mm/minute (20 inches/minute); this relatively high speed allows simulation of rapid tearing as would likely be found in actual site conditions.

The results of puncture and tear resistance testing for HDPE are shown in Figures 2 and 3. It can be seen from these plots that a linear relation between puncture resistance and thickness, and between tear resistance and thickness, is closely approximated within the thickness range tested.



TESTING ACCORDING TO SIA 280/14

PUNCTURE RESISTANCE TESTING
CRITICAL DROP HEIGHT VS. THICKNESS
FOR HDPE (SCHLEGEL® SHEET)
FIGURE 2



TESTING ACCORDING TO DIN 53515

TEAR RESISTANCE TESTING
TEARING FORCE VS. THICKNESS FOR
HDPE (SCHLEGEL® SHEET)
FIGURE 3

INSTALLATION

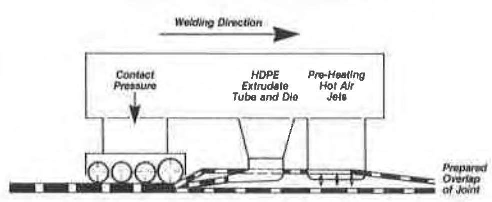
The manufacturer selected to provide the liner also performs its own installation as part of a program of single source responsibility for lining system performance. More than 200 rolls of the HDPE liner were delivered to the Key Lake Mine site in late July 1983. A single liner panel is manufactured in a 10 m (33 feet) width; the length of the 2.5 mm liner roll is 150 m.

The reservoirs and ponds were constructed in a priority order, determined by the operating needs of the facility. The larger water reservoir was installed first, followed by the second reservoir and then the five monitoring ponds.

Installation of the 73,300 m^2 of liner was accomplished in 36 work days, out of 37 elapsed days, using two shifts of 10-man crews. Crew composition varied from day to day, depending on the work to be performed and the weather. A crew generally consisted of laborers, technicians responsible for welding the liner panels together, and a site superintendent or foreman.

All of the laborers were northern Saskatchewan residents of Indian ancestry, as required by the contract. Northern Saskatchewan is an underdeveloped area and is only now making the transition from a traditional, land-based subsistence economy to one that is wage-based. KLMC has committed itself to full employment opportunities for northern residents, and this program was extended to include all subcontractors employed during the construction phase of the facility.

Liner panels were joined with an extrusion welding process that injects molten polyethylene, of the same resin from which the liner is manufactured, between pre-heated and overlapped panels. Immediate compression completes the seal. (Figure 4) The resultant bond strength must be ≥ 100% before it is considered acceptable. No welding of the liner panels was performed when it rained or under conditions where the seams to be joined could not be maintained in a thoroughly dry condition.



Principle of Extrusion Welding for Site Joining

FIGURE 4

QUALITY ASSURANCE AND TESTING OF FIELD SEAMS

The quality assurance program for the HDPE liner that was used at Key Lake begins when incoming raw material is tested, by both the supplier of the resin and in the laboratory of the liner manufacturer. The resin is released for production only after it has been tested and found to meet established criteria for density, carbon black dispersion, melt index, relative viscosity, stress-crack resistance and average molecular weight.

During the manufacturing process, samples from the production line are taken at regular intervals for laboratory testing. These samples are evaluated for their thickness, thermal shrinkage, impact strength and tensile strength. Each roll of the liner leaves the manufacturing facility with its own quality control card, which is rechecked against the roll at the installation site.

Before actual extrusion welding of the panels is carried out, test welds are made to determine seam quality. Extrusion welding begins only when these test welds exhibit at least a 100% bond strength. At the Key Lake site, all welded seams were examined with a continuous and non-destructive ultrasonic impulse echo system. Samples of actual welds were also cut, field tested for bond strength and then forwarded to the laboratory for further testing.

Laboratory reports for the tensile strength of weld samples from each of the seven basins at Key Lake are reproduced below in Tables II, III, IV, V, and VI.

TABLE II
Tensile Testing
Weld Seam Quality Control

Sample No.	Location of Sample Seam # N/N	Bond Strength 100% Minimum
Reservoir #I		
1	East Slope	105%
2	Bottom	> 144%
3	Bottom	> 147%
4	Bottom	> 137%
5	Bottom	134%
6	North Slope	129%
7	West Slope	123%
8	South Slope	127%

TABLE III
Tensile Testing
Weld Seam Quality Control

Sample No.	Location of Sample Seam # N/N	Bond Strength 100% Minimum
Reservoir #II		
1	East Slope	132%
2	North Slope	> 137%
3	South Slope	> 131%
4	West Slope	132%
5	South Bottom	130%
6	North Bottom	143%

TABLE IV
Tensile Testing
Weld Seam Control

Sample No.	Location of Sample Seam # N/N	Bond Strength 100% Minimum
Monitoring Ponds		
Pond I		
1	North Slope	> 129%
2	South Bottom	115%
3	East Slope	139%
Pond II		
1	North Bottom	> 154%
2	South Slope	> 151%
3	East Slope	143%

TABLE V
Tensile Testing
Weld Seam Quality Control

Sample No.	Location of Sample Seam # N/N	Bond Strength 100% Minimum
Pond III		
1	East Bottom	> 144%
2	West Slope	130%
3	North Slope	129%
Pond IV		
1	North Slope	> 147%
2	South Slope	138%
3	East Slope	> 136%

TABLE VI
Tensile Testing
Weld Seam Quality Control

Sample No.	Location of Sample Seam #N/N	Bond Strength 100% Minimum
Pond V		
1	North Bottom	> 142%
2	South Slope	> 153%
3	West Slope	149%

With installation completed, field seams tested and verified in the laboratory for their integrity, KLMC accepted the first reservoir on August 18, 1983 and the remaining ponds on August 31. The reservoirs and monitoring ponds were immediately put into service.

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The Use of Geomembranes for Manure Storage

Manure reservoirs are built to store temporarily the overproduction of manure which cannot be dumped on the land immediately. This dumping would cause serious environmental problems by the chemical components in the manure. Also the manure can be used at a later, more convenient time, which makes savings on artificial manure possible.

Besides a description of the use of geomembranes with constructing these reservoirs, two particular manure reservoirs, built in 1976, have been examined. Especially the performance of the geomembranes has been evaluated by comparing the specifications of field samples with the original specifications of the not used geomembranes.

INTRODUCTION

The problems with the liquid manure in Holland are mainly caused by the fact that many farmers produce, with the modern intensive stock breeding, regularly more manure at their farms than is needed in their own farm and surrounding properties.

The order of magnitude of the manure surplus is about 7 million cubic metres a year. This manure is mostly produced on large poultry- and pigfarms.

To find a proper use for the manure surplus so-called manurebanks have been raised. The farmers pay a certain fee for which their manure surplus will be collected and transported to areas where the manure is needed. However, the part of the manure surplus which is collected and transported by the manurebanks is less than 1 million cubic metres a year. In addition, at the most, 1 million cubic metres is collected by commercial manure traders. So there is a significant surplus left of about 5 million cubic metres a year which is not used elsewhere.

At the moment most of this 5 million cubic metres is dumped on the land by the farmers themselves. In several cases more than twice the amount of manure that is needed for proper manuring of the specific soil. Agricultural studies show that an over-manuring like this will make the soil worthless for agricultural use in about 20 years.

Another major result of the dumping of manure is the pollution of the groundwater. Materials like cadmium, cupper, phosphates and nitrates are considered to be dangerous for the health of man and animal. Considering the last sheep had become ill by eating plants containing too much cupper. The cupper was added to the cattlefood to raise their weight more quickly and held by the manure polluted the groundwater and the plants. The same could have happened to men.

The most critical problem on short term however is the pollution of groundwater which is used for the preparation of drinkwater. In some areas where drinkwater is collected, there is a nitrate pollution of 40 milligram per litre. In 1985 50 milligram nitrate per litre drinkwater will be the maximum allowable concentration. In Holland a special law concerning the manure will be installed. In Germany several local authorities have installed limits for the allowed amount of manure to be dumped on the soil.

As mentioned before, the main reason for the manure problem is that there is a surplus of manure when or where it is not needed. To solve this, many farmers build a manure reservoir, so they can store their manure for themselves or for transportation to other areas.

DESCRIPTION MANURE RESERVOIRS

The reservoirs are constructed by excavation and using the outcoming soil for raising dikes around the excavated area.

Trenches are made at the bottom of the reservoir and a relatively cheap membrane is placed on the contour of the trenches and on the bottom of the reservoir. Then a PVC drain pipe is placed in the trenches and the trenches are filled completely with a granular material. These trench drains are connected with a vertical control pipe which makes it possible to see if any leakage occurs through the high performance geomembrane.

This high performance geomembrane which forms the actual impermeable liner of the reservoir is placed over the bottom, and the drainage system, and the slopes of the reservoir. On top of the embankment the geomembrane is anchored in the soil. Most of the time the geomembrane is delivered on the site as one unit able to cover the complete reservoir.

The contents of the reservoirs are variable. Most of the farmers prefer reservoirs of 1200 m3. The depth of the reservoirs is usually not larger than 3 m.

To prevent people from drowning when falling into the reservoirs and for preventing lifting of the geomembrane, at several places rows of tires are tied to the slope.

For a schematic overview of a reservoir and more details see Fig. 1 and Photographs 1, 2, 3 and 4.

DESCRIPTION TESTED GEOMEMBRANES

With this research program, carried out in 1983, two manure reservoirs were involved. One in Hengelo, reservoir A, and one in Vijfhuizen, reservoir B. Both reservoirs have been constructed in 1976.

The geomembrane used as an impermeable liner is the so-called Nicotarp 81. It is a polyethylene (HDPE) slitfilm woven fabric with a polyethylene (LDPE) coating on both sides. Specifications are:

Weight of the fabric		128 gr/m2	± 10%
Weight of the geomembrane		228 gr/m2	± 10%
Weight of the coating		50 gr/m2	on both sides
Breaking strength	warp	18 kN/ml	± 15%
Breaking strength	weft	18 kN/ml	± 15%
Strain at break	warp	20%	± 5%
Strain at break	weft	16%	± 5%
Number of ends		24 per 50 mm	
Number of picks		24 per 50 mm	

The separate geomembrane rolls are fabricated as a larger unit by sealing. The sealing method consists of a strip welded by hot air on the two edges of adjacent geomembranes. The strip is of the same material as the geomembrane itself. Usually the width of the strip is designed to meet a specific strength required. In these cases a 7 cm wide strip had proved to be satisfactory and had been used for reservoir A as well as for reservoir B. The strip, and the seal, runs parallel to the warp of the geomembrane. In both reservoirs the geomembranes have performed very satisfactory up to now. The maximum manure depth is about 3 m and at this pressure no leakage has been observed.

The next 4 samples have been tested by the "Staatlisches Materialprüfungsamt Nordrhein - Westfalen Dortmund".

Sample 1: Nicotarp 81, which had not been placed.
Sample 2: Nicotarp 81, from reservoir A, above the manure level.
Sample 3: Nicotarp 81, from reservoir A, below the manure level.
Sample 4: Nicotarp 81, from reservoir B, below the manure level.

TEST PROGRAM

1. Weight per unit area according to DIN 53352.
2. Thickness according to DIN 53370.
3. Breaking strength according to DIN 53857. This is a 2 inch wide strip test with a strain rate of 100 mm/min.
4. Strain at break according to DIN 53857.
5. Modulus according to DIN 53857.
6. Tear strength according to DIN 53363. This is a trapezoidal tear test.
7. Vapour permeability according to DIN 53122. The weight of vapour transmitted through the geomembrane is measured.
8. Water permeability according to DIN 16937. A water pressure of 100 kN/m2 is applied until water permeability can be observed.
8.1. Water permeability of the geomembrane itself.
8.2. Water permeability of the sealed joints.
9. Seal strength ratio according to DIN 53857. This is the ratio of the breaking strength of strips of geomembranes with and without sealed joints.

TEST RESULTS

The results of the tests are given in Table 1.

EVALUATION TEST RESULTS

Considering the weight per unit area, the thickness and the tear strength, no substantial difference can be found between the 4 samples. The variance in the vapour permeability is quite large.
The seal strength varied between 0,65 and 0,81 and is higher for the used geomembranes.
The water permeability tests for the geomembrane itself showed no leakage at all after 24 hours under a water pressure of 100 kN/m2 for all the 4 samples. The water permeability tests of the sealed joints showed some transmission of water under a water pressure of 100 kN/m2 for all the 4 samples.

These results however show no indication of relevant deterioration of the geomembrane. The only data which give real information on the deterioration of the geomembranes by environmental influences and manure, are the breaking strengths. Assuming the geomembranes placed in 1976 are comparable with the presently made geomembranes a slight decrease of the breaking strength can be noticed. It is not likely that this relatively small strength reduction will influence the performance of the geomembranes. It is surprising to see that the tear strength data are somewhat in contradiction with the breaking strength data and do not show the deterioration so obviously. Visual examination of the field samples showed no decoulering in time.

CONCLUSION

The geomembranes performed rather well and can be seen as a proper impermeable liner for manure reservoirs. However, although satisfactory in performance, the water permeability of the seal joints did not meet the water permeability of the geomembrane itself when tested at an overpressure of 100 kN/m2.

To overcome this difference a new geomembrane, called Nicotarp 100, with an improved seal joint has been developed. For the water permeability tests the improved seal joint gives the same good test results as the geomembrane itself.

Table 1.

Specifications of Nicotarp 81 Geomembranes which have been exposed in a manure storage reservoir since 1976 in relation to original specifications.

Specifications			Number of measurements, resp. test sample per material	Nicotarp 81 which had not been placed				Nicotarp 81 which had been placed in a manure reservoir in:											
								Hengelo (reservoir A), placed in 1976								Vijfhuizen (reservoir B) placed in 1976			
				Sample 1				Above manure level Sample 2				Under manure level Sample 3				Under manure level Sample 4			
				limit value	aver-age	standard devia-tion	variation coeffi-cient %	limit value	aver-age	standard devia-tion	variation coeffi-cient %	limit value	aver-age	standard devia-tion	variation coeffi-cient %	limit value	aver-age	standard devia-tion	vari. coef. %
Thickness		mm	10	0,36 0,39	0,38	0,01	2,6	0,37 0,45	0,42	0,02	4,8	0,37 0,44	0,40	0,02	5,0	0,45 0,49	0,48	0,01	2,1
Weight		gr/m2	5	224 225	224	1	0,5	229 249	243	8	3,3	-	-	-	-	223 230	227	3	1,3
Breaking strength	warp	N/50 mm	per direction 10	975 1172	1047	44	4,2	641 776	708	54	7,6	867 950	918	33	3,6	686 793	742	42	5,7
	weft			1078 1344	1248	84	6,7	1102 1188	1138	35	31	1283 1360	1328	28	2,1	827 1080	914	101	11,1
Breaking strain	warp	%		15,4 17,8	16,4	0,7	4,3	7,1 9,3	8,4	0,8	9,5	16,0 18,9	17,2	0,9	5,2	14,6 18,2	16,4	1,2	7,3
	weft			11,7 14,9	13,5	0,8	5,9	16,5 18,3	17,7	0,6	3,4	17,8 20,6	19,4	0,9	4,6	11,8 15,3	13,2	1,4	10,6
Modulus	warp	N/mm		98 123	118	7	5,9	129 142	136	5	2,7	97 108	102	5	4,9	80 105	87	9	10,3
	weft			162 178	169	5	3,0	119 127	122	3	2,5	133 143	138	4	2,9	123 129	127	2	1,6
Tear strength	warp	N	per direction 5	213 284	248	25	10,1	280 396	348	47	13,5	317 363	346	18	5,2	285 302	294	7	2,4
	weft			265 315	281	20	7,1	171 310	227	56	24,7	168 237	210	28	13,3	262 338	289	30	10,4
Water permeability		gr/m2/day	5	0,52 0,70	0,61	0,07	11,5	0,93 1,10	1,00	0,09	9,0	0,31 1,87	0,90	-	-	0,54 1,38	0,84	0,35	41,7
Vapour perme-ability	geomembrane itself		per 3	After 24 hours exposure to a waterpressure of 100 kN/m2 no leakage could be observed with the 4 samples															
	seal joint			After about 2 minutes exposure to a waterpressure of 100 kN/m2 some leakage could be observed with the 4 samples															
Seal factor			-	0,65				--				0,72				0,81			

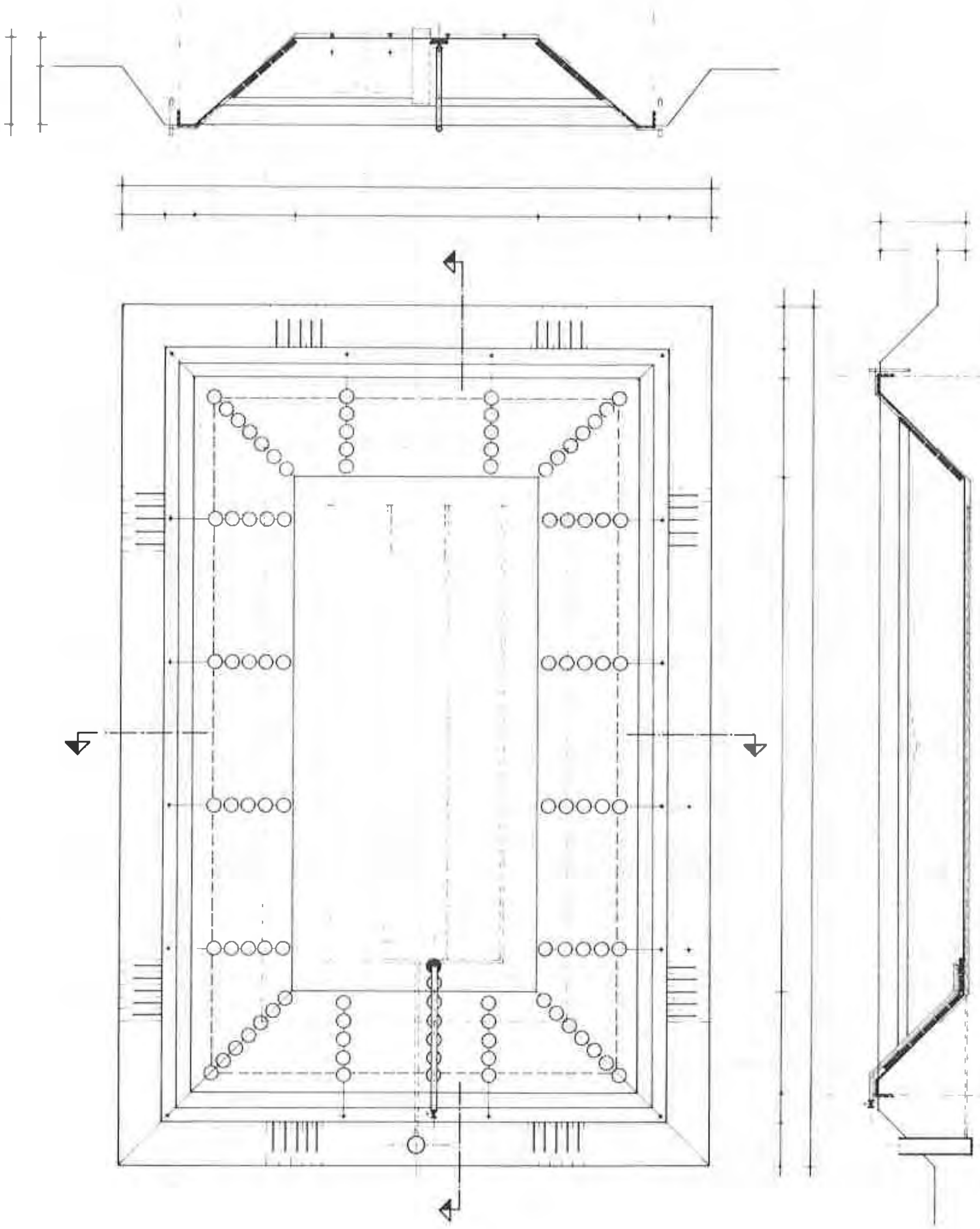

Fig. 1. Schematic description manure reservoir

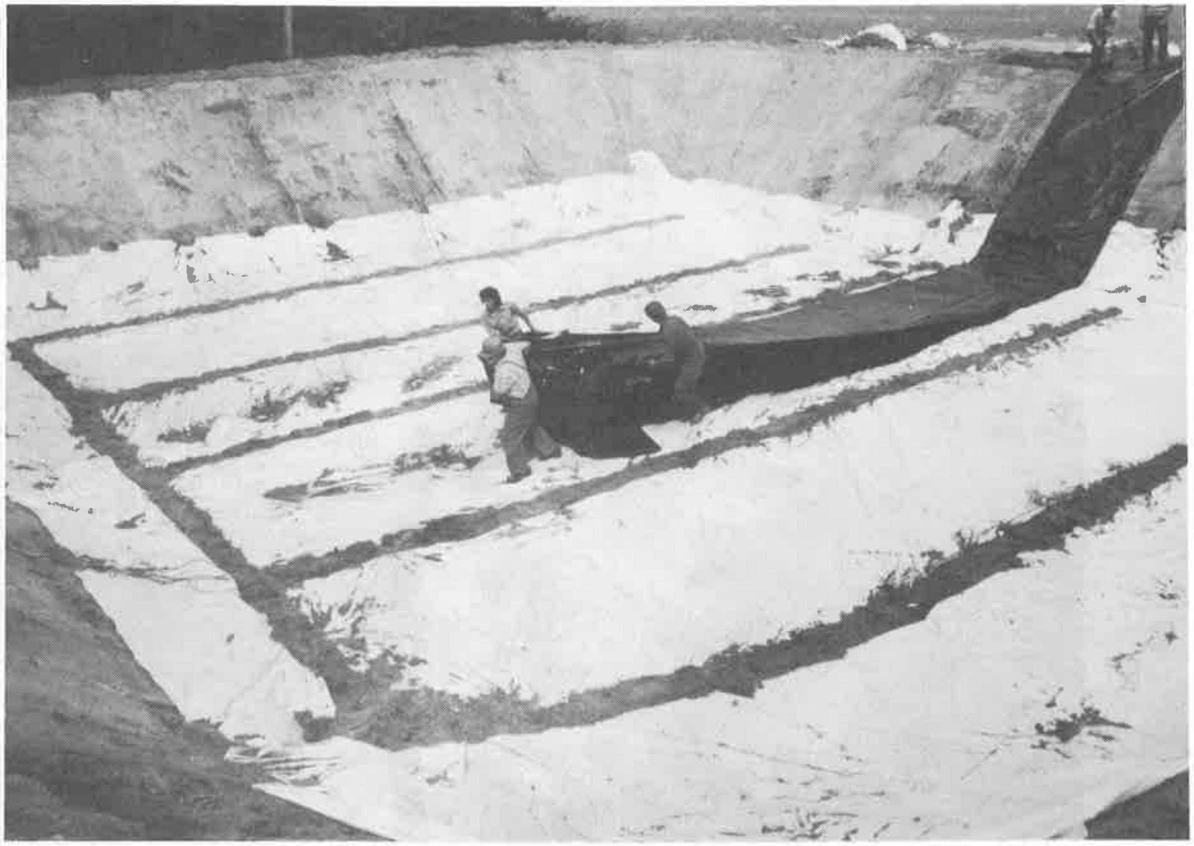

Photo 1.

Photo 2.

Photo 3.

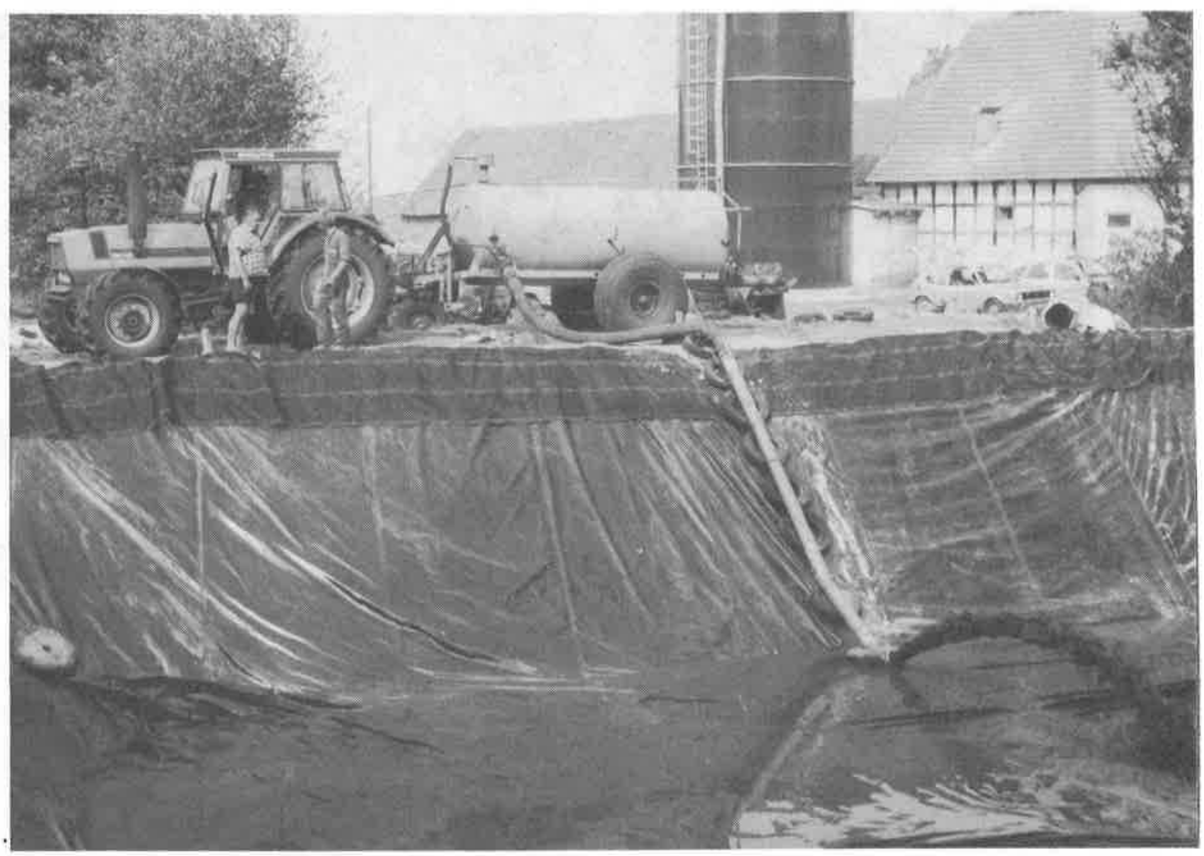

Photo 4.

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Use of Prefabricated Asphaltic Geomembranes for Storage of Hazardous Materials

In France, the waterproofing of major hydraulic structures - the Ospedale dam in Corsica (26m high, stored volume : 3.5 million m3), or the recreational lake in La Courneuve (near Paris) with a water-tight surface of 125,000 m2, or the fresh water reservoir of the three EDF nuclear plants in Paluel, Flamanville and Panly on the Coast of Normandy - was soon followed by th waterproofing of many industrial water storages (for sugar refineries, paper mills, canning factories or water treatment plants), and of industrial platforms used for treating uranium ore by lixiviation.

In view of these references and of the results of the behavior tests carried out on asphaltic geomembranes with respect to physical or chemical attacks, reinforced asphaltic geomembranes have been used for several years in France and West Germany for waterproofing household, industrial or special non-toxic refuse dumps.

DESCRIPTION OF THE PREFABRICATED ASPHALTIC GEOMEMBRANES

The industrially-produced asphaltic membranes that we have used for the two French and German structures described hereunder, are made of a non-rotting, non-woven polyester fiber reinforcement impregnated on both sides by blown asphalt or elastomer-modified asphalt. A glass fiber layer distributes the heat stresses created during induction. One side is sanded to prevent them from sticking when rolled up, and the other side is lined during the manufacturing process with a (Terphane) polyester film as a protection against perforations by vegetable roots.

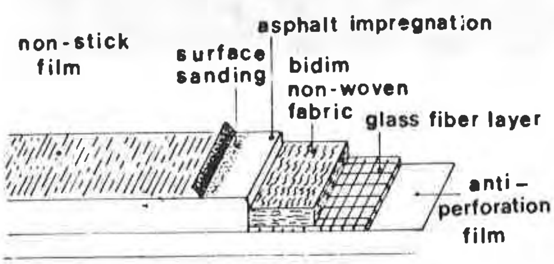

Figure 1 : Asphaltic Geomembrane

When the geomembrane is to withstand special chemical attacks, it may be protected by a plastic sheet glued during the manufacturing process as an alternative to the non-stick layer of sand, or by a course of cold tar laid in situ, for example :

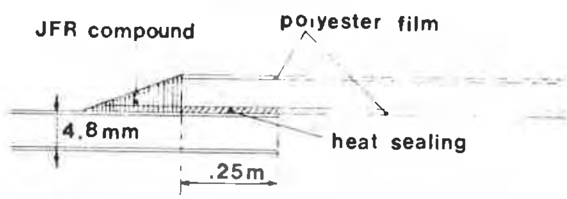

Figure 2 :

The asphaltic geomembranes used in these structures belong to one of grades 2-3-4 or ES.

Delivery and installation

The geomembranes are supplied by the factory in rolls of 1,500 kg, 4 meter wide and from 55 to 80m long.

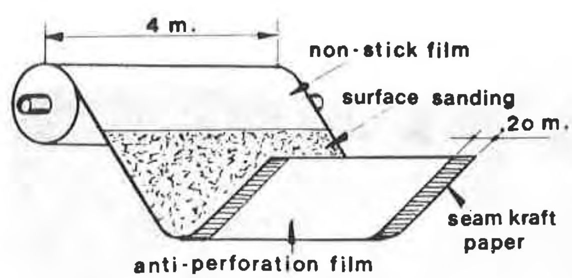

Figure 3 :

The rolls are unrolled using a hydraulic shovel fitted with an unroller.

Photograph n° 1 .Placing of Geomembrane (West Germany)

The seams are performed on a strip overlap of at least 20cm and, in view of the sufficient amount of asphalt on each strip, are sealed by heating asphalt on the surface with hot air or a blow torch. The overlap is then compressed mechanically while the asphalt is still liquid.

Connection to the structures is achieved by heat-sealing of the geomembrane after application of a bituminous coating. The operation is completed by mechanical fastening.

EXTRA PRECAUTIONS WHICH MUST BE TAKEN UPON EQUIPPING REFUSE DUMPS.

1 - Frequently, the bottom of the structures must be fitted with draining systems to prevent water or gas from rising under the effect of the uplift pressures created by the environment or by gas emissions resulting from the fermentation in the soil.

2 - The geomembranes used for refuse dumps must be protected against the mechanical attacks of the spreading and compacting machines or of the products laid.

Photograph n° 2 : Protecting sand layer on Geomembrane - Fosse Marmitaine Landfill (France)

This is generally achieved by placing a layer of inert materials on the bottom and cohesive materials - e.g. bituminous mix or lean concrete - on the slopes.

3 - The refuse dumps which are widely made of successive areas, are equipped with wells which are used for pumping the leachates created by rain water and the deterioration of refuse, and also for collecting the fermentation gases

DESCRIPTION OF THE TWO STRUCTURES PERFORMED IN FRANCE AND WEST GERMANY.

1 - Controlled refuse dump at La Fosse Marmitaine, near Rouen (France)

1.1. - Geometric definition

The controlled refuse dump at la Fosse Marmitaine covers a 39 hectare area on a 45-hectare plot of land which belongs to the Seine Maritime Department. The available depth is of 12m approximately over the whole dump. It is therefore a site which can be used for storing about 6 million m3 of non toxic, household, industrial and special refuse.

1.2. - Operating principle - Phasing

The total surface of the dump is divided into ten pits which are prepared and operated successively : each pit is divided into compartments where refuse is dumped, the 2.5m compacted layer being separated from the upper layer by 20 or 50cm of inert materials found, if possible, on the spot.

Phasing therefore consists in operating a pit and in rearranging and reafforesting the recently filled pit.

Photograph n° 3 : Landfill operation phase one and gravel emulsion protection on the slope

1.3. - Construction processes

Two special thin waterproofing devices have been selected for bottoms or slopes. The total supplied surface is about 42,000m2. On the bottom, where leveling and compacting have been carefully controlled, waterproofing is provided by a tight antikerosene controlled, waterproofing is provided by a tight antikerosene membrane made up of a bituminous membrane lined at the plant with a thick Terphane film. This extra requirement is due to the possible presence of some hydrocarbon in the special refuse.

The protection layer at the bottom of the pit consists of a course of 0/2mm ultra-fine sand from the site spread over a .50m thickness

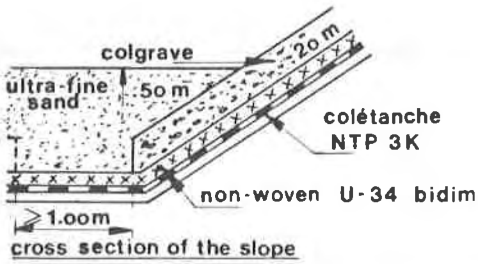

Figure 4 :

After a 1/2 slope adjustment and compacting, the slopes are lined with a thin waterproofing system made up of a Type 2 waterproofing film without any special anti-kerosene treatment, and of a protection layer incorporating a U 32 protective Bidim and a 20cm layer of 0/16 sand and gravel emulsion with 88% flint materials, 12% 0/2 limestone and 5.6% 60%-H acid emulsion.

- LCPC compactness $>$ 85%
- simple compressive strength (80/100 asphalt) $>$ 30 bars
- immersion/compression coefficient $>$ .55.

Its good resistance to punching and impacts, its draining qualities and good slope behavior in particular under the effect of rain water have made the sand and gravel emulsion a good protection against impacts from the buckets or blades of the specialized bulldozers or from tamping foot compactors, which could be laid a long time before any prolonged operation.
Given the geographic situation of la Fosse Marmitaine, special care has been given to the draining system and to the treatment of the rain water which percolates through the dumped refuse.

Water is then recuperated at the center of the pit by a draining system placed on the waterproofing, and consisting of clay pipes Ø 300 coated with a 0/60 draining sand and gravel mixture and covered with a layer of 0/20 draining sand, gravel and asphalt mixture.

The system is completed by four inspection pits installed as the pit fills up. These inspection pits are also used for discharging the fermentation gases. Then, the leachate is recovered in a pool to be analyzed and treated, if necessary, before flowing back into the River Seine.

Toning-in, peripheral afforestation.
Besides the performance of the civil engineering works described above, and in compliance with regulations on the toning of the controlled refuse dump in the site, this firm has been entrusted with the peripheral afforestation of the site which aims at protecting the frontagers ; part of the trees are planted on the periphery while others are planted on a tree nursery with a view to create a forest area 50m deep which will be planted on a layer of topsoil or compost placed on top of the waterproofing membrane laid over the whole dump once filled up.

2 - Waterproofing of the controlled refuse dump of Saint-Augustin Niederpleis, near Bonn (West Germany)

2.1. - In 1981, the association for the elimination of the refuse of the Rhein-Sieg district in Bonn-Saint-Augustin opted for the reinforced prefabricated asphaltic geomembranes for waterproofing the first bottom trench pf a 43,000m2 refuse dump.

Photograph n° 4 : Saint Augustin Landfill (West Germany)

The geomembrane used is of Type 2, i.e. 3.9mm thick and weighing 4.5kg/m2. The structure of the geomembrane is as described above. A control system has been added to the conventional laying processes. It consists of a vacuum test performed on the membrane in the overlapping and sensitive areas.

Photograph n° 5 : Vacuum Test Saint Augustin Landfill (West Germany)

This is achieved by using a plexiglass bell with a .3 bar vacuum maximum ; beyond that limit, the membrane is sucked against the dome of the bell. Soap suds are spread over the test surface and leaks are detected by noticing the formation of bubbles. The smallest holes can thus be detected.

The test is blatantly convincing, rapid, simple and economical.

2.2. - Description of the structure at the bottom of the dump

Structure is as follows :

- subgrade made of materials excavated from the former gravel pit
- a 20cm foundation course and a layer of gravel for uplift pressure distribution, with E_{V2} = 40MN/m2.
- fine adjustment of the compacted grade at + 2cm, type 2 geomembrane, - water-tight bituminous membrane with an overlap of at least 20cm on the bottom and on the slopes.
- 20cm gravel draining course used as a protective layer
- surface drainage through clay pipes 125 and 250mm in nominal diameter.

The downgrade is .7% from the median axis. Water is drained sideways to a storage pool waterproofed with asphaltic membranes and then to a water treatment plant. The water-tight membrane is anchored in the slopes.

2.3. - Properties of the Type 2 reinforced asphaltic geomembrane

2.3.1. Mechanical specifications

- Tensile strength and elongation at breaking point, lengthwise and crosswise, for temperatures T ranging from - 15° and + 50° C and test speeds v ranging from .1 to 100mm/min.

Example : T = 20°C, v = 100mm/min. (average of 4 samples) :

	Tensile Strength N/5cm	Elongation %
Lengthwise	1120	35.5
Crosswise	1420	36.1

- Tensile strength after aging and elongation at breaking point (behavior after storage in a hot environment).
Ex. after two months of storage at 50°C
T = 20°C, v = 100mm/min.

	Tensile Strength N/5cm	Elongation %
Lengthwise	1170	41.8
Crosswise	1415	32.5

- Irreversible tensile elongation. Ex : elongation at v = 1mm/min. to 25% of the average elongation at the breaking point ; after 24 hours, relaxation of elongation at the same speed ; measurements are performed again after 30 minutes

elongation at T = 20° C	54%
elongation at T = 70° C	68%

- Fatigue strength ;
Ex : for T = 0°C, v = 100mm/min.
and alternate stress at up to 10% of elongation at the breaking point :

Lengthwise test	Tensile Strength N/5cm	Elongation %
Before the fatigue test	415	30.4
After 5,000 alternations	280	30.5

- cold behavior : after a 190° bending round a pin 2cm in Ø, at 0°C for 20s, no crack was noticed,

- water permeability after placing the sample in water for 28 days : .56% in vol. from the initial volume.

- resistance to water pressure (coefficient of permeability according to Darcy's Law) : with a pressure of 1 bar for 4 days,
k is $< 10^{-12}$ m/s.

- resistance to water pressure (pressure test on a crack) : test performed on a free surface 1 cm x 8cm submitted to a pressure increase from 1bar/15min. to 30 bars ; no crack.

- thermal expansion after storing the sample for one hour at 70°C and cooling it down to 20°C :

	Expansion in Volume
Across	+ .03
Lengthwise	+ .38

- punch strength : a steel punch with a surface area of 10mm falls on the sample from a height of 75cm ; no imprint.

- weight per square meter of the reinforcement : 210g/m2.

- soluble body content : 67.85% in pds.

- strength of the seams (welding coefficient) : it is not necessary to perform this test on bituminous membranes given their creep behavior : stresses are always nil in the overlapping area, given the tractive speed and temperature.

2.3.2. - Physical specifications

- resistance to UV radiation. Test : exposure to the radiation of a xenon arc lamp and repeated moistening of the sample ; after 500 hours, there is no blistering nor crack.

- reduced tensile strength after an exposure to UV radiation.
Example : T = 20°C, v = 100mm/min., relative moisture ranging from 55 to 60%

USE FOR PREFABRICATED ASPHALTIC GEOMEMBRANES
FOR STORAGES OF HAZARDOUS MATERIALS.

	BREAKING STRENGTH	
	before exposure to UV radiation N/5cm	after exposure to UV radiation N/5cm
Across	840	950
Lengthwise	740	740

2.3.3. - Chemical specifications

Photograph n° 6 : Aerated lagooning in Vic-sur-Aisne (France)

- resistance to weak acids, phenol, detergents, salt : proven.
- resistance to petrochemicals such as gasolene, fuel oil, Diesel fuel, benzol, etc : it is reduced and it is necessary to protect the surface of the membrane with a extra plastic sheet or a tar emulsion.

Photograph n° 7 : Sulfuric Acid leaching process Uranium Ore - (France) -

CONCLUSION

The structures described above and performed over the past few years have been developed on the very sites or have served as references when applying the process to other structures of the same type or to structures where doughy industrial refuse is dumped.

The qualities of the prefabricated asphaltic geomembranes and the constant care given to their installation and to the construction of supplementary equipment have led to the erection of structures which are giving full satisfaction to the developers, operating companies and neighboring communities.

Their reliability, durability and easy use may well permit further applications in the future.

Many tests have been carried out both in North America and in Europe on the products under very hard conditions, for example at extremely low temperatures ; the tables of results cannot be included in the present document, but they provide engineers with numerical values which can be taken into account when doing calculations on the structures or estimating their behavior. (These data are available).

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Oil Spill Containment Liners for Artificial Drilling Islands

SUMMARY

Esso Resources Canada Limited is constructing 6 artificial drilling islands in the Mackenzie River at Norman Wells, N.W.T., Canada. The island design includes a geomembrane oil spill containment liner below the work pad surface. Field tests were performed on CPE, Hypalon, and cross-laminated LDPE to determine the degree of puncturing caused by placement of the coarse sand and gravel fill material over the membranes. The liners were tested without geotextile support, with a geotextile on top, and geotextile on both sides of the liners. The tests indicated that the stiff LDPE material is badly damaged by coarse cover placement, although the geotextile reduced the number of punctures. Some puncturing occurred in the flexible CPE and Hypalon materials; geotextile protection reduced the number of punctures somewhat. Small punctures may be due to angular sand particles on top of smooth, rounded gravel surfaces. The geotextile protection mechanism appears to be provision of a passive mat rather than tensile support.

1. INTRODUCTION

Field tests to assess geomembrane resistance to puncturing during placement of sand and gravel cover material were performed by Esso Resources Canada Limited to aid in the selection of a suitable geomembrane material and installation procedures for oil spill containment liners on artificial drilling islands in the Canadian arctic. This paper discusses the test procedures and results, the liner design and restraints imposed by the service environment, and the significance of punctures in geomembrane liners used for oil spill containment.

2. PROJECT DESCRIPTION

2.1 Island Design

Esso is constructing 6 artificial drilling islands in the Mackenzie River, Northwest Territories, Canada, as part of the Norman Wells oil field expansion project. The river is about 2 km wide and up to 12 m deep at Norman Wells, and is subject to flooding during spring break-up of ice jams. The islands measure 45 m by 80 m on the work pad surface and support structures for the drilling and production of 16 wells. The islands rise about 12 m above normal river levels to provide clearance above design flood levels. One of the constructed islands is shown on Fig.1.

Fig.1. View of constructed artificial drilling island

2.2 Oil Spill Containment Liner

The island design includes an oil spill containment liner placed below the work pad surface (Fig.2). Liquids that might be contained by the liner include oil and gas products associated with drilling and production activities. The liner forms a basin below the work pad surface that drains toward a weeping tile system adjacent to the wellhead cellar. Collected liquids are pumped to the mainland for disposal. A geomembrane liner design was chosen over compacted clay because of the potential for drying and cracking of a clay liner and because of permafrost conditions in the clay borrow deposits. Installation and service environment constraints affecting liner performance include the low ambient temperatures (near 0^{o} C during installation), the remote location, the river construction environment, the coarse nature of the island fill, the chemical properties of the liquids to be contained, and potential differential settlement of the island fill.

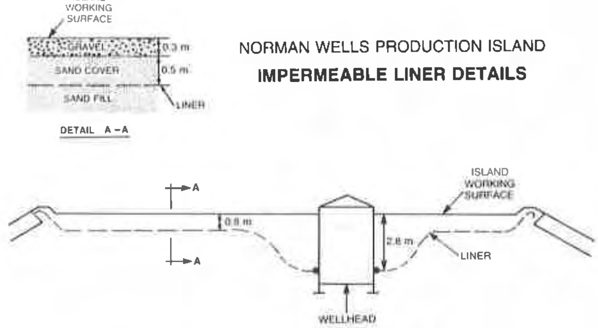

Fig.2. Cross-section showing liner design details.

2.3 Liner Field Tests
Although the island fill material is rounded to sub-rounded alluvium, there was concern that the coarse particle sizes might damage the geomembranes if used as liner bedding and cover material. However, the cost of processing fine granular material would be high, and since there was little published data on the degree of puncturing anticipated using coarse, rounded alluvium under field loading conditions, Esso decided to carry out field puncture tests as described in the following sections.

3. TEST PROCEDURE

3.1 Material Properties
The liner field puncture tests were carried out on one of the islands during construction, using the actual island fill and construction equipment. Three different geomembrane materials were tested -- cross-laminated low-density polyethylene (LDPE), chlorinated polyethylene (CPE), and chlorosulfonated polyethylene (Hypalon®). Unsupported membranes were used because the material would be sandwiched in the island fill below a relatively level surface. The properties of the CPE and Hypalon are shown on Table 1, and the LDPE properties are shown on Table 2. The LDPE is a fairly strong but stiff and brittle material that has been successfully used by Esso in the Beaufort Sea in sub-zero temperatures. The CPE and Hypalon are well known thermoplastics. Each geomembrane was tested under 3 conditions: without geotextile protection; with a thick nonwoven geotextile on top of the geomembrane; and with the geotextile on both sides of the geomembrane. The properties of the geotextile are shown on Table 3.

Table 1. Physical Properties of CPE and Hypalon.

Property	CPE	Hypalon
color	black	black
mass, g/m^2	1025	1075
thickness, mm	.75	.75
specific gravity, ASTM D792	1.29	1.38
tensile strength, MPa, ASTM D882	10.3	10.3
elongation at break, %, "	325	300
tear strength, N/mm, ASTM D1004	44	39
cold bend, °C, ASTM D1790	-32	-43
resistance to soil burial: ASTM D3083	pass	pass
ozone resistane, ASTM D1149	no effect	no effect
bonded seam strength, % of tensile, min., ASTM D3083	90	90
percent resin	50	45

Table 2. Physical Properties of LDPE.

Property	Value
color	gray
thickness, mm	.15
specific gravity, ASTM D1506	.93
mass, g/m^2	151
tensile strength, MPa, ASTM D882	51.7
elongation at break, %, "	120
impact strength, kg/cm, ASTM D781	250
tear strength, N/mm, ASTM D1004	70
max. service temp., °C	93
min. service temp., °C	-57

The alluvium used for constructing the islands is a well graded sand and gravel with rounded to sub-rounded gravel sizes. The grain size distribution envelope for the island fill is shown on Fig.3, while the material used in the field tests is represented by the curve within the envelope. A photograph of the prepared test area subgrade is shown on Fig.4. Photomicrographs of the sand fraction of the fill show that the grains are rounded to angular (Fig.5).

Table 3. Physical Properties of Nonwoven Geotextile.

Property	Value
color	white
compound	polyester
mass, g/m^2	400
specific gravity	1.38
thickness (virgin)	3.8
tensile strength, N, CSA2-4.2-M77(9.2)	800
elongation at failure, %, "	70-100
Mullen burst, kPa, CSA2-4.2-M77(11.2)	2800
ball burst, N, "	1550
pore size, μm	30-80
hydraulic conductivity, cm/s	0.26

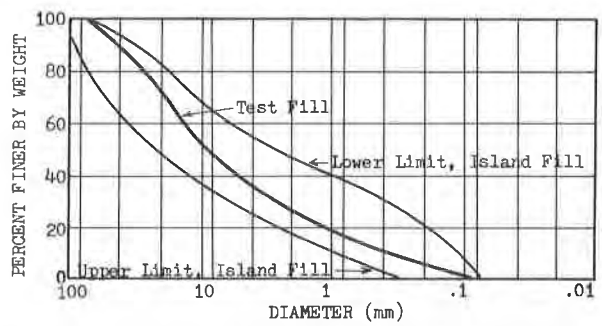

Fig.3. Grain size distribution of sand and gravel fill.

Fig.4. Photograph of test area subgrade surface.

3.2 Field Procedure
Geomembrane specimens 2 m^2 in dimension were laid out on a relatively uniform, compacted fill surface in the test area (Fig.6). The same fill material as used for the liner subgrade was then pushed onto the liner test specimens by a D-6 Caterpillar bulldozer on wide pads (Fig.7). The D-6 has a bearing pressure of 3650 kg/m^2. A minimum of 450 mm of fill was maintained between the D-6 and the liner surfaces at all times. A final fill cover of 500 mm thickness was compacted with 10 passes of the D-6 to simulate construction loads. The upper

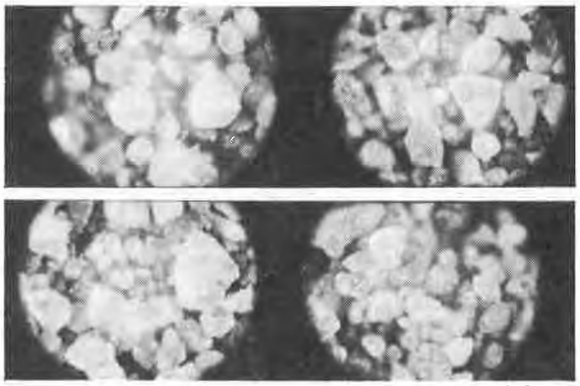

Fig. 5. Photomicrograph of sand particles in fill.

portion of the fill placed over each liner specimen was removed by a small backhoe. The final 150 mm of fill were removed by hand. The exposed liner segments were then observed and photographed in situ, then 1 m^2 samples were cut out for more detailed inspection.

← 2m →

2m	Geotext. HYPALON	Geotext. CPE	LDPE	CPE	Geotext. CPE
	Geotext. HYPALON Geotext.	Geotext. CPE Geotext.	HYPALON	Geotext. LDPE	Geotext. LDPE Geotext.

Fig.6. Layout of liner test specimens in field.

Fig.7. Fill being placed over test area by D-6.

4. TEST RESULTS

4.1 Observations of In Situ Liners

In all cases the exposed portion of the test specimens showed signs of distress due to the soil cover placement. This was characterized by a bumpy surface, abrasions, folds, holes, and some tears. The more flexible Hypalon and CPE materials (eg. Fig.8) showed less damage than the LDPE (Fig.9), although the surfaces of the Hypalon and CPE were more dimpled and plastically deformed. The LDPE membranes showed obvious signs of severe distress, including large holes and tears. In most cases holes could not be identified in the Hypalon and CPE membranes prior to cutting out a portion and holding it up to the light.

Fig. 8. CPE after test, geotextile both sides.

4.2 Puncture Counts

Sections measuring 1 m by 1 m were cut from the exposed liner specimens and held up to the light to count the number of visible punctures. Punctures were divided into two general size ranges -- less than or equal to 2mm in average diameter, and greater than 2mm in diameter. Although the 2mm size is arbitrary, it appeared to be a suitable boundary between the larger number of small, more uniform holes, and the fewer number of large, more irregularly shaped holes. Dividing the puncture count this way may provide some additional qualitative information on the puncture mechanisms. The results of the puncture counts are shown on Fig.10 for the different geomembrane and geotextile combinations.

5. DISCUSSION

5.1 Existing Information on Geomembrane Puncturing

There is little information in the literature on the occurrence of punctures in geomembranes, and on the resulting seepage rates through the liners However, there is strong evidence to suggest that puncturing is not infrequent, particularly when covered with soil, and that holes together with faulty seams and joints with structures make a completely leak free installation a very costly if not unlikely event (1). In all cases a finite amount of liquid transmission will occur by diffusion of molecules through the membrane under vapor pressure and possibly hydraulic pressure differentials. In addition, dissolved solids can diffuse through membranes under osmotic forces (2). However, transmission of liquids and contaminants by these later mechanisms is likely to be negligible compared to transmission rates through discontinuities in the liners.

There is contrary opinion regarding how smooth the liner bedding surface must be and maximum allowable particle size; however, it is commonly agreed that the surface must be free of sharp and angular objects and gravel (2). Similar recommendations are made for cover material, and the EPA recommends a minimum first lift thickness of 450 mm (2). Others recommend lift thicknesses as thin as 150 mm (1). In all cases there is no indication of the likely number of punctures or their effect on seepage rates.

In tests carried out by the Waterways Experiment Station a fine grained sand cover material was placed over various geomembranes that were placed on different textures of subgrade (3). The sand cover thickness varied from 150 to 450 mm. In some cases a thin (0.75mm) nonwoven geotextile was placed below the geomembranes. The sand cover was placed and compacted by 10 and 30 passes of rubber-tired, tracked, and cleated vehicles. The results of the tests were somewhat inconclusive with respect to the effect of cover thickness and the value of the thin nonwoven geotextile, but did indicate that a significant number of holes could be expected when the subgrade material contained gravel sizes. The actual grain size distributions and particle angularity were not provided. In several cases holes still resulted when the subgrade material was the same sand as used for the cover material.

Laboratory tests have indicated that thick nonwoven geotextiles can significantly reduce the number of punctures in some geomembranes (4). Nevertheless, a few punctures still occurred over a relatively small liner area with the geotextile protection. The relation between the laboratory and field loading conditions is not clear.

Some reports of field observations of liner punctures below soil covers indicate that no puncturing occurred (5,6). However, in the tests reported herein small holes in the liners could not be detected until the sheets were held up to the light.

Assuming that some puncturing does occur, there is little information available to allow prediction of the seepage rate that might occur because of the punctures. Earlier studies by the East Bay Water Company of Oakland (1) resulted in the creation of a seepage tolerance equation:

$$q = A\sqrt{H}/80 \qquad 1$$

where q is the maximum acceptable seepage rate in U.S. gallons per day, A is the lined area in thousands of square feet, H is the pond depth, and 80 is an empirical factor. The authors have intentionally left the equation in the original units for easy identification. The equation infers that lower seepage rates will be extremely costly and difficult to guarantee, although the denominator has been increased with time to values as high as 160 to 200 (1).

Eqn.(1) is similar in form to the theoretical equation for the rate of flow through an orifice in a large tank:

$$q = A\sqrt{2gH} \qquad 2$$

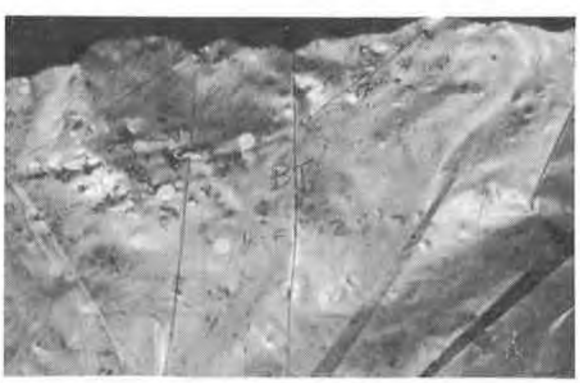

Fig. 9. LDPE after test, geotextile both sides.

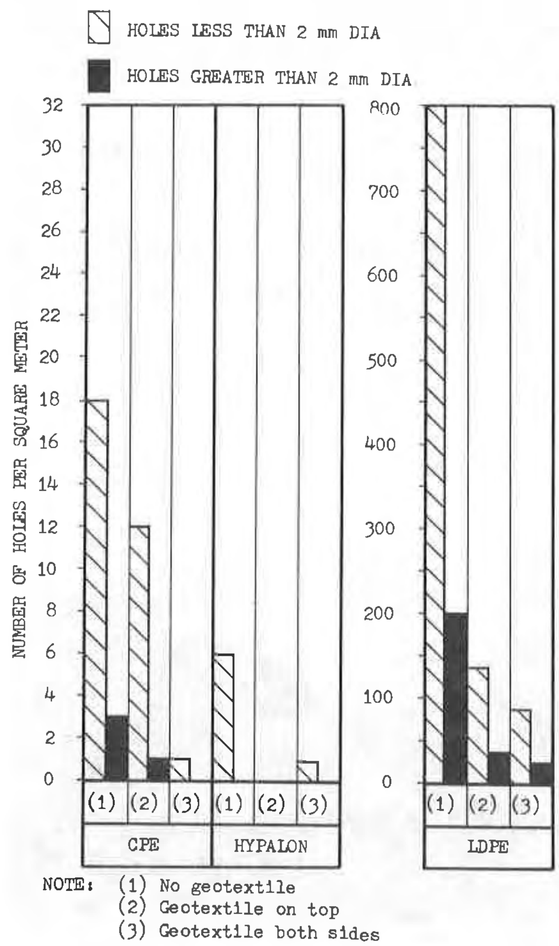

Fig. 10. Number of punctures observed in geomembranes.

where g is the gravitational constant, H is the hydraulic head, and A is the area of the orifice. If eqn.2 is used without correction for entrance effects or flow jet contraction, then an orifice (i.e. pinhole) area of 0.4 mm^2 per 1000 m^2 of liner area results in the same computed seepage rate as by eqn.1. If a denominator of 200 is used in eqn.1, then the pinhole area is only 0.2 mm^2. It should be noted that these seepage rates are not insignificant and are comparable to those that might occur through a good compacted clay liner with a hydraulic conductivity value in the order of 10^{-7} to 10^{-8} cm/s. The actual seepage rate through liners in the field will be affected by factors such as the hydraulic properties of the underlying and overlying soil, saturation conditions, hole shape, and the presence of particles blocking the holes. However, even if the seepage rates calculated by eqn.2 are an order of magnitude too high, the seepage rates through a relatively small number of pinholes would still be significant.

Giroud (7) presents an equation for predicting seepage through a geomembrane based on the hydraulic conductivity of the membrane, seepage through pinholes, and seepage through larger holes. However, the derivation and use of the equation were not described in the reference.

Surface tension forces will inhibit seepage through small pinholes when the hydraulic head is low and the void space below the pinholes is unsaturated (1). The hydraulic head required to drive water through various diameter pinholes is shown on Table 4.

Table 4. Hydraulic head supported by capillary forces in pinholes.

Radius of Pinhole (mm)	Hydraulic Head Supported (mm)
10	1.5
1	15
0.1	150
0.01	1500

5.2 Field Test Results

The results of the field puncture tests reported herein indicate that significant puncturing can occur when geomembranes are placed on and covered by coarse grained sand and gravel, even when the gravel sizes are rounded, a lift thickness of 450 mm is used, and the material is placed by low bearing pressure construction vehicles.

The LDPE was badly damaged by the soil cover placement due to the stiff and brittle nature of the material, and despite its relatively high strength. The quoted elongation at break for the LDPE may be misleading in these applications because of the stiff initial modulus of LDPE. The CPE and Hypalon were more deformed by the load, but this flexibility resulted in a very low number of holes compared to the LDPE. The thick geotextile contributed significantly to the prevention of punctures in the LDPE on a relative scale. Since the LDPE modulus at low strains is stiffer than that of the geotextile, it is questionable whether the geotextile provided any tensile support for the liner. It appears more likely that the geotextile protected the LDPE by providing a passive mat (Fig.11). This would explain why the tests using a much thinner geotextile (3) did not indicate any significant geotextile protection. The thick geotextile may provide tensile support for the more flexible CPE and Hypalon materials; however, this support may diminish with time due to fabric creep. The relative performance of the LDPE and CPE and Hypalon leads to some question regarding the merits of fabric reinforcement in geomembranes when subject to similar loads, since the reinforcement may reduce flexibility and concentrate stresses in a manner that increases the number of punctures.

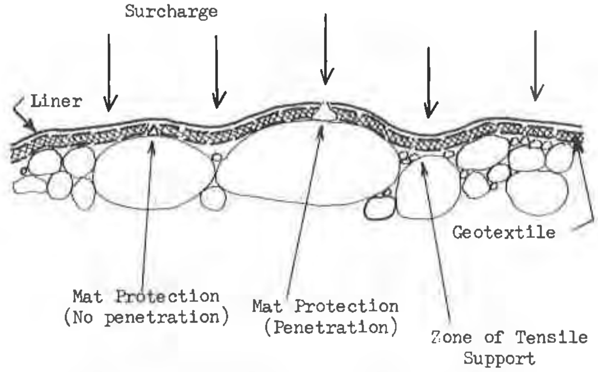

Fig. 11. Geotextile support of geomembranes.

The predominance of small holes in the geomembrane suggests that the more angular sand particles, rather than the more rounded gravel sizes, are causing puncture of the geomembranes. It is possible that angular particles create artificial sharp asperities on the surfaces of the larger stones (Fig.11).

6. LINER INSTALLATION AND PERFORMANCE

CPE material, without geotextile support, was chosen for the Norman Wells island liner design. The selection was made on the basis of the liner test results, cost and resistance to hydrocarbons.

Although the field tests indicate that slight puncturing of the liner would occur as a result of soil cover placement, the majority of these punctures would be less than 2mm in diameter. Since the liner could only be subject to small heads of liquids, surface tension forces would prevent seepage through most of the holes.

CPE liner material was successfully installed on the four artificial islands constructed in 1983. Installation temperatures were below 0°C for some of the islands; thus temporary shelters and heaters were required for seaming of the liners. Based on the field puncture test results D-6 dozers, 980 loaders and 12G graders were permitted to traffic on a minimum cover of 500mm of fill over the liner. The liners were inspected in-situ from test pits dug through the fill and appeared no more distressed than in the field tests.

Due to safeguards built into the oil drilling and production operations, the risk of a spill into the lined area is small. However, if a spill were to occur, the liner would limit the seepage of spilled fluids into the island fill, allowing collection of the fluids for drainage. In this light, the CPE material, as installed on the Norman Wells island, provides an effective oil spill containment liner.

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Installation and Field Quality Control of Multiple Layer Liner System for Residual Waste Disposal

In contrast to past industry practices of uncontrolled dumping and ravine-filling, current solid waste disposal operations are governed by well-documented regulatory agency solid waste management rules and regulations. These regulatory criteria establish the design requirements of liner systems for the collection and management of leachate at these facilities.

To effect the intent of a proper design, however, actual construction of the leachate collection systems, liners, and final slope caps must be satisfactorily accomplished in the field. At a recently completed expansion of a non-hazardous landfill, a construction quality control program was developed and implemented for inspection and field testing to monitor liner system construction and assure compliance with the design. The end product was a high quality functional liner system, whose general integrity was checked in the field prior to the initiation of waste placement operations.

INTRODUCTION

The subject quality control program was performed in conjunction with an expansion of the non-hazardous solid waste disposal landfill facility operated by Geological Reclamation Operations and Waste Systems (GROWS), Inc., a wholly-owned subsidiary of Waste Management, Inc. Located approximately 35 km northeast of Philadelphia, Pennsylvania, the facility is situated adjacent to the Delaware River. The facility is operated under a permit for solid waste disposal granted by the Pennsylvania Department of Environmental Resources for a 35 acre area known as the Hughes Landfill.

This expansion of an existing landfill operation and the appurtenant leachate collection system and liner were designed to meet, at a minimum, the requirements of the Pennsylvania Solid Waste Management Act, Title 25, Part I, Subpart C, Article I, Chapter 75 "Solid Waste Management - General Standards For Sanitary Landfill".

LINER SYSTEM DESIGN DETAILS

Pertinent design features of the project include a flexible double liner system with appurtenant pipe drain networks for leachate collection and management. A controlling factor of the total system design was that approximately 75 percent of the expansion area is underlain by an abandoned landfill. The flexible liner systems were selected to accommodate the anticipated total and differential settlements predicted within the old refuse due to the loading of the new landfill expansion.

The upper liner is 0.76 mm (30 Mil) high density polyethylene (HDPE) and the bottom secondary liner is compacted clay, ranging in thickness from 0.61 m (2.0 ft) for areas not underlain by refuse to 1.22 m (4.0 ft) for all areas underlain by the old landfill. The clay liner forms the bottom barrier isolating the system from the existing site soils. Design requirements of the clay were an in-place permeability not exceeding 1.0×10^{-7} cm/sec. For all cases, the selected liner thicknesses exceeded regulatory minimums.

The two liners are separated by a granular free-draining material used as a liner monitoring (leak detection) zone 0.3 m (1.0 ft) thick with an internal perforated-pipe drain network. The pipe drains were sized and spaced to expeditiously evacuate potential fluids that could accumulate within this zone should a breach of the primary (HDPE) liner occur. The HDPE liner is overlain by a combination protective granular cover and leachate collection zone. This zone, approximately 0.46 m (1.5 ft) thick, is site processed (screened) sand and is incised with a perforated-pipe gravity drain network. The pipe drains were sized and spaced to effect a low-head system for the evacuation of leachate from above the primary liner. This low-head system reduces the hydraulic head from leachate on the primary liner and correspondingly reduces the flow gradient for passage of liquids through potential breaches, thereby reducing the chance of significant releases.

The liners, collection and detection pipe systems and drainage zones were constructed at minimum grades of approximately 1.5 percent (50% greater than State requirements to accommodate settlement). The clay liner is underlain by a compacted granular subgrade fill constructed to form the required system slope configuration. All overlying drainage zones, pipes and liners are situated on grades parallel to that of the subgrade surface. The separate liner system components are contour graded to form separate drainage "bays", each of which drain to independant perimeter manhole structures for visual inspection of system performance, i.e., presence or absence of flows.

This segregation was provided to permit evaluation of long term system performance and allow isolation of the collection and detection drain systems of subareas of the total landfill area. The above-ground sloped configurations allows generated leachate to be removed continuously by gravity through the pipes and drains. Perimeter containment is provided by compacted clay berms. A general detail of the liner system is shown as Figure 1.

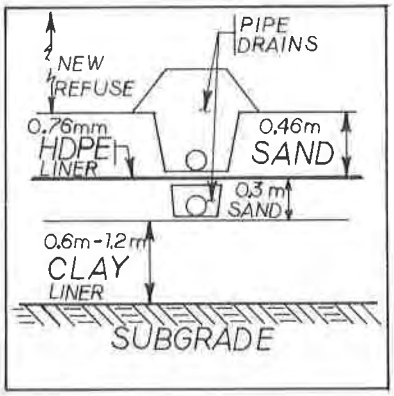

FIGURE 1: LINER SYSTEM DETAIL

QUALITY CONTROL PROGRAM

The scope of the quality control program implemented during the landfill expansion was developed to adequately monitor the various aspects and phases of construction such that the end product could be professionally certified as meeting all design requirements and regulatory criteria. To accomplish this, the required construction sequencing and technical specifications were prepared to clearly define the project workload and establish quality requirements. Enforcement of the written construction plan was effected through full-time field inspection of the construction operations, field testing to test all liner seams, materials and components as well as to measure densities and laboratory testing of clay and sand materials to measure gradation and permeability characteristics. The total program was necessary for proper certification by the design engineer, who worked closely with the field engineer and client during the performance of the construction. Of all aspects of this project, the maintenance of open channels of communication between the involved parties was paramount to the successful accomplishment of the established goals within a very short construction schedule.

A pre-construction laboratory testing program verified the acceptability of site borrow source sands and clay for use in the liner system construction. In addition, manufacturers were required to submit certifications for all other materials used in the liner system.

The natural sands and clay materials were tested for classification indices (ASTM D423/D424), moisture-density relationships (ASTM D698), grain size distribution (ASTM D422) and permeability (triaxial chamber). During construction, random samples of in-place materials were selected for laboratory testing to assess and verify continued compliance with the design criteria. For instance, samples of the in-place clay were obtained by either hand excavating undisturbed blocks or pushing Shelby tubes and then tested for physical and engineering properties. Sand samples were obtained by the grab method and gradations were checked. Responsibility for sampling was assigned to the quality control field engineer.

Subbase

As noted, more than 75 percent of the existing ground surface was underlain by 14 m to 27 m of municipal refuse placed by past separate site disposal operations. The area was subsequently used to stockpile slag processed at an adjacent plant for approximately 15 years. To form a base for these large stockpiles, the surface of the old landfill had been covered with slag of varying thickness.

Subsequent to removal of the slag stockpiles, the resultant surface was graded and proof-rolled. The proof-rolling was performed with a 10-ton vibratory roller to effect a "stable" surface for liner construction. All encountered soft areas were overexcavated and backfilled with compacted lifts of slag. The field engineer closely monitored the rolling and directed overexcavation requirements based on previous field investigations and design analysis. Final checks and verification of density was made through field density tests by the nuclear gauge method (ASTM D2922). This subbase preparation work evolved from the engineering analysis (bearing capacity, stability and settlement) of the underlying landfill and natural glacial outwash deposits.

Subgrade

The subgrade fill was placed on the prepared ground to the base of the secondary clay liner. The surface of the subgrade fill established the slopes and grades of the overlying liner components. In addition to the density testing and verification, the field engineer was responsible for the check and verification of construction survey control. Since this surface establishes the slopes and grades of all overlying liner components, accurate construction was critical to proper installation of the upper units to the design plan. No unusual material problems were experienced because the fill was unclassified sand obtained from the site borrow source.

Secondary Clay Liner

The clay liner was constructed on the completed subgrade surface. This liner continued to the perimeter berms which serve as the edge containment structures of the sloped liner system. Quality control required close coordination and continued evaluation of the borrow source excavation with the placement, grading and compaction earthwork.

Because of unexpected variations in the stratigraphy and lithology of the clay unit - known locally as the Raritan Clay Formation - a significant level of effort was needed to outline efficient yet effective clay borrow excavation procedures and sequences. The stringent permeability requirements dictated excavation selectivity to avoid inclusion of significant percentages of coarse-grained material. The field engineer was thoroughly knowledgeable of the borrow area subsurface stratigraphy and clay requirements based on his involvement in pre-construction boring and testing programs. This experience proved invaluable to the maintenance of efficient earthwork operations while also meeting the high quality requirements, all accomplished in close cooperation with the site supervisory personnel.

The clay placement, grading and compaction earthwork utilized standard equipment. Discing of the clay was necessary to break down the blocky structure resulting from the borrow excavation by a large hydraulic backhoe. Compaction was accomplished by sheepsfoot-type compactors. Final grades were constructed by means of a laser-guidance system installed by the owner on a standard motor grader. This setup greatly simplified the final grading operation and resulted in accurate grades conforming to the design plans. Figures 2 and 3 show the laser-controlled system.

FIGURE 2: GRADE CONTROL LASER UNIT

FIGURE 3: LASER-GUIDED MOTOR GRADER

Undisturbed samples of clay were taken during various stages of the secondary liner construction. Six (6) samples were tested for the initial 7 acre expansion area. The test results indicated in-situ permeabilities of the clay ranging from approximately 6 x 10^{-9} cm/sec to approximately 4 x 10^{-8} cm/sec, all less than the design maximum. Final in-place densities were checked and verified by the nuclear gauge method. Clay thickness was monitored by spot test pits but generally verified by field survey of the liner elevations before and after each stage of construction.

Leachate Detection System

Construction of this system included placement and grading of the 0.3 m (1.0 ft) sand zone, installation of the pipe drains and construction of pipe penetrations through the perimeter clay berms. Gradation limitations were established on the sand to mitigate puncture of the overlying synthetic liner and optimize permeabilities. To meet the design gradation, a screening plant was operated in the borrow area. Samples of the processed sand were periodically obtained and laboratory tested to determine grain size distribution. Results of the grain size tests were utilized as a secondary check on the permeability of the detection zone sand.

Drainage pipe installation quality control checks included: surveying of laterals and main collectors; grade checks of the trenches; visual inspection of geomembrane fabrics and installation details; inspection and sampling of aggregate; and, inspection and checking of all pipe and pipe jointing procedures.

Along the clay berm there were two pipe penetrations for each manhole group, one for the detection pipe and one for the leachate collection pipe. Each was fitted with a PVC anti-seep collar. The collar was securely fastened to the pipe by means of adhesive and band clamps and placed approximately 0.3 m (1.0 ft) into the inboard slope of the berm. The inset was excavated by hand and backfilled with clay to the original slope surface. This construction was systematically inspected by the field engineer to assure proper completion of each stage of installation. Figure 4 shows the placement of backfill around a completed collar. The white bead is sealant between the collar and pipe. All piping used in this landfill expansion from the berm penetrations outward was high density polyethylene (HDPE).

FIGURE 4: ANTI-SEEP COLLAR CONSTRUCTION

Primary Synthetic Liner

The HDPE primary synthetic liner was installed on the final-graded surface of the detection sand in 6.86 m (22.5 ft) wide sheets. Seaming of the sheets was accomplished through the use of hand-held extrusion welders which applied a fillet weld of HDPE over the seam and produced a fully-integrated homogeneous fusion weld. All seams were vacuum tested for integrity and were repaired and retested as required prior to final acceptance. The field engineer inspected the liner installation and vacuum testing work as it progressed. Upon completion of the work, the liner installation subcontractor issued a letter of certification to the owner on the liner installation, seaming and vacuum testing work. The owner required that the vacuum testing be performed as a final check and verification of the seam integrity. This verification documentation of liner integrity was then submitted to the regulatory agency for approval.

Geomembrane installation required a labor force of approximately ten (10) men. Each roll of fabric was placed along the perimeter by a wheeled loader and then unrolled and arranged by the laborers. Using two welding crews, the seaming averaged approximately 3,530 m^2 (38,000 ft^2) of HDPE liner per 8-hour shift. However, during one shift a peak installation of 5,110 m^2 (55,000 ft^2) was accomplished. The total area of liner installed for the first phase expansion was approximately 28,335 m^2 (305,000 ft^2).

Future areas to be lined as a continuation of this expansion exceed approximately 105,000 m^2 (1,000,000 ft^2). Figures 5 and 6 shows placement of the HDPE and keying the HDPE into the perimeter clay berm, respectively.

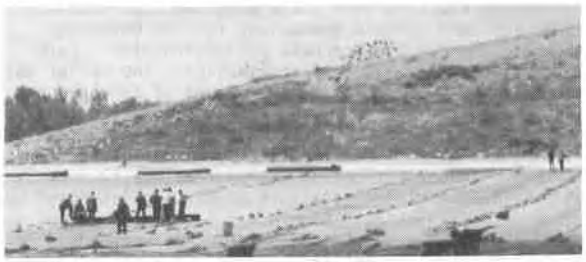

FIGURE 5: HDPE LINER PLACEMENT

FIGURE 6: ANCHORAGE OF HDPE INTO PERIMETER BERM

Protective Cover and Leachate Collection System

The final phase of the liner system construction involved placement of the protective sand cover over the surface of the completed HDPE liner and the subsequent incision of the leachate collection pipe drain network into the sand cover. Quality control requirements were basically the same as those for the detection system construction. For this component however, special care was exercised as the work was being performed above the HDPE liner.

As with the detection pipe drains, the pipe trench was excavated by a small rubber-tired backhoe. A flat strike plate was welded to the bucket teeth to avoid gouging of the teeth below the bottom of the trench. Survey control of the drain grades and elevations was rigorously maintained during construction. Probe holes were hand dug, where necessary, to spot check the location of the HDPE liner ahead of the backhoe excavation.

Figure 7 shows a section of the collection drain ready for the placement of coarse aggregate. The bottom and sides of the trench were lined with a geotextile fabric to prevent the migration of fines into the coarse aggregate. The aggregate was mounded approximately 0.3 m (1.0 ft) above the sand surface to provide a large flow area. Then, a bonnet of geotextile fabric approximately 0.61 m (2.0 ft) wide was placed over the top area of the aggregate. This was also designed to function as a barrier to the migration of refuse fines into the coarse aggregate. Figure 8 is a plan view of a section of the completed liner system showing collection drains and the final graded sand surface. Subsequent refuse placement was initiated directly on the shown sand surface. Figure 8 also shows the on-site leachate treatment plant in the background.

FIGURE 7: LEACHATE COLLECTION DRAIN PIPE

FIGURE 8: FINAL LEACHATE COLLECTION DRAINS AND COVER

Post-Construction Monitoring

The liner was ultimately field tested by sealing the leachate collection pipes with air plugs at their outlets to the manhole structures. This was done to impound all rainfall runoff within the limits of this liner expansion until refuse placement was initiated. Several very intense rainfalls were experienced and the water impoundment level rose to the perimeter berm crest elevation. In one area, topographic conditions outside of the landfill limits resulted in runoff actually being impounded to a level approximately 12.7 cm (5.0 in.) above the HDPE liner edge, which was keyed into the berm. At approximately this same time, a discharge flow was noted in the detection manhole of one of the independant drainage bays. All other detection manholes showed no flow from the system for their respective drainage bays.

The detection zone flow rates and the corresponding impoundment surface elevations were closely monitored from this point on. Water was pumped out of the impoundment to lower the water level below the crest elevation so the area could be accessed for further investigation. However, upon lowering the water surface below the crest level, the detection pipe flow rate dropped dramatically. Because of this change, it was immediately concluded that the key trench was the avenue of access for the water into the detection sand zone. To correct this apparent problem, the HDPE was uncovered along the berm, removed from the key trench and resealed with a clay and bentonite mixture. The fill was thoroughly compacted and encased with 0.9 m (3.0 ft) of additional clay above the key trench. This was then topsoiled and hydroseeded to provide protection against drying, cracking and erosion of the berm seal. During this period with the impounded water level maintained at a nearly constant elevation slightly below the crest, the detection discharge flow continued to decrease with time. Subsequent heavy rainfalls effected no response from the discharge flow of this single bay, and the rate continually regressed until a time when no measureable flow was noted.

Therefore it was concluded that the observed discharge had apparently entered the system via the key trench. Developed data on flow vs. time vs. impoundment level suggested that the problem had been satisfactorily corrected. The data and assessment were subsequently submitted to the regulatory agency and approval for waste disposal was granted within a week.

ACKNOWLEDGEMENTS

The authors wish to express their sincere gratitude to the project management staffs of Waste Management, Inc. and G.R.O.W.S., Inc., for their complete cooperation in the performance and completion of this very successful project. In particular, we thank Mr. Gary Brown, P.E., District Engineer, for his insight and expertise which helped effect the final product, and also for the opportunity to prepare this paper documenting the construction and quality control procedures.

Finally, the authors wish to acknowledge that the subject work was performed while previously employed by Orbital Engineering, Inc. of Pittsburgh, Pennsylvania.

LIST OF SYMBOLS

1. GENERAL SYMBOLS

1.1 Dimension symbols

Symbols used for the dimensions are:

L:	length
M:	mass
T:	time
–:	dimensionless

1.2 Unit Symbols

m	meter
m^2	square meter
m^3	cubic meter
mm	millimeter
μm	micron
g	gram
mg	milligram
kg	kilogram
s	second
N	newton
kN	kilonewton
Pa	pascal
kPa	kilopascal
MPa	megapascal
J	joule
tex	tex
L	liter
°	degree
%	percent
–	pure number

The following relationships exist:

1 Pa = 1 N/m^2
1 tex = 1 mg/m
1 J = 1 mN

1.3 Geometry and kinematics

L	L	(m)	length
B	L	(m)	breadth
H	L	(m)	height, thickness
D	L	(m)	depth
z	L	(m)	vertical coordinate
d	L	(m)	diameter
A	L^2	(m^2)	area
V	L^3	(m^3)	volume
t	T	(s)	time
v	$L\,T^{-1}$	(m/s)	velocity
g	$L\,T^{-2}$	(m/s^2)	acceleration due to gravity ($g = 9.81\ m/s^2$)

1.4 Hydraulic parameters and properties of FLUIDS

h	L	(m)	hydraulic head or potential sum of pressure height (u/γ_w) and geometrical height (z) above a given reference level
q	L^3T^{-1}	(m^3/s)	rate of discharge volume of water seeping through a given area per unit of time
v	$L\,T^{-1}$	(m/s)	discharge velocity rate of discharge per total unit area perpendicular to direction of flow
i	–	(–)	hydraulic gradient loss of hydraulic head per unit length in direction of flow
j	$ML^{-2}T^{-2}$	(kN/m^3)	seepage force per unit volume force per unit volume of a porous medium generated by action of the seeping fluid upon the solid elements of the porous medium ($j = i\gamma_w$)
ρ_w	ML^{-3}	(kg/m^3)	density of water
γ_w	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of water
η_w	$ML^{-1}T^{-1}$	(kg/ms)	dynamic viscosity of water

NOTE: Instead of w, use any other appropriate subscript of other fluids (e.g.: η_a, dynamic viscosity of air)

2. Hydraulic properties of a net, geogrid, or geotextile associated with a geomembrane

k_n	LT^{-1}	(m/s)	coefficient of normal permeability of a geotextile
k_p	LT^{-1}	(m/s)	coefficient of permeability in the plane of a geotextile
ψ	T^{-1}	(s^{-1})	permittivity of a geotextile ($\psi = K_n/H_g$)
θ	L^2T^{-1}	(m^2/s)	transmissivity of a geotextile ($\theta = k_p/H_g$)

3. Physical/mechanical properties of a geomembrane

μ		(kg/m^2;g/m^2)	mass per unit area
α_ε	MT^{-2}	(kN/m)	force per unit width of the geomembrane
α_f	MT^{-2}	(kN/m)	force per unit width of the geomembrane at failure
α_y		(kN/m)	force per unit width at yield
ε		(%)	strain (also called elongation)
ε_f		(%)	strain at failure (elongation at failure)
ε_y		(%)	strain at yield (elongation at yield)
J	MT^{-2}	(kN/m)	"modulus" of the geomembrane

NOTE: The "modulus" of a geomembrane is defined as a force per unit width while modulus is usually defined as a force per unit area.

J_i	MT^{-2}	(kN/m)	initial (tangent) "modulus" of the geomembrane
$J_{t\varepsilon}$	MT^{-2}	(kN/m)	tangent "modulus" of the geomembrane at elongation (e.g. J_{t30} is the tangent "modulus" of the geomembrane at 30% elongation)
$J_{sec\ \varepsilon}$	MT^{-2}	(kN/M)	secant "modulus" of the geomembrane between 0 and elongation (e.g. J_{sec30} is the secant "modulus" of geomembrane between 0 and 30% elongation)
F_G	MLT^{-2}	(N,kN)	breaking force of geomembrane as measured in grab test
F_p	MLT^{-2}	(N,kN)	breaking force of geomembrane in a puncture test (to be defined)
F_T	MLT^{-2}	(N,kN)	breaking force of geomembrane in a tear test (to be defined)
P_B	$ML^{-1}T^{-2}$	(kPa,MPa)	bursting pressure of a geomembrane

International Conference on Geomembranes

Definition of Terms

Geomembrane:

Synthetic membranes, polymeric membranes, flexible membrane liners, plastic liners, and impervious sheets are a few examples of the many names given to these relatively new materials. Although many users of these materials often prefer to use trade names, this practice is deemed inappropriate because it creates considerable confusion.

Geomembrane is the generic term proposed to identify these liner and barrier materials. Geomembranes are impermeable membrane liners and barriers used in civil engineering for geotechnical projects. They can be either sprayed on a surface or prefabricated and transported to the construction site. Sprayed-on geomembranes are composed predominantly of asphalt. They are either sprayed directly on a surface (earth, concrete, etc.) or onto a geotextile. Prefabricated geomembranes are usually composed of synthetic polymers, elastomers (rubbers), or plastomers (plastics); some are reinforced with a fabric. There are also prefabricated asphaltic geomembranes.

Classification of Geomembranes[1]

Geomembranes can be classified according to production process and reinforcement:

1. **Made in situ, non-reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material directly onto the surface to be lined (earth, concrete, etc.). The non-reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) non-reinforced geomembranes." Typical materials used are based on asphalt, asphalt-elastomer compound, or polymers such as polyurethane. Due to the spray application, the final thickness of such geomembranes is not easy to control and may vary significantly from one location to another. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

2. **Made in situ, reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material onto a fabric. The reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) reinforced geomembranes." Typical materials used are the same as for the made in situ non-reinforced geomembranes described above. Typical fabrics used are the needle-punched nonwoven geotextiles because they can absorb viscous materials. As discussed above, the final thickness of such geomembranes is not easy to control. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

3. **Manufactured, non-reinforced geomembranes** are made in a plant by extrusion or calendering of a polymeric compound, without any fabric reinforcement, or by spreadying a polymer on a sheet of paper removed at the end of the manufacturing process. Typical thicknesses range from 0.25 to 4 mm (10 to 160 mils) for geomembranes made by extrusion and 0.25 to 2 mm (10 to 80 mils) for geomembranes made by calendering. Typical roll width for geomembranes made by extrusion is 5 to 10 m (16 to 33 ft), although some are narrower. Typical roll width for geomembranes made by calendering is 1.5 m (5 ft), with some manufacturers producing 1.8 to 2.4 m (6 to 8 ft) wide rolls.

4. **Manufactured, reinforced geomembranes** are made in a plant, usually by spread coating or calendering. In spread-coated geomembranes, the reinforcing fabric (woven or nonwoven) is impregnated and coated on one or both sides with the compound, either polymeric or asphaltic. In calendered reinforced geomembranes, the reinforcing fabric is usually a scrim. Calendered geomembranes are always made with polymeric compounds and are usually made up of three plies: compound/scrim/compound. Sometimes they are made of five plies: compound/scrim/compound/scrim/compound. Geomembranes with additional plies can be made on a custom basis. Typical thicknesses of asphaltic spread-coated geomembranes are 3 to 10 mm (⅛ to ⅜ inch). Typical thicknesses for polymeric spread-coated and three-ply calendered geomembranes are 0.75 to 1.5 mm (30 to 60 mils). Typical thicknesses for five-ply calendered geomembranes are 1 to 1.5 mm (40 to 60 mils).

5. **Manufactured, reinforced geomembranes laminated with a fabric** are made by calendering a manufactured geomembrane (usually a non-reinforced

geomembrane previously made by calendering or extrusion) with a fabric (usually a nonwoven) which remains apparent on one face of the final product.

Classification of Geomembrane Polymers (National Sanitation Foundation):

1. *Thermoplastics:* Polyvinyl Chloride (PVC); Oil Resistant PVC (PVC-OR); Thermoplastic Nitrile-PVC (TN-PVC); Ethylene Interpolymer Alloy (EIA);

2. *Cristalline Thermoplastics:* Low Density Polyethylene (LDPE); High Density Polyethylene (HDPE); High Density Polyethylene-Alloy (HDPE-A); Polypropylene; Elasticized Polyolefin;

3. *Thermoplastic Elastomers:* Chlorinated Polyethylen (CPE); Chlorinated Polyethylene-Alloy (CPE-A); Chlorosulfonated Polyethylene (CSPE), also commonly referred to as ''Hypalon;'' Thermoplastic Ethylene-Propylene Diene Monomer (T-EPDM);

4. *Elastomers:* Isoprene—Isobutylene Rubber (IIR), also commonly referred to as Butyl Rubber; Ethylene-Propylene Diene Monomer (EPDM); Polychloroprene (CR), also commonly referred to as ''Neoprene;'' Epichlorohydrin Rubber (CO).

[1]Giroud, J. P. and Frobel, R. K. ''Geomembrane Products'' *Geotechnical Fabrics Report*, Vol I, number 2 (1983).

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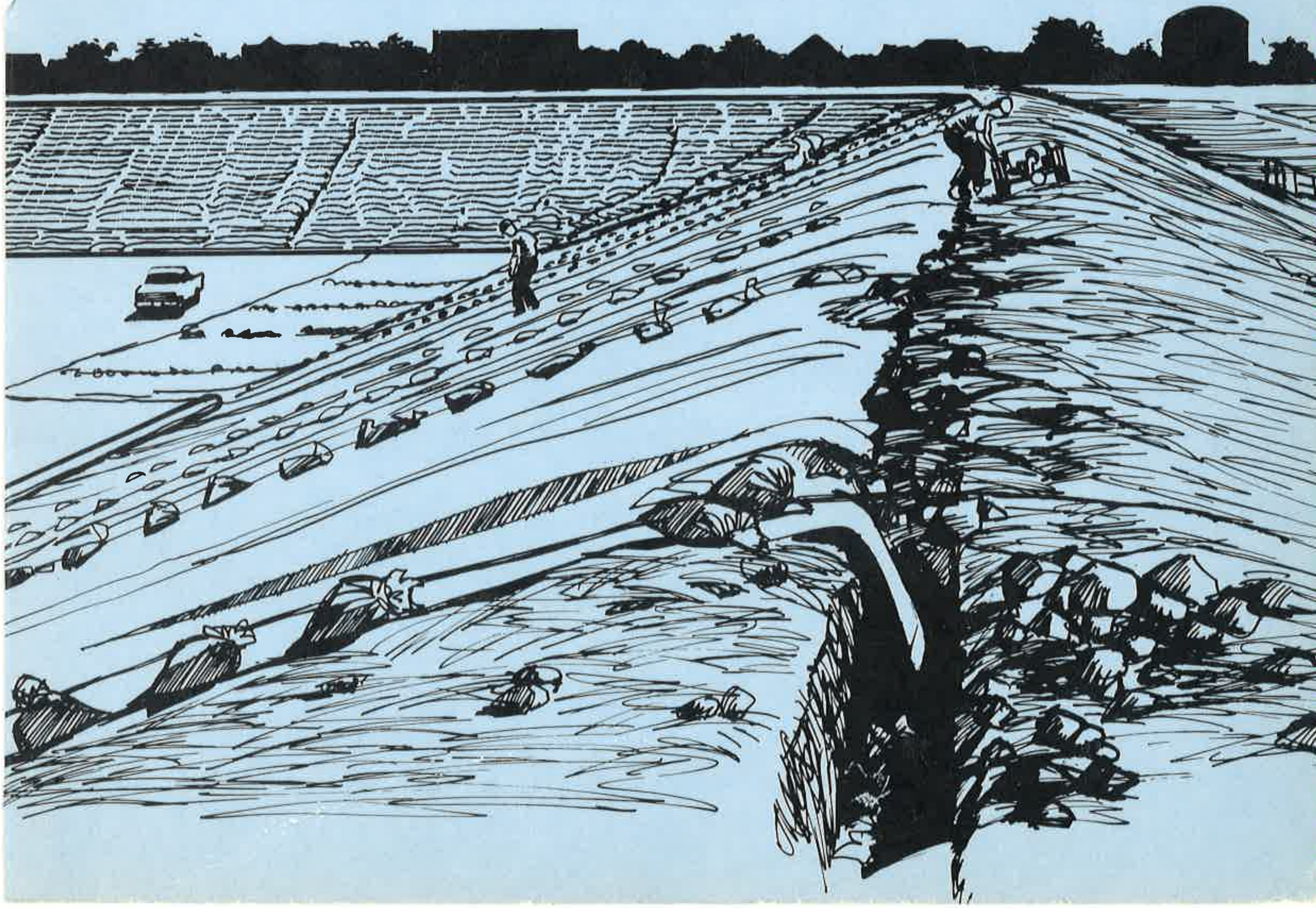

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L. Kamp

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Volume II

STRONG, ALAN G.
Polysar Limited, Sarnia, Ontario, Canada

Longevity Aspects of Polymeric Linings For Water Containment

Longevity, which is still an imprecise characteristic, can be adversely influenced by the incidence of stresses. The nature and effect of these are examined. The ability of a membrane to bridge a gap created by ground movement with minimum property loss is a major contribution towards the expected longevity. The behavior of membranes under such conditions is studied; the value of thickness and the role of textiles is emphasized. The importance of high strength and elongation is inferred and tests more specific to membrane service are discussed. These include the effect of stress measured at site temperature after aging in the unstressed and strained states.

1.0 INTRODUCTION

Geomembranes are polymeric materials which have to serve in direct contact with the ground. While the performance of such membranes has generally been good, inspection and examination of many installations after a few years service have indicated some localized faults, even with optimized materials and design. It is evident that membranes can resist the elements even when exposed. However, the stresses which are developed in the material to accommodate the conditions of localized ground movement can lead to premature failure. Quantifying the extent of movement for studying membrane behavior is necessarily subjective. In a study program, parameters were selected corresponding to a severe condition reported in the Middle East.

2.0 THE NATURE OF MEMBRANE DETERIORATION AND FAILURES IN SERVICE

Improved membrane compositions and reservoir designs have reduced the incidence of strain affecting large areas created by the weight of the membrane on the slope and by wind up-lift. It is possible, therefore, to make a general distinction between longevity of the unstressed portions of membranes, and those portions affected by ground movement. The possibility of such movement cannot be dismissed in foundationless structures.

2.1 Deterioration in the Absence of Stress

2.1.1. Natural Exposure. In general, the relative tendencies for surface discoloration, hardening, embrittlement, small cracks, and crazing can be reproduced at outdoor testing stations. On actual sites, this trend is often accelerated by more severe exposure conditions such as unremitting heat and the absence of wind. These factors contribute to surface temperatures greater than 80°C for long periods.

The property loss resulting from radiation and thermal oxidative degradation is a measure of membrane longevity (1) (2). A lower level of properties can result when stress is applied under temperature conditions which can weaken or affect the membranes (3). This may not be detected if aged materials are only tested at room temperature. As aging proceeds, the margin of safety to accommodate combined strain and temperature effects diminishes, affecting the longevity. On a practical basis, the use of thicker membranes, particularly laminates and reinforced construction, has reduced temperature sensitivity.

2.1.2. Laboratory Simulated Natural Exposure. The limitations of standard simulated weathering tests in reproducing exposed conditions are well recognized. It has been shown (4) that simple heat aging can reproduce the general changes in polymer properties which result from natural exposure. This is particularly applicable to black rubber membranes. The carbon black generates considerable heat by absorbing the infrared component of the solar radiation. The black also converts much of the uv component into heat, although the residue can cause surface degradation, which penetrates deeper with time. Therefore, at the outset the property loss is attributed primarily to heat degradation, but the effect of radiation degradation becomes more significant with exposure.

This effect can be followed by increasing the infrared component in a Xenotest 150 in order to attain surface temperatures of 80°C. After 250 h, it was shown (3) that measurable degradation could occur in some membrane materials, reproducing the surface features in service. Moreover, for the same exposure, the level of degradation measured with black membranes was similar to that of heat aging at 80°C. This confirmed the role of heat in the radiation process.

2.1.3. Exposure Under Burial or Immersion Conditions. The various effects that cause deterioration are the chemical effects of soil and water immersion; a reduced level of properties for buried membranes was attributed to the effect of warm air beneath the membrane (1). Burial or water immersion effectively shields the membrane from radiation effects.

2.2 Deterioration and Failure Resulting from Stress

Physical failures which can occur in geomembranes have been categorized comprehensively (5). In this study, they are considered both with respect to stresses inherent in the membrane itself and to those that arise specifically from their role as membranes, in particular, the effect of differential settlement and hydrostatic pressure.

2.2.1. Inherent Stresses. Built-in stresses can arise during manufacture and become evident as shrinkage when the membrane is unrolled. These stresses can aggravate the effects of thermal expansions and contractions. This is further affected by the presence of volatiles and other ingredients, which evaporate slowly. These features can be evaluated by the Dimensional Stability Test: ASTM D 1240. The suggested values for the maximum percentage change measured in each direction for unsupported plastic membranes after 15 min at 100°C varies from 2-15%. The comparable value for vulcanized membranes is 2%, but after a heating period of 7 days at 100°C (5). This is a more severe test and would detect any volatility effects.

On small exposed test samples, shrinking is indiscernible. However, under site conditions in which the membrane is attached on all sides, the effect is cumulative and can cause failures at the structurally weaker points.

2.2.2. Strain Effects Resulting From Geomembrane Applications and Service. Stretching a membrane during laying can lead to shrinkage effects similar to those noted above. Laying the membrane slack is an attempt to offset possible shrinkage resulting from some strain during application, inherent strain of the material, and volatile loss. The susceptibility of the early butyl compositions to ozone attack under strain revealed areas of strain in membrane installations; this led to material blends with EPDM as well as design improvements. Stones projecting under the membrane were a source of strain; this was resolved by using a soft substrate layer. The membrane which lined a deep installation with steep banks was obviously strained and resulted in membrane thinning, adhesive tape joints opening, splits resulting from surface defects, and ozone attack close to joints.

Here, the joint welding operation had caused some property loss. Peeling of adhesive joints after long aging periods at areas restricted to the water line signified cyclic thermal movement created by the differential membrane temperatures above and below the water line.

Membrane stresses are significantly increased under the action of hydrostatic pressure. This can be demonstrated practically by the difficulty of pulling out a fold in the membrane at the waterline. Buried linings, when exhumed, have the appearance of three dimensional maps of mountainous terrain and, may be punctured. This signifies the distortion caused by hydrostatic pressure and the relative movement of stones, etc., caused by differential ground settlement.

While the extent of ground movement can be mitigated by constructional techniques, the prime advantage of flexible membranes is their ability to accommodate some inevitable ground movement without leakage. Highly extensible membranes are regarded as more suitable than those with lesser extensibility (6). This feature is particularly important where the membrane is attached to rigid points including pipes, concrete overflows, etc.

Other sources of stress can result from wind up-lift or wave action. Minor earthslides beneath the membrane can expose stones and create stresses. Hail and traffic are other sources of applied stress which can be mechanically damaging.

3.0. THE INFLUENCE OF STRAIN ON MEMBRANE LONGEVITY

High levels of strain can result in immediate failure due to puncture and tearing. Lower levels of stress can cause thinning, yielding, and creep, as well as accelerated aging, and therefore be significant in reducing longevity.

The behavior of membranes under strain has not been widely studied. Early work using heat aging (2) and a weather cabinet (3) indicates that rubbers are hardly affected, whereas a PVC membrane was adversely affected by strain, particularly at low temperatures. In such a program, both the relevance of the value of strain selected and the behavior of the membrane under actual site conditions are crucial factors.

4.0. EFFECTS OF GROUND MOVEMENT

Koike (7) examined the behavior of a roofing membrane bonded to a substrate in which a crack developed. This corresponds approximately with a membrane application in which the hydrostatic pressure holds a membrane in contact with the substrate in which a fissure is formed. Koike found that the extent to which the membrane was capable of bridging the crack was influenced by the thickness and extensibility of the adhesive when these values were constant, and by the product of the membrane strength and elongation (i.e., the strain energy). The elongation accommodates movement and strength to reduce thinning and allows the strain to be distributed over an increasingly large area on both sides of the crack.

In an analogous way, a study of the comportment was made under practical conditions. This had particular relevance for an installation in the Middle East. Unstable gypseous ground was found to form fissures with horizontal and vertical displacements of up to 20 mm. Should the membrane fail to accommodate this movement, leaking water could dissolve the substrate.

An engineering specification (8) was developed to meet this requirement using a 0.76 mm rubber membrane placed loosely between two nonwoven textiles layers. On the opening of a substrate fissure, it was proposed that the textile, having a high strength and a modest extension, could accomodate some limited movement and distribute the stress more evenly over the membrane sandwich. Under excess movement, the textile would break and the membrane would then provide considerable additional extension, thereby preventing leakage.

4.1. Equipment for Measuring Strain Distribution

A pressure vessel (Figures 1 and 2) was designed and constructed to simulate the behavior of a membrane bridging a fissure in service and to measure the developed strain distribution.

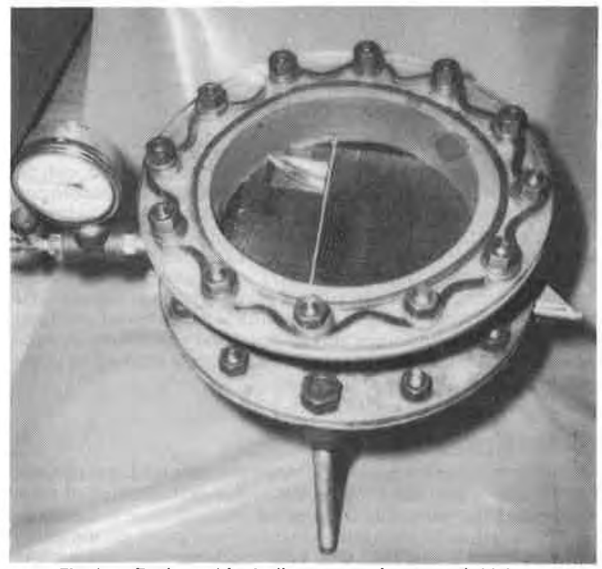

Fig. 1. — Equipment for testing geomembrane gap bridging properties under hydrostatic pressure.

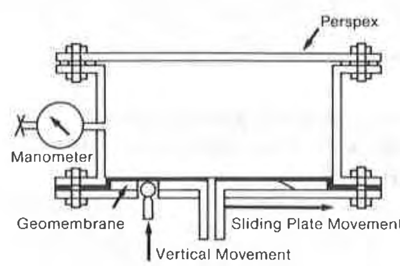

Fig. 2. — Equipment for simulating ground movement beneath a submerged geomembrane and observing the resulting strain distribution.

The lower base plate represented the ground surface, and a special sliding device operated by a mechanically driven screw permitted a shaped slot to open rapidly, creating a fissure. The base plate contained a further sliding panel through which a 20 mm ball bearing could be pushed. This represented the action of a stone, which could work its way through to the surface, or be left protruding when surrounding areas subsided. A glass prism placed on the geomembrane adjacent to the projection enabled the strain at the side elevation to be photographed from above.

Substrate surfaces were represented by a sheet of polyethylene with a low friction surface, a sheet of fine sandpaper, and a plastic sheet to which was bonded coarse gravel. The gravel pieces were isolated from each other and projected to a maximum height of 10 mm. These test surfaces were attached to the base plate.By controlling the hydrostatic pressure, water depth corresponding to 5 and 20 m could be simulated. The movement of the membrane could be observed through the upper transparent plate and measured by photography. This was facilitated by superimposing a grid of spots, and later 6 mm lines, on the membrane by a photographic technique. Under pressure, the membrane "ballooned" into the gap, and the unit extensions of the convex membrane surface were measured from beneath.

It was recognized that the edge effect at both ends of the slot, as seen in Figure 1, and the precision of the measurements influenced the results. The intention was to assess the general scale of distortion arising and to test the efficacy of the membrane/textile systems rather than to undertake a detailed strain analysis.

The strain distribution over the gap was expressed by plotting the extension (%) of 6 mm intervals marked on the membrane with reference to their initial positions. It was noted that since only one side of the gap opened, the distribution curves produced were not symmetrical.

4.2. Tests and Results

4.2.1. Effect of Thickness and Gap Size. Using a butyl/EPDM membrane in a range of thicknesses, the effect of increasing gap size was first investigated.

With a 20 mm joint gap, the level of strain increased fairly regularly as the thickness decreased (Figure 3). The distribution of stress was slightly narrowed, indicating a corresponding reduced strength capacity for pulling the adjacent membrane into the gap.

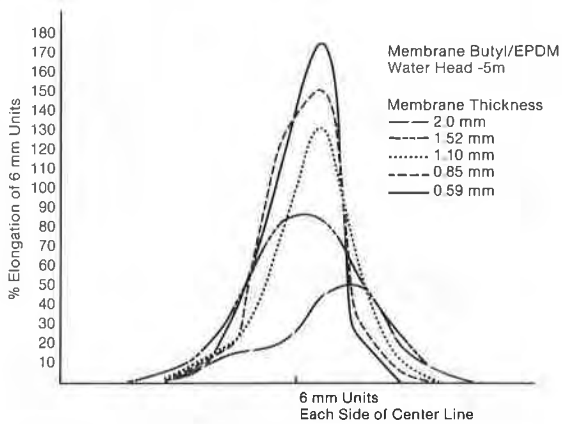

Fig. 3. — Effect of membrane thickness on strain distribution over a 20 mm joint.

In Figures 4 and 5, 1.10 and 2.0 mm thick membranes were used. The gap was opened 5, 10, 15, and 20 mm and the distribution strain was measured and featured. The pressure was increased to the equivalent of a 20 m depth of water for a final reading.

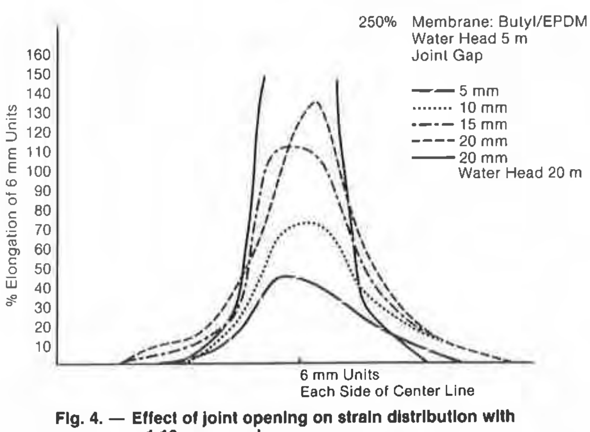

Fig. 4. — Effect of joint opening on strain distribution with a 1.10 mm membrane.

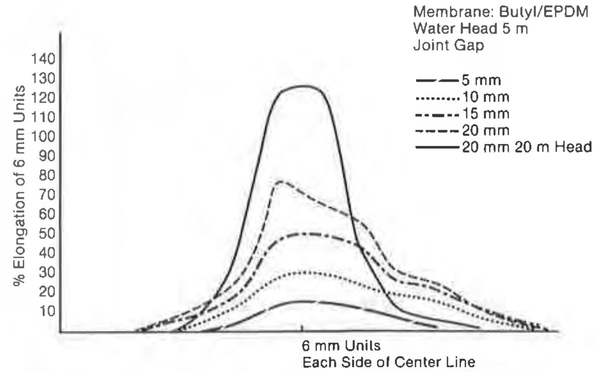

Fig. 5. — Effect of joint opening on strain distribution with a 2.00 mm membrane.

The results show that as the gap size increases, the strain level and distribution increases for all thicknesses. Interestingly, when the pressure is increased to a 20 m depth, the strain is significantly increased and the extent of strain distribution is narrowed. This is attributed to the membrane relaxing prior to application of additional pressure. This incremental pressure prevented more rubber from being pulled into the gap, although at the gap center, the extensibility increased. In this respect, the increase in water pressure gave an effect comparable to a membrane of reduced thickness and therefore, reduced modulus and strength. Under all pressure conditions, the strain only extends 30 or 40 mm on either side of the crack.

The percentage strain in the central unit over the gap is shown (Figure 6) to be fairly linear with reduced differences between the 0.58 mm and 1.10 mm thick membranes. This effect was also seen under high pressure. Extension values under the worst conditions are between 150 and 225%, well within the capacity of most unreinforced membranes. The advantages of thicker membranes in terms of strain level and stress distribution are apparent.

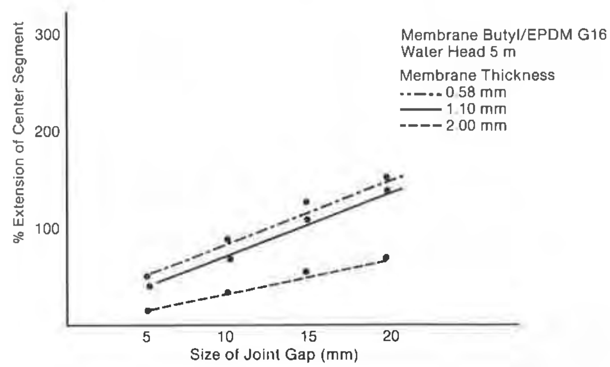

Fig. 6. — Effect of joint opening and membrane thickness on % extension of membrane over the joint.

4.2.2. Effect of The Nature of The Ground Surface and Different Membrane Construction. The polyethylene ground surface was replaced firstly by sandpaper, then gravel, and finally a textile inserted between the gravel and the membrane.

As shown in Figure 7, the maximum extension noted with the polyethylene substrate was reduced when sandpaper was used. Most signficantly, the geomembrane behaved similarly over the sandpaper and gravel surfaces. This supports the view that in the case of extensible membranes, the hydrostatic pressure localized the membrane strain at and around the joint gap, almost regardless of the surface nature of the substrate. The presence of the textile below the membrane significantly reduced the extension of the membrane in the region of the gap.

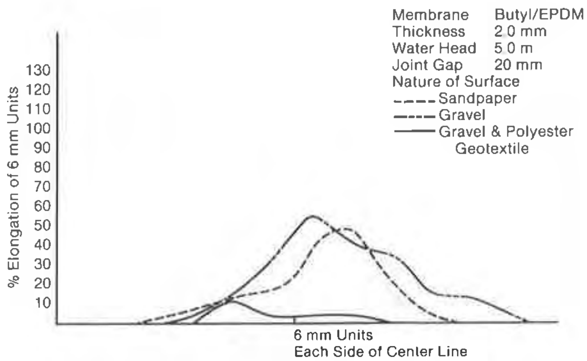

Fig. 7. — Effect of the nature of the surface on strain distribution over a joint opening.

4.2.3. Distortion over surface irregularities. On commencing the test, the membrane featured in the first profile (Figure 8) rested initially on the ball. As the pressure increased, the membrane was forced around the sides causing a more even distribution and a maximum strain value of 40%. When the ball was pushed up after pressurization, as seen in the second profile, a greater strain occurred at the crown. Since the membrane at the base was held firmly in position, the stress values around the sides of the ball were greater, the highest values of 68% being located below the midway point.

In the third profile, a textile was used beneath the membrane. As a result of the upward movement of the ball against the water pressure, the membrane was carried by the textile and was able to resist the encapsulation effect of the hydrostatic pressure. The presence of the textile reduced localized extension from 68 to 8%.

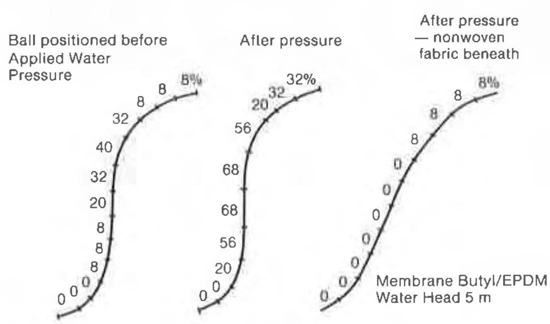

Fig. 8. — Strain distribution around a projecting ball beneath membrane in pressure vessel.

4.3. Discussion

4.3.1. Geomembrane Behavior. In tensile testing, the membrane strain is distributed evenly between the grips until the yield, or break point, occurs. For exposed membranes on site, the surface friction and irregularities can restrict such an even strain distribution. Below the water line, and most likely when buried, it is seen that the extension will be localized around the point of movement. Thus, the concept that the effects of shrinkage can be minimized by laying a slack membranes is invalid. This also applies to the practice of laying membrane folds along the base of a reservoir slope in order to accommodate movement which could subsequently occur.

In practice, a clean cut fissure rarely occurs, unless a concrete foundation cracks. Normally, the edges would crumble irregularly still supporting the membrane. The hydrostatic pressure acts as an adhesive and restricts the movement to the crack area. Placing a loose geotextile beneath the geomembrane serves to distribute the stress and reduce the thinning effect. The strength of the textile and extensibility of the membrane contribute to maximizing the strain energy of the waterproofing system. Unfortunately, the equipment was unable to test the system to the point of textile rupture.

For membranes exposed above the water line, the strains would not be so localized and therefore reduced, although the aging conditions and effects could be more severe. The critical areas for both exposed and buried membranes are the points of fixation to rigid structures.

4.3.2. Comparative Behavior of Various Geomembranes. As noted, the geometry of membrane distortion in practice is different to laboratory tensile extension. The movement is often restricted and localized extension increased. At points of restriction, multi-dimensional stresses are more likely to cause yielding, thinning, and tearing than rupture. Tensile testing may only give a guide to the behavior of unsupported membranes and in Figure 9, the behavior of commercial materials at thicknesses commonly used is seen to be very different.

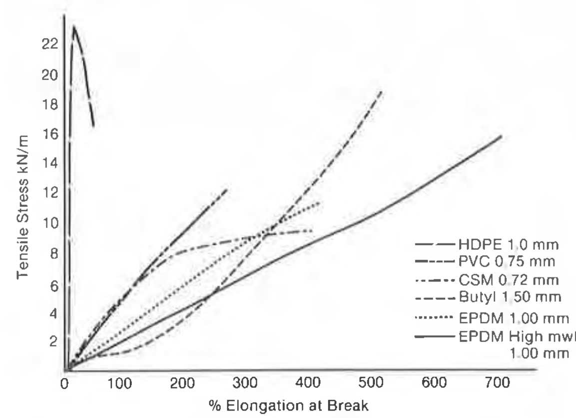

Fig. 9. — Stress strain properties of unsupported geomembranes tested unaged at room temperature to break or yield point.

While the strength is related to the thickness, especially for thicker membranes, the elongation is a more fundamental property of the material. The use of a textile reinforcement bonded to the membrane material can increase the strength and reduce the temperature sensitivity. However, in restricting the extension to around 20%, the membrane relies upon its strength to resist the magnitude of ground forces with only a limited contribution from its extensibility to accommodate movement. The associated role of a loose textile is seen to endorse the value of having both high strength and high elongation, implicit in the strain energy concept, as key parameters.

5.0. CONCLUSIONS

The relative inertness of membranes, the practical experience gained, and the difficulties of evaluation have tended to diminish interest in longevity. However, it is still an important parameter since a high level of retained properties are required during service to accommodate the effects of strain encountered.

On examining the behavior of membranes under hydrostatic pressure following the opening of a fissure, it has been shown that high localized extensions could be minimized by thicker membranes, and particularly by the combination of a membrane and textile. This favors high strength and elongation membranes. The relevance of the markedly different stress strain behavior of commercial membranes could not be evaluated in the pressure test equipment.

Improvements are constantly being introduced into new installations as a result of site experience. This process can be expedited by a better understanding of polymeric and textile performance under conditions specific to geomembranes, not comprehensively addressed by current specifications. This includes the long term resistance to the appropriate elements involved - heat, oxidation, solar radiation, water, and soil burial - measured particularly over the range of temperatures encountered on site.

The sensitivity of materials/systems to strain induced by shrinkage and ground movement both in terms of rupture or reduced performance under the above mentioned conditions are equally critical. Improved tests for simulating environmental conditions are emerging filling an important need. A criterion for assessing the validity of a specific test is its applicability to the wide range of membrane material currently used.

ACKNOWLEDGEMENTS

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Factors in the Durability of Polymeric Membrane Liners

Durability in a wide range of environmental conditions is essential to the use of polymeric membranes as barriers for the lining of waste storage, treatment, and disposal facilities. Low permeability, the principal function of a barrier, must be maintained over long exposure. As an aid in the process to select membranes and design impoundments, it is important to know the durability characteristics of various membranes. This paper discusses factors contributing to the durability of polymeric membranes under different exposure conditions. Inherent ability of a membrane to resist aggressive agents is determined by the polymer, the compound, the construction, and the manufacture. Environmental factors that are encountered by liners in different services are reviewed. Data are presented on the interaction of some commercial liners with a selection of solvents and wastes. They indicate the importance of the characteristics of the liquids and potential adverse effects of minor amounts of organics in waste streams.

INTRODUCTION

Polymeric membranes used to line hazardous waste storage and disposal facilities must be durable and maintain their integrity and performance characteristics over the designed life of the specific facility. Since the principal function of the membrane is to prevent leakage and migration of the wastes and their constituents, low permeability to the contained materials must be maintained. Also, resistance to physical damage of membranes and the integrity of the seams must be maintained so that breaks, tears, and other holes in the liner system do not develop.

Ultimately, the service life of a given liner will depend on the intrinsic durability of the material and on the conditions under which it is exposed during service. Differences in polymer compositions and construction will cause membranes to vary in their response to the exposure conditions which, even within a given facility, can differ greatly. Durability is also important during installation so that an effective barrier to waste migration can be achieved.

This paper discusses important factors that contribute to the durability and service life of polymeric membranes under different exposure conditions. These factors are divided between compositional and environmental factors.

The results and discussion in this paper are based on polymer science literature (2, 7), controlled testing and laboratory research on commercial membrane liners, and limited field experience in the testing of such liners that have been in service.

TYPE AND COMPOSITION OF POLYMERIC MEMBRANES

Polymers used in the manufacture of lining materials include rubbers and plastics differing in basic chemical composition and structure, solubility characteristics, chemical resistance, and the mode in which they deteriorate and ultimately fail. These polymers can be classified into four general types:

- Rubbers (elastomers) that are generally crosslinked.
- Thermoplastic elastomers that are not crosslinked.
- Plastics that are generally not crosslinked.
- Plastics that are thermoplastic and have a relatively high crystalline content, such as the polyethylenes.

Table 1 lists the types of polymers that are used in membranes. The polymers most frequently used in liners have been PVC, CSPE, CPE, IIR, EPDM, CR, and HDPE. The thickness of commercial polymeric membranes for liners ranges from 0.5 to 3.0 mm, with most in the 0.75 to 1.5 mm range.

Table 1. POLYMERIC MATERIALS USED IN LINERS

Polymer	Abbreviation	Type of compound[a]	Thickness, mm	Fabric reinforced
Butyl rubber	IIR	XL	0.9-2.3	Yes, no
Chlorinated polyethylene	CPE	TP	0.5-1.0	Yes, no
Chlorosulfonated polyethylene	CSPE	TP,XL	0.6-1.0	Yes, no
Elasticized polyolefin	ELPO	CX	0.6	No
Elasticized polyvinyl chloride	PVC-E	TP	0.8-0.9	Yes
Epichlorohydrin rubber	ECO	XL	1.67	Yes, no
Ethylene propylene rubber	EPDM	XL,TP	0.5-1.6	Yes, no
Neoprene	CR	XL	0.5-1.8	Yes, no
Nitrile rubber	NBR	XL	0.8	Yes
Polyethylene, low-density	LDPE	CX	0.3-0.8	No
high-density	HDPE	CX	0.5-3.1	No
Polyvinyl choride	PVC	TP	0.3-0.9	Yes, no

[a]TP = Thermoplastic; CX = Partially crystalline; XL = Crosslinked.

Most polymeric membranes are based on single polymers, but blends of two or more polymers are being developed and used in liners. Also, different grades of a given type of polymer can be used. Generic classifications based on individual polymers have become increasingly difficult even though one polymer may predominate. All polymers are compounded with auxiliary ingredients which serve different purposes.

The basic compositions of the different types of compounds are shown in Table 2. The crosslinked compositions are usually the most complex because they contain a crosslinking system. Thermoplastics, except for CSPE compounds, contain no curatives. Although thermoplastic as supplied, CSPE liners contain crosslinking agents that allow the polymer to crosslink during service. Crystalline materials have the simplest composition and generally consist of the polymer, a small amount of carbon black for ultraviolet protection, and antidegradants.

TABLE 2. BASIC COMPOSITIONS OF POLYMERIC MEMBRANE LINER COMPOUNDS

Component	Composition of compound type, parts by weight		
	Cross-linked	Thermo-plastic	Crystal-line
Polymer or alloy	100	100	100
Oil or plasticizer	5-40	5-55	0-10
Fillers:			
Carbon black	5-40	5-40	2-5
Inorganics	5-40	5-40	...
Antidegradants	1-2	1-2	1
Crosslinking system:			
Inorganic system	5-9	a	...
Sulfur system	5-9	...	...

[a]An inorganic curing system that crosslinks over time is incorporated in CSPE liner compounds.

Several of the auxiliary components of a formulation can be affected during service when they are either immersed in the liquid or exposed to the weather. Low molecular weight fractions in the original polymer or blend can be lost. The oils and plasticizers are potentially extractable and, in some cases, biodegradable; antidegradants can be extracted. Loss or change in any of these components can affect properties and durability of the compound.

Most of the polymeric membrane liners currently manufactured are thermoplastic. Even if the polymer in the crosslinked form is more chemically resistant (such as CPE and CSPE), it is generally supplied as a thermoplastic because it is easier to obtain reliable seams and to make repairs in the field. Thermoplastic polymers can be heat-sealed or seamed with a solvent or bodied solvent. Crystalline sheetings are generally seamed by thermal welding or fusion methods.

Membranes of all but the crystalline type compositions are available with fabric reinforcement to increase strength and thermal stability. The fabric constructions vary from end counts of 6 x 6 to more than 20 x 20. As the end count increases, the area between the threads that allows contact between the plies is reduced. The adhesion between plies is dependent upon this area and good "knitting" of the polymeric layers during manufacture. Good initial ply adhesion and its retention during service are important to prevent delamination.

Information on individual polymers and liners is presented in the EPA Technical Resource Document on liners (5).

FACTORS IN THE DURABILITY OF POLYMERIC MEMBRANES

The intrinsic durability of a membrane depends upon the polymer, the auxiliary compounding ingredients, and the construction and manufacture of the membrane. The durability of polymers can vary greatly with respect to different exposures. The deterioration of polymeric compositions is manifest in one or more of the following ways:

- Softening and loss of physical properties due to polymer degradation by depolymerization and molecular scission. Some polymers can gel and crosslink to yield brittle materials.

- Stiffening, and embrittlement due to loss of plasticizer and other auxiliary ingredients.

- Reduction in values for mechanical properties and increase in permeability due to swelling which, in the extreme case, results in dissolution.

- Failure of membrane seams due to interaction with the waste liquids and due to stress on the seams.

The principal agents aggressive to polymeric compositions are heat, oxygen, light, ozone, moisture, atmospheric NO_2 and SO_2, solvents, low temperatures, stress and strain, and enzymes and bacteria. All of these agents can be operative in the exposure of polymeric membranes in service. In most situations two or more of these agents act together. Table 3 outlines the various degradation processes that might occur with membrane liners in service.

Membranes rarely encounter the temperatures that would cause polymer decomposition. However, sometimes the elevated temperatures involved in weathering and possibly the impoundment contents might cause oxidative thermal degradation in the presence of oxygen. Photodegradation is only encountered on weather exposed surfaces. Most polymers are susceptible to UV degradation; however, the introduction of UV absorbers, such as carbon black and UV stabilizers, can greatly reduce and essentially eliminate this effect. Ozone can be particularly damaging and cause cracking in polymers that have unsaturation in their main chains. Ozone-cracking can only occur at points of strain of 15-25% or more. Of the polymers listed in Table 1, only IIR and CR have unsaturation in their main chains and can crack.

Polymeric compositions under constant or cyclic stress and strain can fatigue and lose mechanical strength, and crack. As is characteristic of all materials, polymers creep under stress. With lining materials this can result in thinning and puncturing or rupturing of a membrane. Environmental stress cracking, a type of failure of some polyethylenes, involves the cracking of a strained material in the presence of aggressive chemicals.

TABLE 3. POTENTIAL DEGRADATION PROCESSES IN POLYMERIC LINERS DURING SERVICE

Process	Effect on membranes
In weather exposure[a]	
Oxidation	Stiffens and loses tensile strength, elongation, tear
Elevated temperature	Reduction of mechanical strength and degradation; generally stiffens, but sometimes softens
Ozone	Cracks at points of strain
UV light	Stiffens and cracks
Loss of volatile plasticizer	Stiffens and can become brittle
High humidity	Water absorption, leaching of antidegradant resulting in greater susceptibility to oxidation and UV
In waste exposure[b]	
Swelling	Softens accompanied by loss of properties--including increase in permeability
Dissolves (if solubility parameter of waste constituent equals that of membrane)	Hole or general loss of barrier function
Extraction of plasticizer	May stiffen and lose elongation
Extraction of antidegradant	Make more susceptible to degradation
Stress	Creep of liner; cracking and breaking
Interface of waste and weather	Combination of weather and waste exposure often more severe than either alone
Biodegradation if oxygen is present	Plasticizers, oils and monomeric organic molecules can be degraded

[a]Liner exposed on either a berm or a slope.
[b]Liner is buried, covered, or below the waste/weather interface area.

Polymers are generally considered to be resistant to biodegradation, although some types are known to degrade (7). However, oils, plasticizers, and possibly other monomeric type ingredients in compounds are biodegradable in the presence of air and humidity. Their loss can result in stiffening and embrittlement of some compounds.

Though the mechanism is primarily physical, the swelling of a polymeric material by a solvent, including water, is considered a chemical attack on the material. Polymeric materials can vary greatly in their interaction with solvents. The solvents are absorbed without affecting the molecular weight of the polymer and generally extract plasticizers and other ingredients that are soluble in the particular solvents. Also, it is possible that solvents can dissolve some of the polymers.

Some observations that reflect basic characteristics of polymeric compositions can be drawn from the data in Table 4. The swelling of a given polymeric membrane varies greatly with the solvent. Also, swelling by a given solvent varies with the polymeric membrane. The plasticizers that are in some of the compounds can be extracted and thus result in shrinkage of the membrane. Because those compounds with high plasticizer contents will have low polymer contents, the amount of swelling is reduced (only the polymeric component of a compound swells). Crosslinking limits the swell. The alloying of HDPE increases the sensitivity to solvents due to lower crystalline content. The data show that, within a range of solubility parameters for the solvents, there is a maximum swelling for each membrane.

The anomalous behavior of the acetone shows that parameters in addition to the solubility parameter are involved in swelling. The acetone possesses relatively high hydrogen bonding and dipole moment values, two parameters that probably are involved in polymer swelling. The results, however, show the importance of solvent type.

ENVIRONMENTS ENCOUNTERED BY MEMBRANES IN SERVICE

The environment in which a membrane liner must exist will ultimately determine its service life. Table 5 enumerates environmental factors that can affect the durability of polymeric liners in service. Environmental factors in geotechnical applications of liners are discussed in the following sections.

Municipal Solid Waste (MSW) Landfills

The environment in which a liner exists in a MSW landfill differs substantially from that in water

TABLE 4. SWELLING OF MEMBRANES AFTER 28 DAYS OF IMMERSION IN VARIOUS ORGANIC CHEMICALS AT ROOM TEMPERATURE

Membranes			Solvent and solubility parameter, % weight increase									
Polymer[a]	Type	Extractables, %	Iso-octane 6.8	Normal octane 7.6	Cyclohexane 8.2	Xylene 8.8	Acetone 9.8	Tetralin 9.8	Tetrachloroethane 10.4	Normal propyl alcohol 11.9	Methyl alcohol 14.5	DI water 23.2
IIR (44)	XL	11.79	64	79	133	102	-4	136	109	-4	-1	1[b]
CPE (100)	XL	17.42	-10	-10	5	100	-2	149	294	-9	-7	1[c]
CPE (154)	TP	11.57	2	3	18	89	46	159	>500[d]	1	2	4[e]
CSPE (148)	TP	6.17	2	4	108	>200[d]	4	>700[d]	>1000[d]	-2	6	5[e]
ECO (178)	XL	7.63	70	85	124	114	-3	160	120	-3	-1	28[e]
ELPO (172)	CX	4.01	27	34	67	61	2	59	46	3	1	0.4[e]
EPDM (94)	TP	14.1	-2	0	5	44	6	86	6	-5	-8	...
HDPE (184)	CX	0.73	5	6	8	8	1	9	9	0	0	...
HDPE-A (181)	CX	2.09	12	14	27	23	1	22	20	0	0	...
PB (221)	CX	3.68	...	19	47	22	...	...	28	...	...	...
PVC (150)	TP	32.73	1	0	4	8	83	60	392	-4	-8	3[e]
PVC-OR (144)	TP	30.97	-3	-1	1	2	117	37	492	-8	-13	4[e]

[a]Matrecon liner serial number in parentheses.
[b]174 days.
[c]49 days.
[d]Specimen too deteriorated to weigh.
[e]35 days.

reservoirs and canals, applications in which membranes have shown good durability (1). The liner is placed upon a prepared surface that has been sloped for drainage. Above the liner is a porous soil on which refuse is compacted. Leachate generated by water percolating through the refuse is intercepted by the liner, drained through porous soil, and collected for ultimate disposal.

TABLE 5. ENVIRONMENTAL FACTORS AFFECTING DURABILITY AND SERVICE LIFE

Compatibility factors with waste liquids:
- Chemical
- Physical

Weathering factors - geographic location:
- Solar radiation
- Temperature
  - Elevated
  - Depressed
  - Cycles and fluctuations
- Water -- solid, liquid and vapor
- Normal air constituents, e.g. oxygen and ozone
- Freeze-thaw and wind

Stress factors:
- Stress, sustained and periodic
- Stress, random
  - Physical action of rain, hail, sleet, and snow
  - Physical action of wind
  - Movement due to other factors, e.g. settlement
- Discontinuity at penetrations

Use and operational factors:
- Design of system, groundwork and installation
- Operational practice

Biological factors

Important conditions at the bottom of a MSW landfill that can affect the properties and the service life of the liner are:

- An oxygen-free environment.
- Absence of ultraviolet light.
- Humid to wet conditions that can result in swelling of a liner and leaching of compounding ingredients if the leachate is flowing.
- Cool and uniform temperatures, e.g. 10-20°C.
- Moderate acidity and dissolved organic constituents in the leachate.
- Relatively high overburden pressure, but only moderate hydraulic head pressure of leachate, as drainage above the liner should take place continually, and localized strains in the liner when load is placed, due to irregularity of the soil base.

To assess the durability of lining materials, a research project was undertaken by the EPA to evaluate membranes under conditions that simulated service in MSW landfills (4). During the 56 months of exposure in landfill simulators, six membranes did not seep and changed only modestly in physical properties, as shown in Table 6. The CPE and CSPE liners did swell and lose some of their physical strength. All lost some extractables, particularly the plasticized compositions of IIR, EPDM, and PVC. The seams, though intact, lost adhesive strength.

Knowledge of changes in membrane liners in MSW landfills is limited. Samples of lining materials from several small lined facilities containing MSW have been recovered and tested after exposures of up to 9 years (5). The membranes that were beneath the wastes tended to absorb leachate and appeared to sustain only modest changes in properties based upon information of comparable unexposed membranes. Portions of several of the liners were exposed to the weather and sustained changes; PVC became brittle, CPE and CSPE membranes stiffened and nonreinforced CSPE shrank. The thin PVC showed mechanical damage on the slopes of an impoundment due to sloughing of a sludge on the liner causing it to tear on underlying rocks.

Overall, polymeric membrane liners in a MSW environment retain their property values well. Seams

TABLE 6.[b] EFFECT ON POLYMERIC MEMBRANES OF 56 MONTHS OF EXPOSURE TO MSW LEACHATE

Item	Exposure, mo	IIR	CPE	CSPE	EPDM	LDPE	PVC
Type of compound[a]	...	XL	TP	TP	XL	CX	TP
Nominal thickness, mm	...	1.60	0.81	0.91	1.30	0.30	0.53
Volatiles (2 h at 105°C), %	0	...	0.10	0.29	0.50	0.00	0.09
	56	2.37	7.61	13.90	5.74	1.95	2.08
Extractables, %	0	11.0	7.5	3.8	31.8	3.60	37.3
	56	9.8	5.1	3.4	28.3	3.37	34.4
Tensile strength[c], MPa	0	9.9	15.6	12.1	10.2	14.8	17.8
Retention, %	56	101	87	120	98	120	106
Elongation at break[c], %	0	395	410	250	410	505	280
Retention, %	56	103	94	91	91	107	121
Seam strength							
Location of seam preparation	...	Lab	Lab	Lab	Factory	Lab	Factory
Bonding system	...	Adhesive	Solvent	Cement	Adhesive	Heat	Cement
Shear strength, kN m^{-1}	0	5.2	>10[d]	>10[d]	7.8	>3.5[d]	>6.5[d]
	56	2.9	2.9	1.8	3.1	>1.9[d]	3.8[d]

[a]XL = Crosslinked; TP = thermoplastic; CX = partially crystalline thermoplastic.
[b]Matrecon serial number; R indicates liner is fabric-reinforced.
[c]Average of values in machine and transverse directions.
[d]Break in specimen outside of seam.

made with adhesives tended to lose in strength without leaking; seams that were heat sealed or made with bodied solvent maintained their strength.

Hazardous Waste Facilities

The environments encountered by liners in hazardous waste surface impoundments contrast greatly with those encountered by a liner in a MSW landfill or a water reservoir and can pose a much greater test of the durability of the membranes. The principal difficulties arise in the highly aggressive nature of the wastes to be contained and the stringent requirements to prevent transport of waste constituents out of the impoundment.

Environmental conditions that are encountered by liners in hazardous waste impoundments and can affect properties include:

- Exposure to a vast array of different materials in direct contact or under a soil cover.
- Variable exposure to weather, e.g. sunlight, rain, wind, ozone, and heat.
- Wave action of the fluid in the pond.
- Intermittent exposure to both waste fluids and weather.
- Low and high fluctuating temperatures.
- Ground settlement and movement.
- Irregularity of the soil beneath a liner.

Because of the lack of information with respect to the durability of membranes in contact with hazardous and toxic sludges, a second research project was initiated in 1975 by EPA (3) to obtain such information. The basic approach in the research was to expose specimens of commercial membranes under conditions that simulated real service. We used actual wastes, measured seepage through the specimens, and measured the effects of exposure by following changes in important physical properties of the respective lining materials. The results of immersion testing of a series of liners in eight wastes are presented in Table 7.

The ability of organic lining materials to absorb dissolved organic constituents from an aqueous liquid can have a highly significant effect on long exposures. In an experiment performed to demonstrate this effect specimens were immersed in a saturated aqueous solution of tributyl phosphate. The effects on a group of selected membranes of 17.2 months of immersion are shown in Table 8. The data show a great range in the weight gain of the various materials and the corresponding effects upon properties. The effect of crosslinking in reducing swelling is shown by the crosslinked CPE which gained only 34% in weight compared with 107% for the thermoplastic CPE.

TABLE 8. EFFECTS ON SELECTED POLYMERIC MEMBRANES OF IMMERSION IN WATER CONTAINING A LOW CONCENTRATION OF A DISSOLVED ORGANIC CHEMICAL[a]

Polymer	IIR	CPE	CPE	CSPE	PVC	HDPE
Type of compound[b]	XL	TP	XL	TP	TP	CX
Initial thickness, mm	1.60	0.76	0.91	0.84	0.84	0.81
Analytical properties:						
Weight gain, %	22	107	34	32	46	0.56
Physical properties:						
Final thickness, mm	1.63	1.22	1.04	0.96	0.92	0.80
Tensile strength[c], %	107	10	63	48	31	88
Elongation at break[c], %	115	155	79	79	89	101
Stress at 100[c], %	74	6	46	80	28	92
Tear resistance[c], %	...	14	29	39	23	81

[a]Immersion in solution of 0.1% tributyl phosphate in deionized water for 17.2 months.
[b]TP = thermoplastic, XL = crosslinked, CX = partially crystalline thermoplastic.
[c]Percent retention of the averages of measurements made in both machine and transverse directions.

SERVICE LIFE AND DURABILITY TESTING

At the present time information exists on the outdoor exposure of polymeric materials (8) and methodologies are being developed for durability testing of materials that are exposed to weather, such as on the berms and slopes of uncovered impoundments and reservoirs. Rossiter, et al (6) describe a methodology for predicting the service life of single ply roofing materials which, in many respects, should be applicable to uncovered membrane liners. The

TABLE 7. PERCENT ABSORPTION OF WASTE BY POLYMERIC MEMBRANE ON IMMERSION IN ACTUAL WASTES

Liner data[a]			Waste and immersion time in days at 23°C							
Polymer	Type	Extractables, %	Pesticide 807 d	Nitric acid 751 d	HF acid 761 d	Spent caustic 780 d	Lead 786 d	Pond 104 752 d	Aromatic oil 761 d	Weed oil 809 d
IIR	XL	11.8	1.6	3.8	3.7	0.8	28.7	103.9	31.2	64.2
CPE	TP	9.1	12.7	19.9	12.9	1.1	118.9	36.9	230[b]	Dissolved
CSPE (R)	TP	3.8	17.3	10.0	9.0	4.3	120.7	49.5	105.2	368.4
CSPE	TP	4.1	15.7	10.9	7.7	3.3	116.2	55.0	110.5	347.5
ELPO	CX	5.5	0.5	7.6	1.1	0.6	17.0	28.9	29.4	38.1
EPDM (R)	TP	18.2	4.5	4.2	3.1	1.6	24.8	26.5	19.8	84.4
EPDM	XL	23.6	20.4	50.9	23.9	1.3	34.7	84.7	34.2	76.2
PEEL	TP	2.7	4.2	6.4	2.0	1.5	7.4	8.5	16.6	14.7
PVC[d]	TP	33.9	5.1	22.1	18.1	0.4	-1.5	-10.4	18.5	15.3
PVC[d]	TP	35.9	1.0	-6.1	0.9	-0.9	7.4	-0.5	28.9	24.7
PVC[d]	TP	33.9	1.6	28.2	14.3	1.1	-5.2	-9.8	14.1	25.2

[a]Liner data: R = fabric-reinforced; XL = crosslinked; TP = thermoplastic; CX = partially crystalline. Extractables measured after volatiles are removed.
[b]Specimen partially dissolved in the waste; value reported is an approximation.
[c]Polyester elastomer.
[d]PVC from different manufacturers.

durability testing of materials that are immersed or intermittently immersed in waste liquids, however, has not been fully developed. Chemical compatibility-type tests, in which samples of lining materials are immersed, have been developed (5). In these tests the retention of selected properties are observed as a function of immersion time. These tests, however, do not indicate the effect of immersion under strain and other mechanical stresses due to temperature cycling, soil settlement, etc. In the development of durability tests, field experience is necessary to determine the type of distress that a liner will encounter and the incidence of such distress. With the feedback of such information appropriate durability tests can be developed.

ACKNOWLEDGMENTS

The work reported in this paper was performed under the following contracts with the Municipal Environmental Research Laboratory of the U.S. Environmental Protection Agency, Cincinnati, Ohio: 68-03-2134, 68-03-2173, 68-03-2969, and 68-03-2824. The support and guidance of R. E. Landreth, Project Officer, are gratefully acknowledged.

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Fundamental Aspects of Chemical Degradation of Geomembranes

The prediction of the service life of geomembranes when exposed to chemicals has usually been by way of testing for physical or mechanical property changes after periodic incubation times. This paper presents an alternate approach by evaluating four different diffusion related test properties of the incubated geomembranes. These tests are water vapor transmission, radioactive tracer transmission, water absorption and water vapor absorption. The rational for this approach is that chemical degradation might appear in the geomembrane at the micron level, and be sensitive to diffusion measurements, long before changes in traditional test properties are observed.

INTRODUCTION

Before any meaningful design can begin on a geomembrane based liner or cover, its chemical compatibility with the contained materials must first be assured. Furthermore, this compatibility must be guaranteed for the lifetime of the facility. For containing relatively inert substances, like liners for potable water reservoirs, this usually poses little challenge, and other considerations in the design process take a higher order of priority. When chemicals or materials producing leachates are being contained, however, the task is much more difficult. In this regard, many chemical resistance studies have been performed by manufacturers showing the performance of most geomembranes with a variety of chemicals. These tables and charts are generally reliable and should obviously be consulted before liner selection.

The challenge becomes quite formidable, however, when one deals with a complex waste stream from a landfill, i.e., leachate, or with waste stream from proposed facilities still in the design stage. Even if the individual components of these waste streams can be synthesized, the synergistic effects of the combined chemicals is almost impossible to estimate. The approach generally taken in such cases is to place the candidate geomembranes in the actual or synthesized waste streams, incubate them for a period of time, and then test them for changes in physical or mechanical properties. Depending on the magnitude of these changes, each particular geomembrane is then accepted or rejected. Problems invariably come up in these decisions, however, as to what are tolerable or intolerable changes in properties. Obviously, no change is the ideal. Yet, most geomembranes do respond (even to water) insofar as changes in thickness, volume, stiffness, modulus, elongation and strength are concerned. Furthermore, these changes are often quite uniform and linear over long periods of time. It should be noted that the application of heat (usually 70°C or 50°C) has been the thrust of many manufacturers to accelerate the ageing process, but the basic problem of defining an acceptable or unacceptable limit is still an elusive goal. An alternate method of accelerated ageing assumes that degradation can be modeled as an Arrhenius process(1), i.e., the degrading reaction has an exponential temperature dependence. To make such an analysis, test specimens are aged at several high temperatures until a defined failure level is reached. Failure times are often linear on a Arrhenius plot i.e., time versus inverse temperature. However, this method has severe problems if the material properties themselves are strongly temperature dependent, as they are in many geomembranes.

It is with the above thoughts in mind, that the authors feel an alternate approach is worth pursuing. This approach still uses an incubation of the geomembranes in the proposed liquid, but now assesses the material's performance on the basis of its diffusion protertíes.

Our basic hypothesis is that if the geomembrane has been degraded by the liquid it has been exposed to, its internal structure will have been altered. If so, some form of diffusion properties (actually its mass transport characteristics) would probably be among the most sensitive parameters to evaluate. Note the scanning electron micrographs of Figure 1 which show the edge view of a 30 mil thick PVC geomembrane in the as-received condition and after four minutes exposure in methylene chloride. The inner-connected pore structure seen in these photographs indicate strongly that a diffusion type of study could be very rewarding.

In this regard, a number of diffusion related tests can be performed. In particular, the following tests will be evaluated and discussed, thereby forming the basis of this study.

- water vapor transmission
- radioactive tracer transmission
- water absorption
- water vapor absorption

What is anticipated by these diffusion measurements is that abrupt changes in the measured diffusion parameter will indicate unacceptable performance in a more clear-cut and unequivocal manner than physical or mechanical property testing. Stated differently the microstructure changes at least at the micron level (as measured by diffusion tests) might be more sensitive to chemical degradation of the geomembrane than the macrostructure changes (as measured by physical or mechanical tests).

Graphically, the anticipated behavior is as shown in the three alternates of Figure 2. Curve (a) shows an unacceptable behavioral trend by having a breakthrough occur and a rapid increase in the measured diffusion property. Curve (b) illustrates the case where chemical degradation has occurred and the products of the reaction have "blocked" the microstructure resulting in an abrupt decrease in the measured diffusion property. Finally, in curve (c) the measured diffusion property remains constant showing an unaffected geomembrane diffusion to the liquid it was exposed to. It is obviously the preferred response and therefore a completely acceptable situation.

Before beginning a description of the individual tests, it must be mentioned that the study is still in an early stage and only preliminary data is currently available. Each method will be illustrated, along with data, but the long term behavioral curves (such as illustrated in Figure 2) have not yet been compiled.

CONDITIONING OF GEOMEMBRANES

Conditioning, or incubation, of geomembranes is vital to assessing their ultimate in-situ performance. The procedure selected should conform to the following requirements:

- representative of field conditions
- reproducible from one sample to another
- realistic to perform on a routine basis

In this light, there seems to be three alternates to choose from. Complete immersion of the geomembrane in the test liquid. This method has formed the majority of conditioning work to date. Here no folds, nor seams, are exposed to the chemical liquid but the edges of the geomembrane are. For laminated or reinforced geomembranes this seems undesirable due to capillary effects attacking the edges which is not typical of usual field conditions. The formation of a complete enclosure, or pouch, has also been proposed[2], whereby the chemical liquid is within the geomembrane and it acts as its own enclosure. Folds are at a minimum and no liquid loss can occur, but the seams must make contact with the chemical liquid and could modify the desired reaction. The third method attempts to simulate a miniature pond as closely as possible. Called tub exposure, the geomembrane is folded into an empty tub which is used for its support and then the chemical liquid is placed within the geomembrane. Folds are indeed present, at 90° and 180°, as well as flat surfaces along the bottom and partially wet and dry surfaces along the sides. The setup seems to simulate most of the geomembrane lined containment sites and is the type of exposure we will use in this study.

Three tub exposure setups have been constructed out of 4' x 8' plywood sheets (3/4" thick, exterior grade), each containing 12 tubs with geomembranes in each. See the photograph of Figure 3 for a typical single unit. Three geomembranes (PVC, CPE and EPDM) are being exposed to four chemicals (10% phenol, 100% xylene, 10% hydrochloric acid and 10% sodium hydroxide). They are removed after short, medium and long exposure times (typically 4 weeks, 12 weeks and 6 months) and when combined with the as-received data will give four data points to evaluate the ageing process. The exposed geomembranes are water rinsed and air dried after removal from the tubs. At that time the individual specimens for diffusion testing are taken from the flat areas (0° bend), the side portion (90° bend) and the corners (180° bend). This latter area is critically important since failure often occurs in the folded portion of geomembranes due to high stress concentrations which exist in these locations. Test specimens are then stored in plastic bags until testing commences.

WATER VAPOR TRANSMISSION

Water vapor transmission (WVT) tests are ongoing according to the ASTM E-96-66 standard test [3], the exception being that mechanical clamps are necessary for these relatively thick (30 mils) geomembranes. This replaces the wax method used for thin membranes, which seemed to allow water vapor to flow around the seal. A photograph of the test cups (containing water) and environmental chamber (at constant humidity and temperature) is shown in Figure 4 along with a set of typical data. Calculations proceed in a straightforward manner using the slope of the mass versus time curve as follows:

(a) Water Vapor Transmission (WVT) $= \dfrac{g \times 24}{t \times a}$

where

g = weight loss (grams)
t = time interval (hours)
a = area of specimen (m^2)

$$WVT = \frac{0.12\ (24)}{13(24)(0.003)} = 3.08\ \frac{g}{m^2\text{-}24\ h.}$$

(b) The permeance is given as

$$\text{Permeance} = \frac{WVT}{\Delta P} = \frac{WVT}{S(R_1 - R_2)}$$

where

ΔP = vapor pressure diff. across membrane (mm Hg)
S = saturation vapor pressure at test structure (mm Hg)
R_1 = relative humidity in cup
R_2 = relative humidity outside cup (in environmental chamber)

$$\text{Permeance} = \frac{3.08}{32(1.00-0.20)} = 0.120 \text{ metric perms}$$

(c) Permeability = Permeance x Thickness

$$= 0.120\ (30) = 3.6 \text{ metric perm-mils}$$

This value is in good agreement with literature value and when all four exposure times have been determined, curves such as Figure 2 should result.

RADIOACTIVE TRACER TRANSMISSION

The diffusion coefficient of a particular molecule of a diffusant can be determined if part of the molecule can be made radioactive. The diffusion coefficient is determined by monitoring the radioactive disintegrations versus time at some convenient point. Water can be made radioactive (tritiated water), but it is hard to work with because the radioactive disintegrations produce β-rays (electrons) which are of very low energy and are very readily absorbed in any geomembrane (and even in air). Therefore, we are using more energetic β-emitters, namely benzene with sulfur-35. Figure 5 shows a photograph and schematic diagram of the test device along with actual data illustrating Benzene (^{14}C) flow through 30 mil thick EPDM. It can be shown[5] that the solution for Fickian diffusion in a wide, thin sample is given by:

$$\frac{C_s - C}{C_s} = \frac{4}{\pi}\left[\exp\left(\frac{-D\pi^2 t}{4h^2}\right)\sin\frac{\pi x}{2h} + \frac{1}{3}\exp\left(\frac{-9D\pi^2 t}{4h^2}\right)\sin\frac{3\pi x}{2h} + \ldots\right]$$

where

C_s = surface concentration at $x = 0$
C = concentration at position x in the geomembrane thickness
D = diffusion coefficient
t = time
h = thickness

For all but the very shortest of times, the first term in the series dominates and the logarithm of both sides yields the following relationship;

$$\log\left(\frac{C_s - C_b}{C_s}\right) = \log\left(\frac{4}{\pi}\right) - \left(\frac{D\pi^2}{4h^2}\right) t$$

where C_b is the concentration of the diffusant at the far side, i.e. at $x = h$,

and the $\log\left(\frac{C_s - C_b}{C_s}\right)$ versus time curve gives a slope of

$-\frac{D\pi^2}{4h^2}$, from which the value of D can be extracted. Using this procedure a value of diffusion coefficient of 2×10^{-6} sq.cm./sec. is obtained for this particular geomembrane. This value is four orders of magnitude larger than that given for water diffusion in EPDM. It is known that benzene is a very good swelling agent for rubber-type materials(4) and swelling agents have unusually high values for apparent diffusion coefficients. This consideration, however, does not diminish the value of the method because it is relative changes in diffusion properties that are being sought as shown in Figure 2.

WATER ABSORPTION

The diffusion coefficient of a given species in a particular geomembrane can be determined by immersing the sample in water and monitoring the weight gain (i.e., its absorption) as a function of time. Diffusion theory is again used to obtain the diffusion coefficient. The theory involved in the measurement assumes a wide, thin membrane with lateral dimensions much larger than the thickness. The solution for the weight change, assuming pure Fickian diffusion is as follows:(5,6)

$$\frac{M_t}{M_\infty} = 4\left(\frac{Dt}{h^2}\right)^{\frac{1}{2}}\left[\frac{1}{\pi^{\frac{1}{2}}} + \sum_{n=1}^{n=\infty} (-1)^n \text{ ierfc } \left(\frac{nh}{2(Dt)^{\frac{1}{2}}}\right)\right]$$

where

M_t = the weight gain at time t

M_∞ = the weight gain at very long times, that is when the equilibrium concentration of diffusant is present

D = diffusion coefficient

h = half the thickness of the sample

ierfc = the integral of the error function

Figure 6 shows the general solution of M_t/M_∞ for the dimensionless parameter $(Dt/h^2)^{\frac{1}{2}}$. The M_t/M_∞ time dependence is strictly $t^{\frac{1}{2}}$ up to a value of M_t/M_∞ approximately equal to 0.50. This occurs at a $(Dt/h^2)^{\frac{1}{2}}$ value of about 0.5 also. Note that it takes a very long time (with respect to the time for the break from the $t^{\frac{1}{2}}$ dependence to occur) for complete saturation to occur, i.e., the $M_t/M_\infty = 1.0$. Therefore, if t_{break} is known, the diffusion coefficient, D, can be obtained from the break of the curve of Figure 6. Figure 7 gives some typical data from this study for CPE in water at 30°C. Note the break in the curve for a time = 1.0×10^6 seconds. Therefore the diffusion coefficient for water into CPE is given as

$$\left(\frac{Dt_{break}}{h^2}\right)^{\frac{1}{2}} = 0.5$$

With h = 0.038 cm and $t_{break} = 10^6$ seconds, one obtains:

$$D = \frac{(0.5)^2(0.038)^2}{10^6} = 2.9 \times 10^{-10} \text{ cm}^2/\text{sec.}$$

Also note from Figure 6 that it will take a value of $(Dt/h^2)^{\frac{1}{2}} \approx 2$ to obtain near-saturation. That is, it will take a time of order of $(2/0.5)^2 \times 10^6 = 1.6 \times 10^7$ seconds, or approximately 185 days to be able to determine M_∞. This makes the t_{break} approach very attractive from a logistical standpoint.

WATER VAPOR ABSORPTION

In precisely the same manner as with water absorption test just described, the use of water vapor as an absorbed gas can be used to obtain the diffusion coefficient. The theory is exactly the same, and only a slight modification in the experimental setup is necessary. Now, instead of suspending a geomembrane in a controlled temperature bath, it is encapsulated in a separate glass cylinder with water in the bottom which in turn is suspended in the water bath. Thus 100% humidity conditions are present, which allows for water vapor to enter the geomembrane as a function of time. The increase in weight is plotted against time as in Figure 8, which results in the following calculation: Using h = 0.038 cm and $t_{break} = 7 \times 10^5$ sec, one obtains

$$D = \frac{(0.5)^2(.038)^2}{7 \times 10^5} = 5.15 \times 10^{-10} \text{ cm}^2/\text{sec}$$

However, this calculation is very suspect since the initial part of the curve does not follow the $t^{\frac{1}{2}}$ law. More investigation into this problem will be required. As with the other types of diffusion data the results will be plotted as shown in Figure 2 as a function of incubation time of the geomembranes in the various chemicals noted earlier.

SUMMARY AND CONCLUSIONS

As presented in the introduction, a method of assessing the deterioration of geomembranes as a function of exposure time to various chemical liquids is critically important. Current methods of service life prediction (i.e., chemical ageing) are seriously lacking in this area. The logic behind the work presented in this paper, is that perhaps a diffusion-based measurement will yield more fruitful results than physical or mechanical property testing. Certainly geomembrane deterioration should start at the micron scale before any large scale evidence is noted, recall Figure 1.

Toward this end four different diffusion related tests are being explored. These are water vapor transmission, radioactive tracer transmission, water absorption and water vapor absorption. All result in a type of diffusion coefficient which can be plotted against exposure time of the geomembrane in a specific chemical to possibly indicate degradation. It is anticipated that such indications will occur much earlier in time than in other types of measurements. All four methods being used are generating data and none seems preferred over the other at this point in time. Since the study is still ongoing no further conclusions can be reached.

While not diffusion related, other non-mechanical measurements are also being contemplated. These are the following:

Glass transition temperature(7-9) changes which can be loosely described as that temperature upon heating (from a rather stiff state) where significant excitation of thermal modes can occur in a reasonable time scale (i.e., a few seconds) and the material can become more flexible. The glass temperature is sensitive to many variables. It is sensitive to the cooling rate during fabrication, molecular weight, polymer chain mobility, etc. It has been observed in our work to date that some of the FML's become stiffer with chemical exposure (PVC is a prime example), indicating a raising of the T_g to somewhere near room temperature. (The untreated material has T_g values considerably below room temperature.) Therefore T_g values should be capable of being monitored after various exposure times using differential scanning calorimetry.

Viscosity-molecular weight measurements are possible in which the polymer is dissolved in a solvent and the viscosity of the solvent plus dissolved polymer is measured.(10) The viscosity goes up as the molecular weight increases and many calibration curves are available in this regard for a variety of polymers in many solvents. Molecular weight changes from polymer chain rupture are usually thought to occur from ultraviolet and stress affects. However, it has recently been shown(11,12) that exposure of nylon and polyethylene to various pollutant gases (O_3, NO_X and SO_2) will also cause (together with stress) a molecular weight lowering. The transport properties depend on molecular weight, therefore, it is deemed potentially valuable to monitor a breakdown of the molecular weight as a means of predicting geomembrane lifetime.

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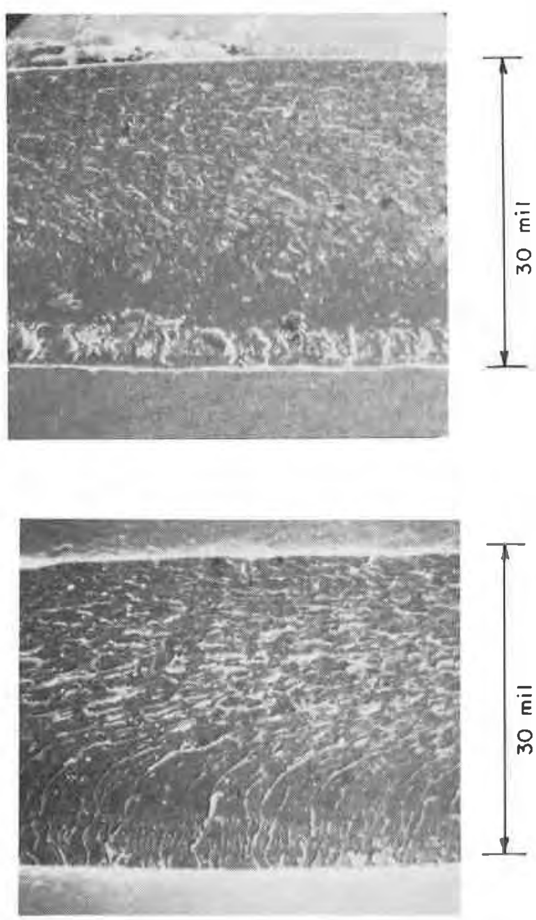

Fig. 1. - Scanning Electron Micrographs of Edges of 30 mil PVC Geomembranes Before (Upper) and After (Lower) Exposure to Methylene Chloride

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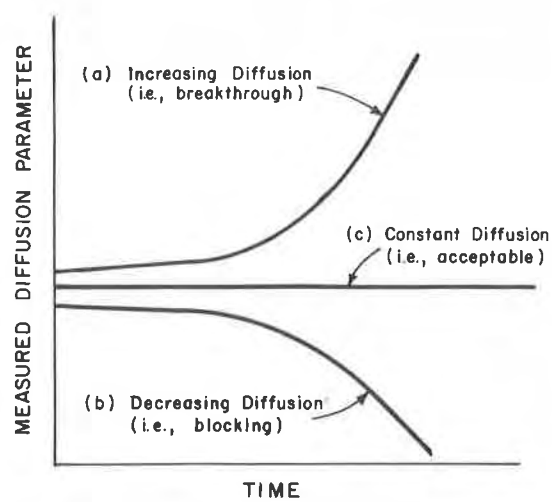

Fig. 2. - Various Possible Trends in Diffusion Properties as a Result of Geomembrane Incubation Time

Fig. 3. - Photograph of Tub Exposure of Geomembranes to Various Chemicals

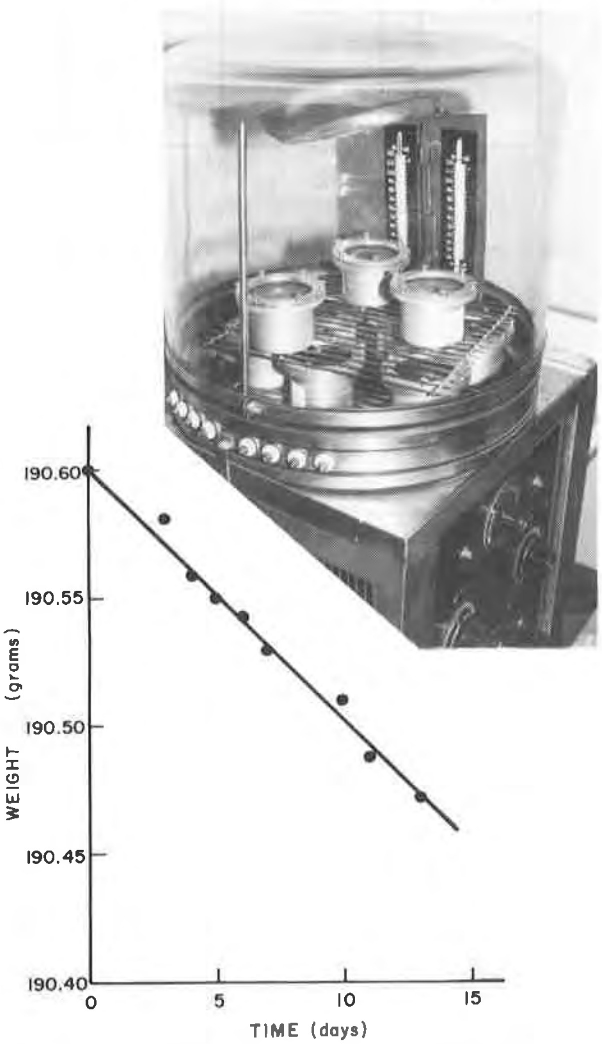

Fig. 4. - Photograph of Water Vapor Transmission Test Setup and Data for 30 mil EPDM at 90°F and R. H. difference of 80%

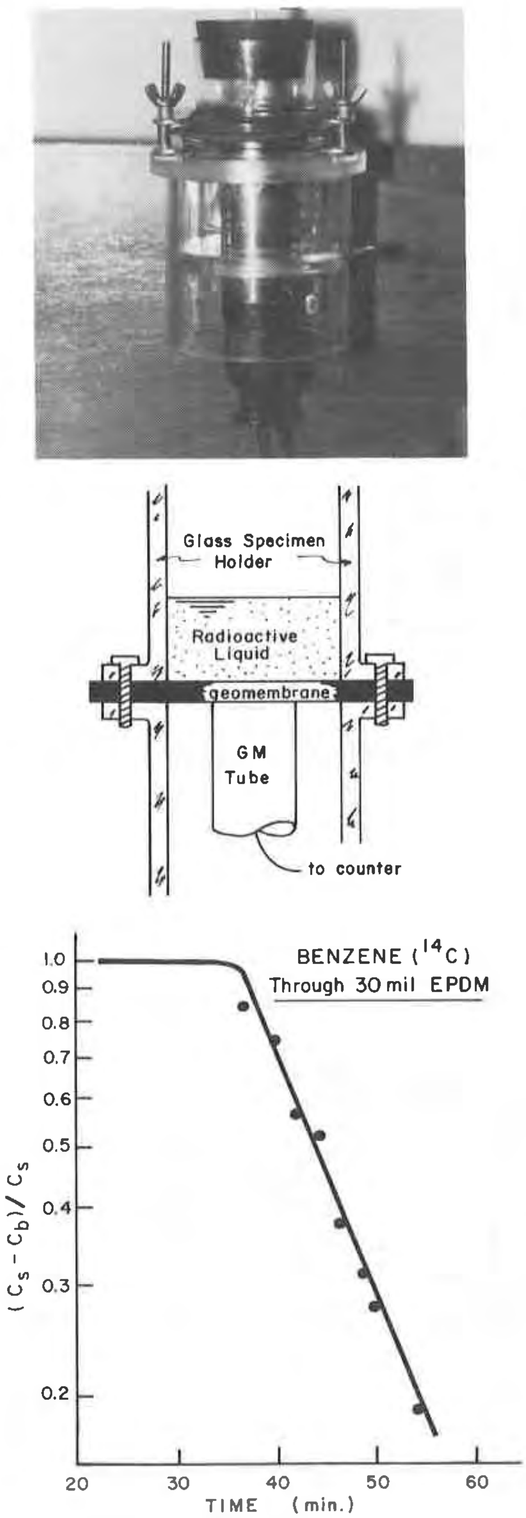

Fig. 5 - Photograph and Schematic Diagram for Determining Diffusion Constant Using Radioactive Tracers along with Test Data for 30 mil EPDM

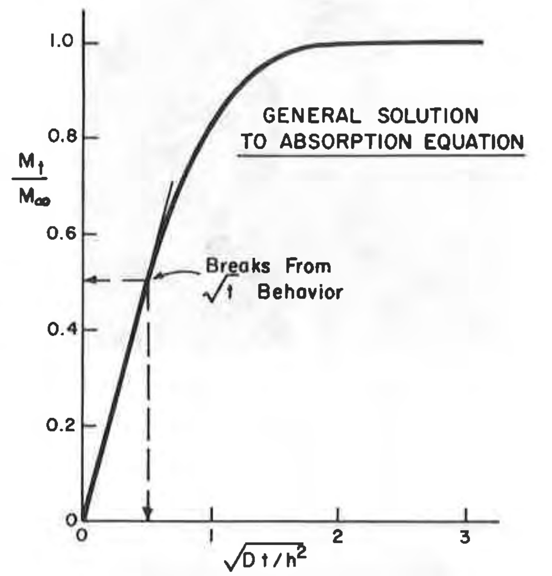

Fig. 6. - General Solution of Diffusion Equation for Use in Absorption Studies

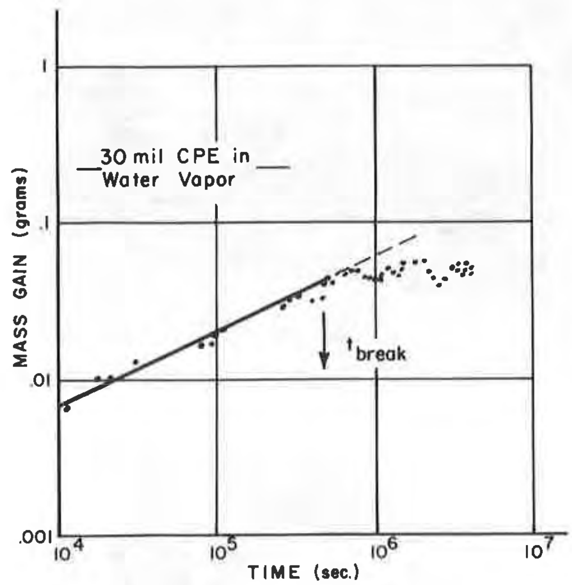

Fig. 8. - Absorption Data of Water Vapor in 30 mil CPE Geomembrane

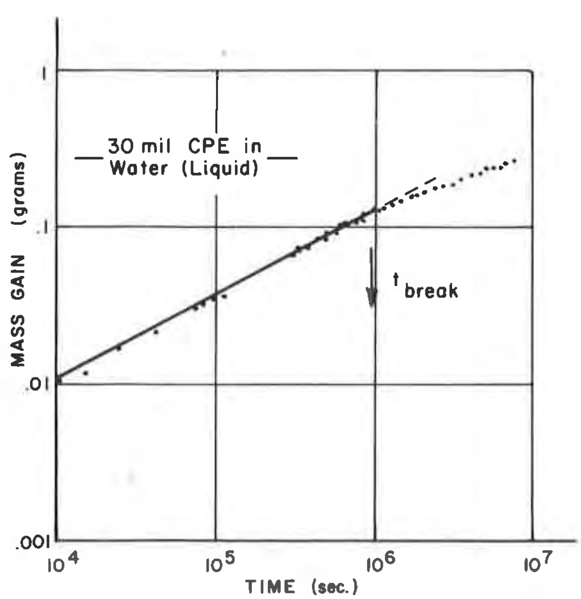

Fig. 7. - Absorption Data of Water (Liquid) in 30 mil CPE Geomembrane

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Contributions to Study of Durability of Butylgeomembranes

In the works where they are used, the geomembranes are submitted at stresses which are on the one hand mechanical and on the other hand chemical. The more or less simultaneous action of the stresses can provide damages in the works.

Being aware of the fact that the studies on the ageing of polymers require simultaneously the evolution of the chemical and mechanical characteristics, a protocal of tests calling to chemical analysis methods such as infrared analysis and to physical methods such as the determination of breaking strain and creep recovery has been defined.

Two samples of butylgeomembranes have been took away from sites which are discribed and studied according to the established methodology. It is tryed to explain the observed degradations by results of the tests.

I INTRODUCTION

Butylgeomembranes result from the polymerisation of isobutylene with a low ratio of isoprene. From this fact, butylmembranes have a mainly paraffinic character, a low rate of ethylenic bonding which is just enough to allow the vulcanization treatment, and a close ordering of the macromolecules, which provide a good resistance against oxidation, chemical agents and UV radiations.

Nevertheless, we can see the damages which occur on the butylgeomembranes by chemical and/or chemical stresses. The latter come either from agressiveness of fluids in contact with them or from the action of UV radiation.

In order to forsee the mechanisms of butylmembranes ageing, two ways can be retained. The first one is to study accelerated ageing tests under wellcontrolled conditions which should be representative of the outdoor stresses. For example, the action of UV must be done with UV generators the wavelengths spectra of which is above 290 nm. Indeed UV radiations emitted by the sun have to pass through the different atmospheric layers before reaching the earth and, by screening effect of high levels of atmosphere, the part of sunlight the wavelength of which is below 290 nm is stopped.

The second way is the examination of samples of membranes taken from the work where they are in using and the comparison with the mechanical and chemical characteristics of the cheek sample. The durability of geotextiles has been undertaken following the sames ways (1, 2). In a earlier study (3), we have shown that the behaviour of geomembranes can be investigated through Infrared spectrography and mechanical tests. Such methods have been used for the examination of geomembranes of two sites.

II DESCRIPTION OF SITES

II.1. A water-reservoir in Normandy

This reservoir is built for the storage of water for industrial uses. Schematically, the reservoir is made up of three basins, a little of one hundred m^2 in which the water is arriving by the bottom and two great ones which are filled from the first basin by a waste-wier.

The surface at the bottom of the great basins is two hectares, the slope of the bank is 3/2 and the embankments are height of six meters. On an average the height of water is between four and five meters. The concentrations of chlorine in the little basin vary between 0,5 and 0,7 mg/l.

On the bottoms and the embankments of these basins, a butylmembrane was put. The characteristics of the membrane are :

Thickness....................	1,5 mm
Density......................	1,2 kg/m³
Minima of breaking stress.....	8,5 MPa
Minima of breaking elongation.	300 %

The filling of the reservoir took place in 1972.

Because of a rupture of the membrane at the bottom of one of two great basins, the use was stopped in August 1981.

Two strips of the membranes were taken in July 1982 and four samples were cut up according to diagrams fig. 1a and 1b.

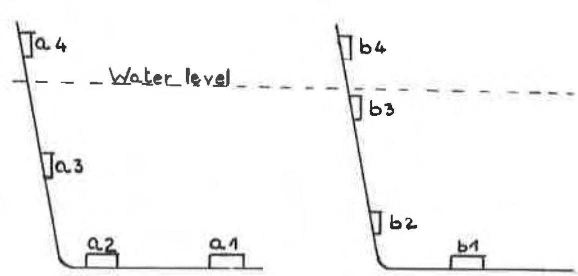

fig. 1a

fig. 1b

The samples a4 and b4 have been never dipped. The other samples have been practically dipped all the time until the damages of the basin happened. Since they have been exposed to the ambient atmosphere.

II.2. Callas Dam near Fayence

It is a little earth dam (fig. 2) the up-side embankment of which has been covered with a butylmembrane (4). Its thickness is 1 mm. The slope of the bank is 2/1.

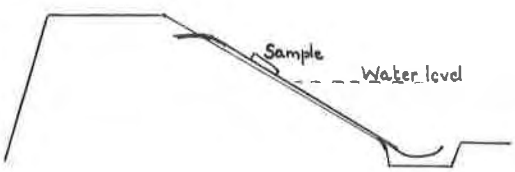

fig. 2

Some repairs have been made in place where the membrane has been punched. A sample was picked up in autumn 1982 in the zone above the waterlevel. At this time, the membrane was working normally.

III EXPERIMENTAL

Experience got during the study of geotextile durability shows that the modification of characteristics of a product could be realistically established only by correlations between its mechanical and chemical properties.

Consequently, the analysis methods have to be able to give the comparing elements which are necessary for a such interpretation. Tests used in this work were tensile experiments, creep recovery and infrared analysis.

III.1. Tensile experiments

They are made according to the French standard NF T 46002 June 1944 with a rate of strength 450 mm/mn. The breaking stress which is the breaking load divided by the cross-section of the sample is given in MegaPascal (MPa), the elongation is in percent.

III.2. Creep recovery tests

They are made from the French standard method NF T 46010. The samples are submitted to a stress of 6MPa during four hours. Rate of putting and removing the stress is 35 cm/mn The recovery is measured thirty minutes after removing the stress. Elongation versus the time is shown by the curve of the figure 3.

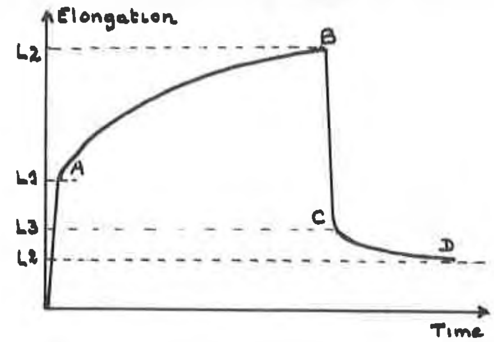

fig. 3 Extension - Time curves

Initial length L_0 of the sample become L_1 after the stress and varies until to L_2 under the action of the constant stress. When the stress is removed, the sample skrinks quickly to L_3 and L_4 after a certain time. The part B, C, D of the curve is the recovery curve.

We can define :

- Instantaneous elongation : $\frac{L_1 - L_0}{L_0} \times 100$
- Viscoelastic elongation : $\frac{L_2 - L_1}{L_0} \times 100$
- Immediat elastic recovery : $\frac{L_2 - L_3}{L_0} \times 100$
- Delayed elastic recovery : $\frac{L_3 - L_4}{L_0} \times 100$
- Permanent deformation : $\frac{L_4 - L_0}{L_0} \times 100$

III.3. Infrared analysis

Surface of the sample is scratched to obtain a fine pow-

		Thickness mm	Breaking stress MPa	Breaking elongation %
Cheek sample of Callas		1,07	11,5	520
Callas		0,96	9	340
Basin of reservoir of Normandy	a1	1,51	10,1	477
	a2	1,54	10,3	540
	a3	1,44	13,4	410
	a4	1,53	12,1	347
	b1	1,48	11,8	453
	b2	1,49	12,9	435
	b3	1,44	13,5	429
	b4	1,40	12,0	387

Table 1 Thickness and breacking characteristics.

		Instantaneous elongation	Viscoelastic elongation	Immediate elastic recovery	Delayed elastic recovery	Permanent deformation
Check sample of Callas		256	73,6	297,6	10,7	21,4
Callas		128	85	191	10	12,5
Basin of reservoir of Normandy	a1	326	79	347,5	27,5	30
	a2	370	75	395	22,5	27,5
	a3	207,5	72,5	248	15	16,5
	a4	169	70	215	11	11,5
	b1	283	76	320	15	22,5
	b2	230	65	262	12,5	20
	b3	200	78	250	9	15
	b4	225	75	270	10	20

Table 2 Characteristics of creep - Recovery.

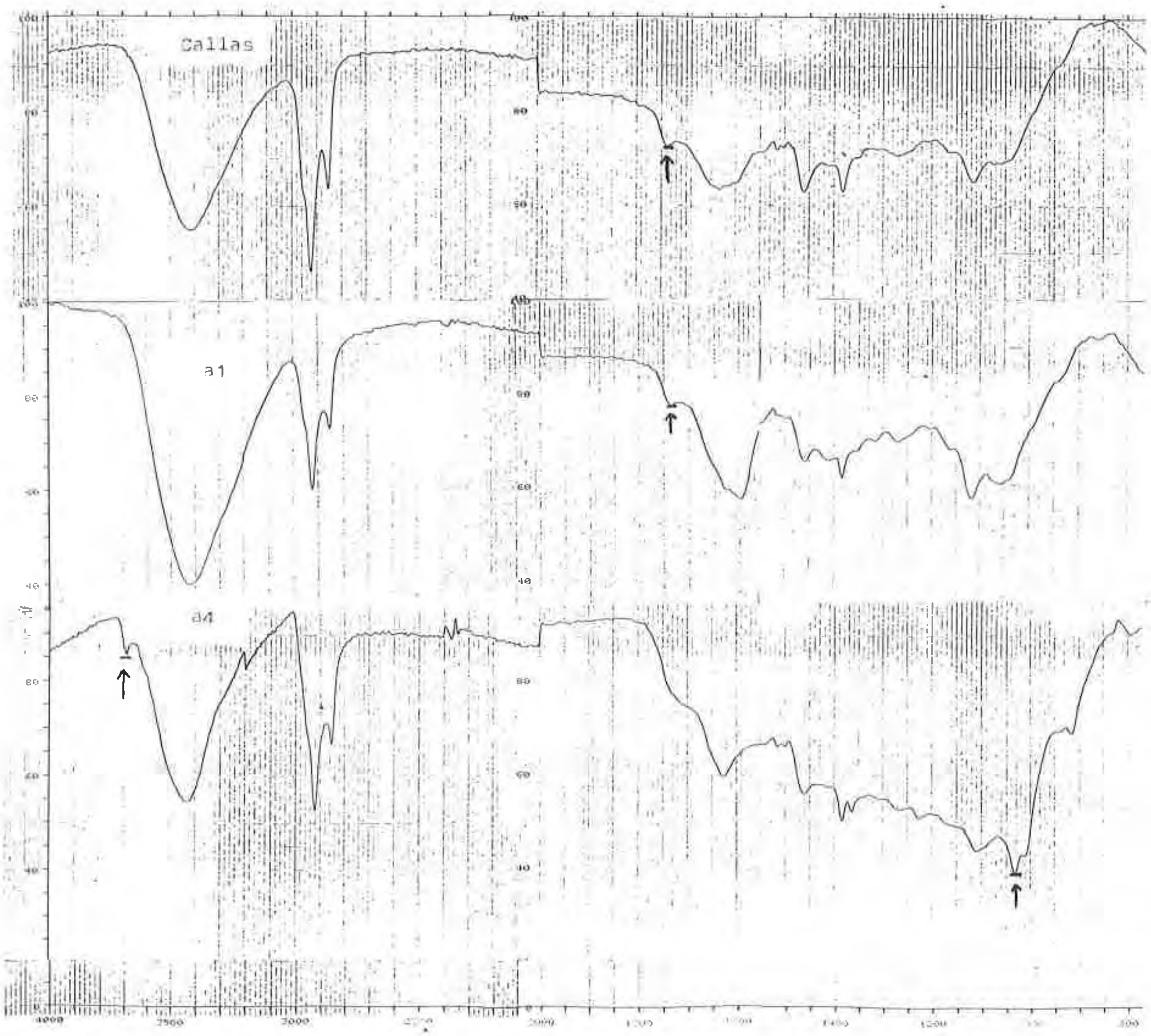

fig. 4 Infrared spectra of butylmembranes

der of butylmembrane which is mixed with potassium bromide. The pellets made with this mixture are observed with an IR spectrophotomer Perkin Elmer 983.

IV RESULTS AND DISCUSSIONS

The infrared spectra corresponding to samples a4, a1 and Callas are shown on the figure 4. Spectrum of b4 is identical with a4, spectra of a2, a3, b1 are identical with a1. Spectra of b2 and b3, which are not shown there, are between a1 and a4. On spectra of Callas and a1, there is a pick to 1740 cm^{-1} which indicates the presence of carbonyl function C = O. Spectrum of a4 shows two peaks, one at 3690 cm^{-1} corresponding to hydroxyl groups OH which can arise from hydroperoxyde groups and the other at 1030 cm^{-1} corresponding to an ether function C-O-C which can come from the interchain decomposition of hydroperoxyde groups. The butylmembranes of Callas and a4 or b4 have a common historic feature. They were always exposed to the weather, though not on the same site and nothing let us explain why their spectra have not the same peaks.

The spectra of Callas membranes is similar to these obtained from the previously studied membranes (3) which had been picked up in two sites where climatic conditions were very different. One came from a basin of water treatment in France and the other came from a tanker in the Reunion Island. Consequently, formation of carbonyl groups can be thought the general rule. Hydroxyl and ether groups of a4 should be explained either by particular outdoor conditions (dusts and chemical products in air and in rain) and/or by additives used during the manufacturing of the membranes.

Results concerning thickness and tensile rupture parameters of the studied geomembranes, which are the average of five measurements at least, are given table 2. Creep-recovery features are given table 3. Because results are reproducible during tests, the average of the table 3 is only from two measures.

Membranes of Callas, a4 an b4, which have been always exposed outdoor, show the most important losses of breaking elongation associated with a light decrease of the breaking stress. Moreover, during the creep-recovery test, they have a relatively weak capacity of deformation resulting from a decrease of the instantaneous elongation.

These results indicate that outdoor exposure induces a rigidification of butylmembranes. From these results, it can be observed that specimens located at the bottom of the basin have kept a good elongation breaking, but, and particularly a1 and a2, have lost in breaking stress and have a retarded elastic recovery and permanent deformation higher than the others. In the part where a1 and a2 were picked up, the membrane was broken because of kartzic soil falling. Before the rupture, membrane has been submitted to a stress which undertook a creep.

The samples such a3, b2 and b3 which were on the slope of the embankment have lost only a few of their breaking elongation. The membrane being flattened against the slope is submitted to a pressure which can produce a physical rigidification and, consequently, decrease the possibility of elongation.

V CONCLUSIONS

The tests realized on these membranes show a variation of their characteristics, but taking into account the number of studied membranes, it seems difficult to give general rules on the ageing of these butylmembranes.

Nevertheless, it appears that the outdoor exposure produces a membrane rigidification which leads to a decrease of the elongation capacity. Il is the same thing from the action of a pressure against the embankments, although by a different mechanism. The knowledge of losses of characteristics of membranes placed in various sites will be enable to adjust the severity of accelerated ageing tests and to forsee the durability of membranes.

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Aging Geomembranes in Uranium Tailings Leachate

Pacific Northwest Laboratory (PNL) is performing a study to provide the U.S. Nuclear Regulatory Commission (NRC) with a database to support licensing of uranium mill tailings ponds with geomembranes. As part of this study, geomembranes have been aged under conditions closely approximating those of uranium mill tailings ponds with acidic leachate. The aging procedure and results of tests with high density polyethylene (HDPE) and polyvinyl chloride (PVC) are presented.

No degradation products were detected in the HDPE or leachate. We expect that HDPE will not suffer significant chemical attack by uranium tailings and acidic leachate over the active life of a pond. No large changes in physical properties were observed.

Although testing is still underway, the PVC geomembrane has undergone some chemical reactions. Elongation declined as exposure temperatures increased, while other physical properties remained constant. The amount of aging simulated will be estimated when chemical analyses on PVC samples are completed.

INTRODUCTION

Geomembranes are required for containment of spent leachate and tailings at most new uranium milling operations in the United States. For proper performance, geomembranes must resist chemical attack and retain important physical properties over the active life of the tailings or evaporation pond. PNL is aging geomembranes in a simulated uranium mill tailings pond environment to predict performance and service life as liners for uranium mill tailings ponds.

Four materials--high density polyethylene (HDPE), polyvinyl chloride (PVC), chlorosulfonated polyethylene (CSPE), and chlorinated polyethylene (CPE)--have been used to line tailings and evaporation ponds at uranium mills in the U.S. For this project, these materials are being exposed to simulated acidic leachate under conditions typical of a uranium tailings pond. This paper presents the procedure for the aging tests and the results of tests completed to date on HDPE and PVC.

THEORY

To determine the resistance of a geomembrane to degradation by the liquid to be contained, geomembrane manufacturers generally perform compatibility tests for various classes of chemicals. These tests usually measure changes in physical properties as a function of exposure time and temperature. In these tests the geomembrane is usually submersed in a solution, so it is exposed on both sides. However, typical chemical compatibility tests do not account for chemical degradation of a geomembrane from combinations of factors, such as trace metals acting as catalysts and mechanical stress. Conventional compatibility tests also do not correlate the service life with the observed physical changes from exposure tests. Additionally, as a tailings pond liner, the geomembrane should only be attacked by chemicals on one side. When aging geomembranes to determine suitability as pond liners, it is desirable to simulate conditions as closely as possible to the real environment.

Pacific Northwest Laboratory has developed a method to estimate the service lives of asphalt liners in uranium mill tailings (1). This same technique is now being used to predict the service lives of geomembranes in uranium mill tailing ponds. Geomembranes are exposed to simulated acidic leachate on one side for 18 weeks in three columns at different temperatures. The membranes are then analyzed for changes in chemical composition and physical properties.

If chemical changes are significant, an activation energy may be determined for different reactions by examining the reaction kinetics. The amount of aging simulated at elevated temperature will then be equal to:

$$t \cdot \exp\left[\frac{-E_a}{R}\left(\frac{1}{T_1} - \frac{1}{T_o}\right)\right]$$

where: t = duration of exposure
E_a = activation energy, cal/g-mole
R = gas constant, 1.987 cal/g-mole°K
T_1 = absolute temperature of exposure, °K
T_o = absolute reference temperature, °K

For example, assume that an activation energy of 15,000 cal/g-mole is determined from exposure tests by plotting the logarithm of the observed rate versus the reciprocal absolute temperature. If the average ambient temperature is expected to be 18°C, then a sample aged for 18 weeks at 78°C will have a simulated age of 29 years. Thus physical properties of the liner exposed at 78°C are assumed to approximate those of a 29 year old liner.

Since it is possible that additional degradation reactions will occur at elevated temperatures, the maximum temperatures used in these tests are limited to 78°C. For this reason, weeks of exposure are required to quantify reaction rates.

Diffusion of reactants into the liner and movement of reaction products away from the liner can affect the kinetics (2). Diffusion effects necessitate chemical analysis as a function of depth into the geomembrane. To determine reaction rates as a function of depth, samples are microtomed to approximately 25 μm layers prior to chemical analysis.

APPARATUS AND PROCEDURE

Figure 1 shows a schematic of a column used for accelerated aging. Geomembranes containing field seams are cut to fit between flanges of the three exposure columns. A sand subgrade is placed in the base of each column to simulate an actual subgrade. Neoprene gaskets are placed above and below the geomembranes and each column is sealed. The simulated tailings (silica sand less than 210 μm) and leachate (see Table 1) are added, and the columns are sealed, leak tested and insulated. Perforated plate presses are loaded to simulate 2.3 m of tailings depth, and air pressure simulates 6 m of leachate depth. The columns are brought to the desired operating temperature (18°C, 48°C, and 78°C) over a period of 3 to 4 days. Temperatures are maintained by circulating fluid through coils on the column exterior. Temperatures are measured radially across the column near the liner. Leachate is circulated for monitoring pH.

Columns are checked 5 days a week for pH, pressure, and liquid level. Temperatures are recorded continuously. The pH is maintained between 2 and 2.5. (Concentrated sulfuric acid is added if needed.) On a weekly basis, pH meters are calibrated and the columns are depressurized and recharged with air to assure ambient oxygen concentrations.

After 18 weeks of exposure at 18°C, 48°C, and 78°C, the column are disassembled and the geomembrane is removed, examined, rinsed, dried, and prepared for physical and chemical tests. Physical tests include thickness, density, tensile strength, elongation, shear strength of seams, and tear resistance.

One additional physical test involves stressing the liners over a rocky subgrade, similar to methods reported by Rigo (3) and the U.S. Bureau of Reclamation (4,5,6). A device was constructed to simulate the stresses of a rocky subgrade (see Figure 2). This 25 cm diameter device contains a gravel base (-38 mm + 19 mm) which is glued to prevent shifting of the rocks between tests. Liners are placed in the device, covered with water, and pressure is increased at a prescribed rate until failure is evident by collection of water below the liner. The liners are then removed and the type of failure is analyzed.

Chemical test methods have included differential scanning calorimetry and differential infrared spectroscopy. Samples are microtomed to approximately 25 μm

TABLE 1. Typical Uranium Mill Leachate Compositions and Composition of Simulated Leachate Used in Aging Tests

Major Species, ppm	Highland Mill (7)	NRC Model (8)	EPA TRU Values (8)	Sweet-water Mill (a)	Simu-lated leach-ate
Al	600	2000	700-1600	151-180	1000
As	1.8	3.5	0.2	0.4	3
Ca	537	500	1.4-2.1	61-127	500
Cd	<0.1	0.2	0.08-5	ND	
Cl	97.1	300		40-100	300
Cr	2.7		0.02-2.9	2.0	3
Cu	2.3	50	0.7-8.6	1.0	5
F		5		0.5-1.6	
Fe	2215	1000	300-3000	495-1350	2215
Hg		0.07		0.004	
K				1-610	20
Mg	688		400-700	124	700
Mn	63.5	500	100-210	23	100
Mo	<5	100	0.3-16	0.1	5
Na	343	200		100-109	376
NH_3		500			500
Ni	3		0.13-1.4	1.3	3
P	30			0.05-0.09	
Pb	<1	7	0.8-2	<1	
Se		20		0.03	20
Si	233.5			186-281	230
SO_4	12850	30000		9312-9529	(b)
V		0.1	0.1-120	2.8-3.2	1
Zn	8.4	80		1.6-31	10
pH	1.8	2.0		0.9-1.99	2-2.5
Radionuclides, pCi/L					
Pb-210		250		1541	
Po-210		250		361	
U		3300		5.4 (ppm)	
Ra-226		250		47.99	
Th-230		90,000		3035	
Bi-210		250			

ND = none detected.
a) from site visit.
b) as required to maintain pH.

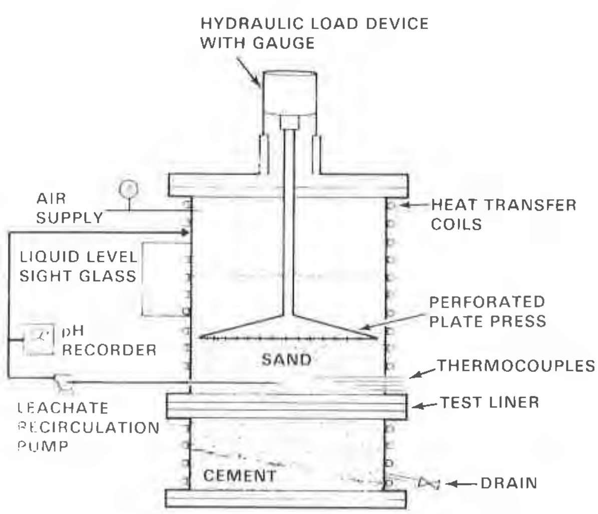

FIGURE 1. Schematic of Accelerated Aging Column

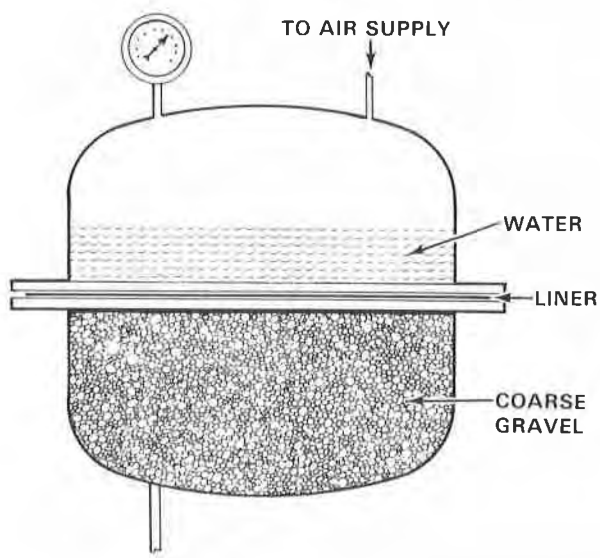

FIGURE 2. Schematic of Anomaly Conformance Test Device

layers so chemical reaction rates can be correlated as a function of depth in the liner. Additional tests may include thermogravimetric analysis and elemental analysis by various methods. Samples of leachate are also collected for analysis.

RESULTS

High density polyethylene and PVC samples were aged according to the previously discussed procedure. Average conditions during the exposures are shown in Table 2.

High Density Polyethylene

The HDPE was 0.99 mm thick with a fillet-weld, extruded seam provided by a HDPE liner manufacturer.

The only visible change evident after aging was wrinkles which were formed during installation of the liner. Thickness and density were unchanged.

The seams did not fail in shear tests; however, the tensile strength fluctuated (see Table 3). Tear resistance slightly increased with the exposure temperature. In tests over a rocky subgrade, there was no change in performance of the HDPE. All liners, virgin and aged, failed at 35 to 42 m of water head by developing very thin areas at stress concentrations and subsequently bursting.

According to the literature, polyethylene degradation in air occurs by oxidation producing carbonyl structures (2,9,10). References to degradation mechanisms in acidic aqueous solutions were not found. We therefore have analyzed the aged HDPE samples for chemical changes (especially carbonyl formation) as a function of depth by infrared spectroscopy. The results of the analysis for carbonyl are shown in Figure 3. These data show that there is no increase in oxidation products in the HDPE. We found no increase in organic carbon levels in leachates from the high and low temperature columns, indicating that oxidation products were not being dissolved in the leachate.

Crystallinity of the HDPE slightly varied with exposure, as determined with a differential scanning calorimeter. Average crystallinity was 52.3% for unexposed HDPE, and 51.5%, 53.5%, and 55.7% for HDPE exposed at 18°C, 48°C, and 78°C respectively.

TABLE 2. Average Conditions of Exposure for HDPE and PVC Aging Tests

	HDPE			PVC		
	Temperature, °C	pH	Head, m	Temperature, °C	pH	Head, m
Column 1	18	2.22	5.8	18	1.66	5.2
Column 2	48	2.27	5.9	48	2.11	4.7
Column 3	78	2.34	5.9	78	2.29	4.9

TABLE 3. Physical Properties of Virgin and Aged HDPE

	Force/Width at Yield, kN/m(a)	F_T (break force), N(b)
Virgin	17.9	139
Aged at 18°C	16.7	135
Aged at 48°C	18.3	147
Aged at 78°C	18.5	149

a) ASTM D882, 5 cm/min separation, 3 samples.
b) ASTM D1004, 5 cm/min separation, 3 samples.

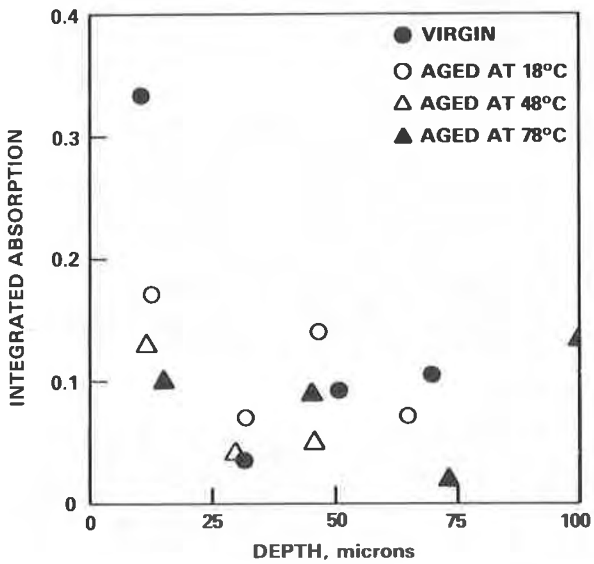

FIGURE 3. Results of Differential Infrared Analysis of Aged, High Density Polyethylene Samples. The integrated absorption is proportional to the carbonyl concentration.

Polyvinyl Chloride

The PVC used in the aging tests was a 0.82 mm thick nonreinforced liner with didecyl phthalate plasticizer, provided by a liner manufacturer. The sample contained a lap seam made by the manufacturer with the recommended solvent for field seaming.

In physical tests, no density changes were observed; however, a 2% increase in thickness was measured. After exposure, the liners were wrinkled and less flexible than the virgin material. Seam tests produced failures only at points other than the seam. Results from the tear tests, tensile and elongation tests, and seam tests are given in Table 4. Elongation declined with increasing exposure temperature, while other properties remained stable.

When PVC was stressed over a rocky subgrade, the failure mode was ripping at 63 to 70 m of water head for all samples except the sample exposed at 78°C. This sample failed at 42 m of water head when a crack developed on a fold in the liner.

TABLE 4. Physical Properties of Virgin and Aged PVC

	Tensile Breaking, Factor, kN/m(a)	E_F (strain at failure), %(a)	F_T* (break force), N(b)	Seam Test Breaking, Factor(a)
Virgin	10.6	309	51.9	10.8
Aged at 18°C	11.3	317	52.1	10.6
Aged at 48°C	10.7	311	51.2	10.5
Aged at 78°C	11.0	251	50.8	10.3

a) ASTM D882, 5 cm/min separation, 5 samples.
b) ASTM D1004, 5 cm/min separation, 3 samples.

The mechanisms of degradation of a PVC liner are expected to be loss of plasticizer and possibly elimination of hydrogen chloride followed by oxidation (1). Thermogravimetric analysis is being used to quantify the loss of plasticizer as a function of depth into the liner, and a method to measure chloride is being tested. At the time of this writing, the chemical analyses of the PVC samples were incomplete.

The organic carbon content increased from a background level of 11 ppm to 60 ppm in the column at 78°C. No increase was detected in the other columns. Additionally, chloride content in the leachate rose in the column at 78°C, indicating that 3.9 g of chloride had been released from the liner. Measurements of chloride released in the other two columns were inconsistent. Aging estimates will be made when plasticizer data and chloride data become available.

CONCLUSIONS

No aging reactions were detected with HDPE in these aging tests, and physical properties were not adversely changed during exposure to conditions typical of acidic uranium mill tailings ponds. Therefore we conclude that this material should be suitable for the active life of a tailings pond, based on chemical and physical stability.

Presently, there is not enough data to draw conclusions on the service life of PVC in a uranium mill tailings pond because chemical analyses are incomplete. Some chemical changes have occurred, as detected by carbon and chloride in the leachate. Physically the material was dimensionally stable during these tests. A 19% decrease in strain at failure occurred in the most aged sample. Tensile strength and tear resistance were stable. Puncture resistance declined significantly with aging, but was still as high as virgin HDPE puncture resistance. Seam strength was not affected by the leachate.

ACKNOWLEDGMENT

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Long Term Field Performance of Geomembranes—Fifteen Years' Experience

Many standard reference works on the use of geomembranes for Pond Lining comment on the adverse effects of weathering on sheeting when used as an impervious liner exposed to the elements. Particular reference is made to the effects of UV exposure causing deterioration of mechanical strength properties. It is the intention of this paper to show that Polyethylene (PE) film can be effectively UV stabilized and it is the experience of one South African producer that PE membranes produced and UV stabilized for exposed installations in South Africa in the late 1960's are still in use today. Samples of material removed from those installations do not show significant deterioration of physical properties such that these membranes are still fit for use as originally designed.

HISTORY

In South Africa prior to 1960 most liquid containment structures were lined either with clay or with concrete if clay was not readily available in the area of construction. Most of these structures had relatively small leakage rates which were not a major cause for concern at that time. However in certain industrial effluent containment projects seepage became of major concern particularly with aggressive acidic liquids which could not be contained in concrete or in areas where there was a scarcity of clay.

The earliest recorded evidence of a search for alternative lining materials in Southern Africa is in the Brine Salt Industry in South West Africa (Namibia) and the Eastern Cape where large evaporation ponds needed to be made impermeable in the area close to the source of natural brine but where there was no clay available and concrete was both too expensive and unsuitable for use because of the corrosive effect of the concentrates. The Sal Nova Salt Works of Port Elizabeth record having used imported 50 micron (2 mil) clear PE sheeting joined by adhesive tape to provide an impermeable barrier in their evaporation pans in 1963. Supplies of black UV stabilized material ex South African manufacture were made to the Jacobsdal Salt Works in 1967. The plastic membrane in both installations was quickly covered by salt on the base and earth on the embankments so little UV exposure took place. However it is of interest to record that both these facilities are still in operation and samples of membranes recently excavated show little or no physical deterioration.

The early 60's was also a period of growing concern in agricultural circles because it was evident that their artesian water supplies were becoming contaminated with industrial waste materials which were being dumped either in unlined impoundments or in leaking concrete or clay pits. Our firm was at that time working with both the Department of Water Affairs and with organized agriculture to develop a tough stable plastic membrane for lining earth embankment irrigation reservoirs and dams. The company developed its own UV stabilization master batch for black polyethylene sheeting which was required to withstand a minimum of 10 years exposure to the harsh UV conditions of Southern Africa.

The earliest properly recorded installations were made in 1966/67 using 250 micron (10 mil) black PE sheeting joined with a mechanical clamping device. They are mentioned here for the record only as no measurements of comparative physicals have been entered.

In 1982 our firm commissioned the Council for Scientific and Industrial Research, Pretoria, South Africa, to carry out an investigation of Synthetic Materials used in Dam Linings (Contract No. 50091248). The Company has also undertaken photographic recording and material measurement from some of its earlier installations. Case studies of those installations follow. The conclusions drawn from these two studies (as reported in the CSIR report) are:

A. There was no serious deterioration of properties of the PE film as a function of time during the time period covered by the dams in this report (since 1968).
B. Membranes that have been protected e.g. buried membranes show hardly any change in properties.
C. No attack by inorganic or biological effluents on the PE could be detected. In the only measured sample of membrane exposed to organic effluent no change could be detected.
D. PE membranes are vulnerable to mechanical and fire damage where exposed and protective measures such as fencing or sacrifical layer covering are recommended.

As is often the case with exercises of this kind care should be exercised when comparing the physical properties of in service membranes with those of control specimens as there are factors such as localized mechanical damage to the surface that can influence results significantly. Furthermore the control material may differ from the original quality of material installed, or the sample taken from a dam may be non-representative of the lining as a whole. Where "Original" properties are quoted these are the properties as specified for its PE membrane manufactured in South Africa.

However, the major conclusion which must surely be drawn from this exercise, whatever the test measurements show, is that all the installations examined are still serviceable with only mechanical damage from non-weather sources showing any visual evidence of any deterioration.

TEST METHODS

Where applicable tests in accordance with SABS 952 (c) of 1969 were used unless otherwise shown. Tests recorded in CSIR report 50091248 of June 1982 are referenced "CSIR", all other tests were carried out by Gundle R & D Laboratories.

a) Tensile Properties - testing speed was 500 mm/min as it was felt that material defects due to degradation were more likely to show up in high speed rather than low speed testing. Elongation is given as total jaw movement in mm with initial jaw separation 65 mm.
b) Tear properties measured in accordance with ASTM-1004 and D1938. Some samples show marked differences between MD and TD which may be caused by crystalline orientation with age or may be due to manufacturing orientation. UV exposure does not appear to be a contributing factor.
c) Puncture resistance and drop impact strength - two tests were carried out - the needle puncture resistance as per SABS 952 and impact puncture resistance using a 32 mm dia. falling dart at 23°C and 0°C (to simulate hail).

CASE STUDY I
Mrs. Kreutzer, Bryanston. A private house fish pond is the application. The liner was installed in 1968 and made of 250 micron (10 mil) black. It was a single width (9 m) of material so no joints were required. It was damaged during cleaning operations in 1983, so it is being relined by Gundle. Samples of materials were taken in 1983 from the exposed portion of the liner.
Original Specification: A
Gundle Test Results: B

TABLE 1

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	10.75	180	10.0	N/A	77.26

CASE STUDY II
Saaiplaas Gold Mine. This application is an effluent dam constructed in 1968. It included an impervious core membrane in the wall of the dam buried since construction. It showed no sign of mechanical damage and no evidence of leakage. It was made of 500 micron (20 mil) black.
Original Specification: A
Buried Sample: B (CSIR Tests)

TABLE 2

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	11.1	262	18.3	110	99

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25	42
B	22.2	23.3	44

CASE STUDY III
Rustenburg Platinum Mines. Effluent dams were constructed in 1970 and 1973 containing sodium sulphate and nickel salts in solution pH7.5. Minor mechanical damage from tractor and pedestrian traffic was observed. The applications used 550 (20 mil) micron PE with Stripseal mechanical joints.
Original Specification: A
Exposed since 1970: B
Exposed since 1973: C
Exposed since 1973: D
Buried since 1973: E

TABLE 3

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	10.9	237	15.3	---	90
C	12.3	239	16.5	92	104
D	13.8	201	14.4	108	107
E	10.8	274	16.6	106	103

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25	42
B	---	23.3	48
C	---	21.2	50
D	---	23.0	48
E	---	19.7	48

CASE STUDY IV

Ucar Minerals Pretoria. An Ore tailings dump liner was installed in 1976 and an effluent dam liner was installed in 1978. Both liners have been fully exposed since installation and contain acidic residues in solution pH4. Some mechanical damage was evident around the edge of the tailings dump. The effluent pond is protected by a sacrificial layer and exposed samples were taken from the top layer. This application used 500 micron (20 mil) PE with mechanical joints.

Original Specification: A
Exposed since 1976: B (CSIR Tests)
Exposed since 1976: C

TABLE 4

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	11.7	255	16.5	74	97
C	12.4	240	18.2	116	105

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25	42
B	---	20	49
C	---	22	45

CASE STUDY V

Stilfontein Gold Mine. This application is a sewage oxidation and maturation pond, installed in 1968. It is made with a permanently exposed PE 500 micron (20 mil) joined using the membrane. Installation is well fenced and there is no mechanical damage to the membrane which still appears in perfect condition. Samples of membrane were taken late 1982 from two positions fully exposed and below the normal sewage level. (CSIR Report).

Original Specification: A
Permanently Exposed: B
Below Sewage Level: C

TABLE 5

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	12.8	162	10.7	97	97
C	13.3	209	12.8	105	99

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25.0	42
B	—	18.4	48
C	—	17.1	48

CASE STUDY VI

Omnia Phosphates, Rustenburg, Fertilizer Company. This application is a gypsum slimes dump and water collection canals. It was first lined in 1972 and extensions were made to the old lining in 1975. Further new dump lining and canals were installed in 1981. The original installations were made using 500 micron (20 mil) PE film with Stripseal mechanical joint. Samples from the exposed edge of the 1972 installation were taken for measurement by Gundle in 1983. The 1981 installation was made of a membrane with extrusion welded joints.

Original Specification: A
Exposed since 1972: B

TABLE 6

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	10.4	160	10.5	—	80

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25.0	42
B	—	—	—

CASE STUDY VII

Triomf Fertilizers, Chloorkop, near Johannesburg. The original effluent dam was constructed in 1969 and lined with 500 micron PE membrane. Joints were mechanical Stripseal plastic clamps buried in trenches. During cleaning operations in 1974 the original membrane was damaged and new 500 micron (20 mil) PE was installed. Samples of the old membrane were extracted and measured (CSIR Report). The effluent in the dam contains 1.6 per cent phosphoric acid and 3000 ppm fluorides.

Apart from the mechanical damage the appearance of the exposed membrane from the edge of the old dam is excellent.

Original Specification: A
Permanently Exposed 1969: B
Partly Buried 1969: C
Partly Exposed 1974: D

TABLE 7

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	11.1	184	10.7	105	86
C	11.1	200	11.1	96	86
D	11.6	200	11.1	74	97

	Drop 0°C J/mm	Drop 23°C J/mm	Puncture N/mm
A	26.4	25.0	42
B	—	17.6	48
C	—	19.5	45
D	—	17.3	46

CASE STUDY VIII

Swazi Spa Hotel Complex, Swaziland. In 1966 a water storage pond of approximately 5000 m2 250 micron (10 mil) black PE was installed to supply water for the Swazi Spa Hotel Complex. The periphery of the pond is permanently exposed to the weather and high UV for its lifetime. There has been a certain amount of sediment build-up in the dam which has resulted in weed growth in the dam. This has, however not reduced the effectiveness of the lining and the dam is still operational.

Samples taken - September 1983, were tested by Gundle.

Original Specification: A
Exposed 1966: B

TABLE 8

	Yield MPa	Elongation mm	Break MPa	Tear Prop N/mm	Init. Tear N/mm
A	11.4	311	19.3	100	103
B	10.94	277	11.09	—	91.6

DISCUSSION OF RESULTS

While all of the membranes in this study were still providing satisfactory service, there was an expected reduction in some of the physical properties.

In almost every instance, there was a decline in elongation. This is indicative of a slight stiffening of the material with age. For exposed samples, the decrease in elongation varied from 23% for Case IV, in service for 8 years, up to 48% for Case VI which has been in use for 12 years. With such a limited data base, it is difficult to draw firm conclusions from the elongation data. However, it is safe to say that, in general, elongation decreased with increasing service time. Also, membranes which had lost up to 48% elongation were still providing satisfactory service.

Case's II and III compare results from exposed and buried membranes. In Case II, elongation decreased by 15.7% after 16 years of service. Case III's buried elongation was 11.8% of original verses 35.4% for the exposed portion. Obviously, protection of the membrane from the environment allows it to retain a higher percentage of its original properties.

The ultimate strength of the material is another important factor in assessing membrane performance. Inspection of the data indicated a close correlation between decrease in elongation and breaking strength. Again, buried membranes retained a higher percentage of the original values than exposed membranes. It is difficult to access tear propogation (tear prop) and tear initiation results. There was no apparent correlation between these properties and length of service. However, in overview all values were within acceptable tolerances.

At first analysis, results from the drop impact test and puncture resistance test don't seem to make sense. One would expect the puncture resistance to increase with time due to the reduction in elongation and corresponding increase in stiffness. This would make the material slightly harder and therefore give a higher value. Conversely, the drop impact test shows a decrease in properties despite the fact that the material is harder. The reason for this lies in the methods used for the drop impact test. As explained earlier in this paper, the drop impact test uses a falling dart which impacts the surface of the material. This tends to mitigate the impact of material stiffness. On the other hand, the puncture resistance test applies uniform pressure with a less angular probe and thus material stiffness comes into play. Nevertheless, both dart impact and puncture resistance were within acceptable tolerances.

MATERIALS - THE NEXT GENERATION

Results of these studies conclusively show that polyethylene based materials are capable of providing acceptable service for extended periods. Most of the membranes tested were relatively thin films, 250-500mm (10-20 mils). As evidenced in some of the case histories, mechanical damage was experienced on several sites. For exposed membranes, this problem can be overcome by increasing membrane thickness. Recent EPA regulations require the use of a minimum 30 mil thickness for all geomembranes. Many designers of exposed membranes are specifying rugged, heavy duty membranes in the thickness average of 1500-2000mm (60-100 mils). The choice of thickness depends on anticipated mechanical abuse, particularly for exposed materials.

The other key to successful long term membrane performance lies in the formulation of the stabilizer package. For PE membranes, this normally represents 2.0-2.5% of the total formulation. It is critical that proper stabilizers be incorporated into the formula and also that they be correctly dispersed as this will significantly affect membrane performance.

SUMMARY

In summary, the following conclusions can be drawn from this study:

1. Geomembranes made from polyethylene have been successfully used in exposed conditions for up to 16 years.
2. Declines in elongation and breaking strength up to 48% were observed without having a negative effect on the functionality of the geomembrane.
3. Buried membranes showed smaller decline in properties when compared to exposed membranes which is to be expected.
4. Thin film geomembranes are subject to mechanical damage. This can be overcome by increasing thickness.
5. Incorporation of the correct stabilizer package in the formulation is critical to long term performance.

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Aging of PVC Geomembranes in Uranium Mine Tailing Ponds

Approximately forty samples of PVC geomembranes were removed from nine large ponds containing acid and four smaller reservoirs containing acid or water. These ponds and reservoirs are part of a large uranium ore treatment facility located in a desert area with an extremely hot and dry climate. Chemical and mechanical tests show the development of ageing for exposure duration between zero and 45 months. The influence of liquid in contact (acid or water) is evaluated. A criterion for acceptability of aged PVC geomembranes is proposed.

Nine large evaporation ponds (ponds 1 through 9), with a total surface area of approximately 700 000 m^2 (7 million sq. ft.) were built and lined with a PVC geomembrane between 1978 and 1981. These ponds contain uranium tailings and sulfuric acid. The PVC geomembrane is not protected by a cover material and is therefore directly exposed to the sulfuric acid, and, above liquid level, to the weather. Climatic conditions are particularly harsh since these ponds are located in one of the hottest and dryest parts of the world.

In 1981 excessive shrinkage resulting from ageing of the PVC geomembrane caused a few tears (Fig. 1). A systematic study was undertaken to assess the state of ageing of the geomembrane lining the various ponds. The fact that the ponds were constructed and lined at different times between 1978 and 1981 provided an opportunity for evaluating the influence of elapsed time (up to 45 months) in the development of ageing. Samples were taken in all nine ponds (and in four small reservoirs in the same complex) and laboratory tests were conducted.

Additional observations were made on PVC geomembranes subjected to extreme conditions (acid spray) in ore treatment pads (Figs. 2 and 3) located in the same complex.

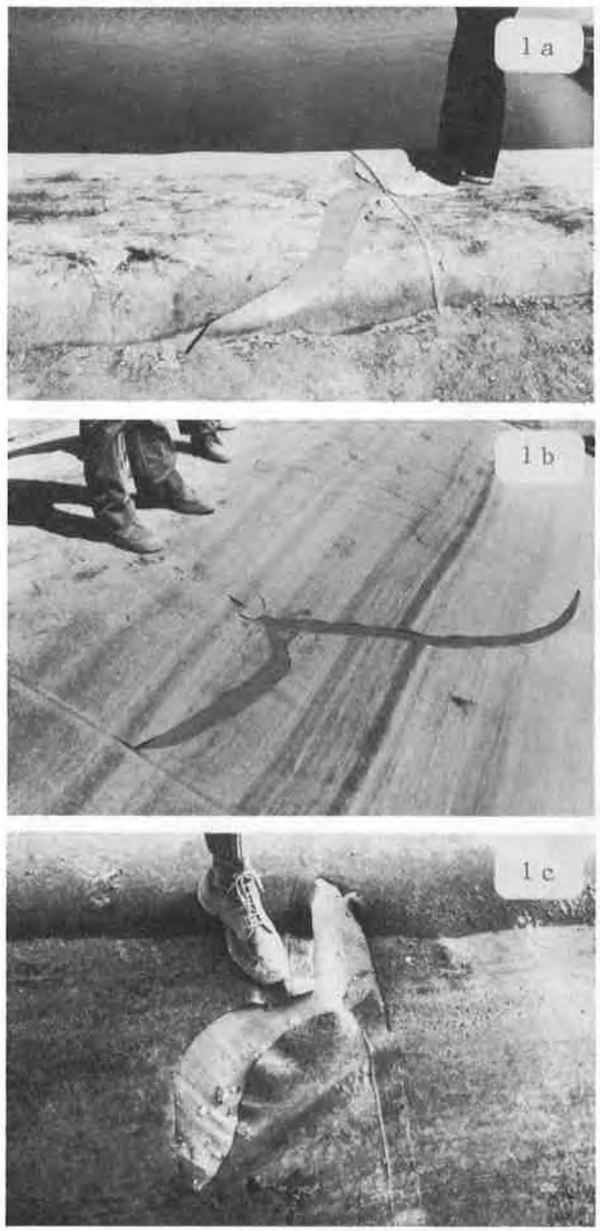

Fig. 1 Examples of tear caused by shrinkage on reservoir slopes. In cases a and b, the tear did not cross the seam. In case c, the tear crossed the seam, but the seam did not fail. In all three cases the geotextile located under the geomembrane is visible.

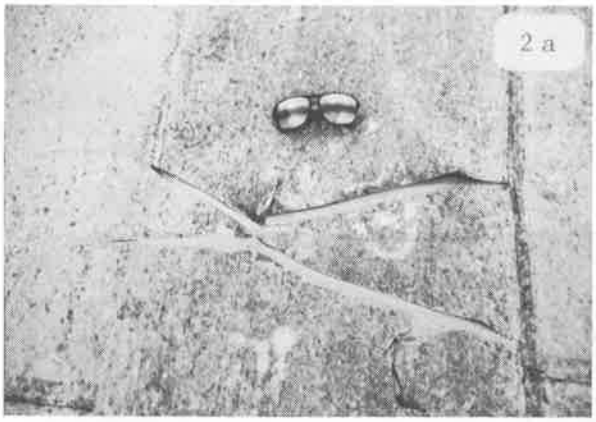

Fig. 2 Final stages of deterioration of a PVC geomembrane after it became stiff as a result of loss of pasticizers due to exposure to sun and sulfuric acid spray from an ore treatment pad. Seams appear to be the strongest areas of the geomembrane.

Fig. 3 Wrinkles in seams in a small dike exposed to sun and acid spray next to an ore treatment pad. The geomembrane had shrunk as a result of loss of plasticizers. However, loss of plasticizer is slower, and shrinkage is less, in seams because thickness is twofold.

1 DATA

1.1 Climate

Data on climate is given in Table 1. This climate is extremely hot. Temperatures of up to 47^{0}C (117^{0}F) are frequently recorded but, probably as a result of the evaporation, the atmosphere seems slightly cooler in the pond area than elsewhere at the site. Precipitation is close to zero and there is almost no vegetation. The sky is almost always clear. As a result, solar energy received at ground level is of the order of 200 kcal/(cm^2.year), which is approximately twice the European figure. During the day, the geomembrane temperature reaches approximately 80^{0}C (180^{0}F) and during liner installation it was impossible to handle the geomembrane after 9 in the morning in summer. Between March and October, there are typically fifty sandstorms, some with high winds.

1.2 Composition of Liquid

Liquid composition provided by the owner of the facility is as follows: (i) amount of salts close to 100 g/liter (sodium sulfate, magnesium, iron, aluminum,etc.); (ii) 10 to 15 g/liter maximum of free sulfuric acid (pH 1.5 to 2); (iii) traces of kerosene (up to 1 liter/m^3); and (4) traces of nitric acid (up to 0.1 g/liter).

A test conducted on a sample taken in pond 1 gave the following results (in g/liter): Iron - 2.8; Magnesium - 1; Aluminum - 2.2; Sodium - 3.8; Calcium - 0.5; Manganese - 0.4; SO_4 -- (sulfates) - 48; Acidity expressed in terms of SO_4H_2- 6.5.

It seems that acid concentration in the sample tested is half acid concentration announced by the owner. Kerosene was mostly present in ponds 1 and 3, at or near the surface.

All nine large ponds were filled with the acidic liquid described above. PVC samples were also taken from four small reservoirs containing respectively: (i) acidic liquid similar to the above mentioned liquid, although with a smaller acid concentration; (ii) industrial water; (iii) waste water; and (iv) potable water.

1.3 Samples

The thirteen ponds involved in this study were lined with PVC geomembranes made by the same manufacturer.

In July 1981, November 1981 and March 1982, approximately forty samples were taken from the thirteen ponds mentioned in the introduction. These samples were approximately 20 cm x 20 cm (8 in. x 8 in.). Samples were selected in the following way to evaluate the influence of different parameters:

- Samples were taken from all large ponds (ponds 1 through 9), to evaluate the influence of elapsed time on ageing, since each of the nine large ponds was constructed at a different time.
- Samples were taken from three small water reservoirs to compare the ageing of these samples with the ageing of samples in contact with acid.
- Samples of 0.5 mm (20 mil) thick PVC geomembrane were taken from bottom of pond 5, and from two water reservoirs, to compare results with results related to other samples, all 1mm (40 mil) thick, in order to evaluate the influence of thickness on ageing.
- Samples were taken from the anchor trench and at 10 different levels along the slope of pond 5 (Fig. 4) and at two or three levels in several other ponds, to compare ageing of submerged and exposed portions of the geomembrane.

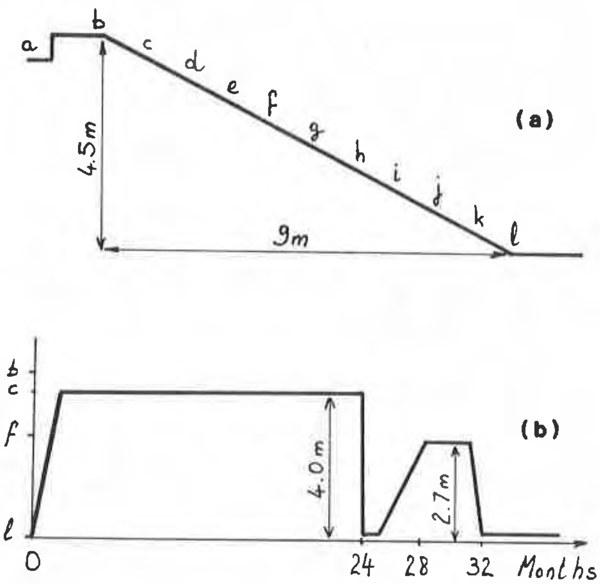

Fig. 4 Study of the influence of immersion on ageing: (a) Cross section of dike of pond 5 showing locations where samples were taken; (b) Curve of level of liquid in pond 5.

Table 1 Data on climate: 1980 was a typical year except for precipitation (average is 35 mm per year).

MONTH OF YEAR 1980		J	F	M	A	M	J	J	A	S	O	N	D
TEMPERATURES (°C)	Average monthly maxima	29.8	30.7	34.8	40.3	43.1	43.3	41.1	41.4	40.0	38.9	34.5	27.5
	Average for the month	20.9	21.8	25.3	30.6	33.3	34.4	33.4	32.9	31.5	30.2	24.9	18.4
	Average monthly minima	12.0	12.8	15.7	20.9	26.7	25.5	25.6	24.3	23.0	21.4	15.3	9.2
PRECIPITATION (mm)	Total for the month	0	0	0	0	0	9.0	12.6	20.3	38.0	0	0	0
DIRECT SUNLIGHT (hours/day)	Average for the month	9.0	8.9	9.7	9.2	6.7	5.9	8.7	9.2	9.8	9.5	9.2	9.1
EVAPORATION (mm/day)	Average for the month	9.2	9.2	12.0	12.6	15.2	14.1	12.2	11.6	11.6	11.5	9.1	6.6

2 TESTS

2.1 Chemical Tests

A PVC geomembrane is made from PVC resin associated with plasticizers and a few other additives including stabilizers and carbon black. Plasticizers are used to impart flexibility to the stiff PVC resin. Stabilizers and carbon black are used to slow down outdoor ageing. Chemical tests were conducted on plasticizers, stabilizers, and resin as discussed below.

Plasticizer Analysis. Although all PVC geomembranes came from the same manufacturer, it was suspected that a change in plasticizer supply had occured in September 1978. To identify the plasticizers used, mass spectroscopy tests were conducted on samples from ponds constructed prior to September 1978 and on samples from ponds constructed after this date. A mass spectroscopy diagram on a sample from pond 1 is shown in Fig. 5a. Comparing similar diagrams obtained on all samples showed that indeed compositions of plasticizers before and after September 1978 were different. However, a mathematical treatment of the results of gas chromatography diagrams (Fig. 5b) giving percentages of plasticizer constituents showed that the average molecular weight of the various constituents of the plasticizers is almost exactly the same in both cases. Since volatility of a plasticizer is linked to its molecular weight (the heavier, the less volatile), it was concluded that all geomembranes used in the ponds could be considered equal from the point of view of plasticizer volatility, which appeared to be the key factor governing ageing, as discussed in section 2.3.

The analysis of plasticizers also showed that the plasticizers used were linear phtalates, which are significantly more stable than the non-linear phtalates typically used a decade or more ago. This clearly means that the rapid ageing discussed in this paper results only from exceptional climatic conditions and not from a poor quality geomembrane.

Plasticizer Loss. A PVC geomembrane just off the production line usually has a plasticizer content of approximately 35%. This percentage (which is the one usually quoted) is related to the weight of the geomembrane, i.e., including the weight of the plasticizer itself. During the process of plasticizer loss, both the weight of plasticizer and the weight of the geomembrane decrease, while the weight of resin remains constant. Therefore, it is more appropriate to use percentages related to weight of resin to express plasticizer loss. The percentages of plasticizer related to resin, Pr, and to geomembrane, Pg, are related by:

$$Pr = Pg/(1 - Pg) \quad \text{and} \quad Pg = Pr/(1 + Pr) \qquad (1)$$

For example, Pg = 35% is equivalent to Pr = 54%. Hereafter, the plasticizer loss is expressed by the relative decrease of Pr, i.e., $\Delta Pr/Pr$.

Samples from all ponds and reservoirs were subjected to plasticizer extraction by distillation using CCl_4 - MeOH. Results are presented in Fig. 6.

Stabilizer Loss. Tests were conducted on seven samples to evaluate the stability of stearate stabilizers. The type of test conducted does not provide the amount of stearate in the sample but only the time necessary to eliminate stearates present in the sample. Comparing the times allows for a ranking of the samples. Results given in Table 2 show that ranking of samples according to stabilizer presence or plasticizer presence is the same. As the geomembrane ages, stabilizers as well as plasticizers are progressively lost.

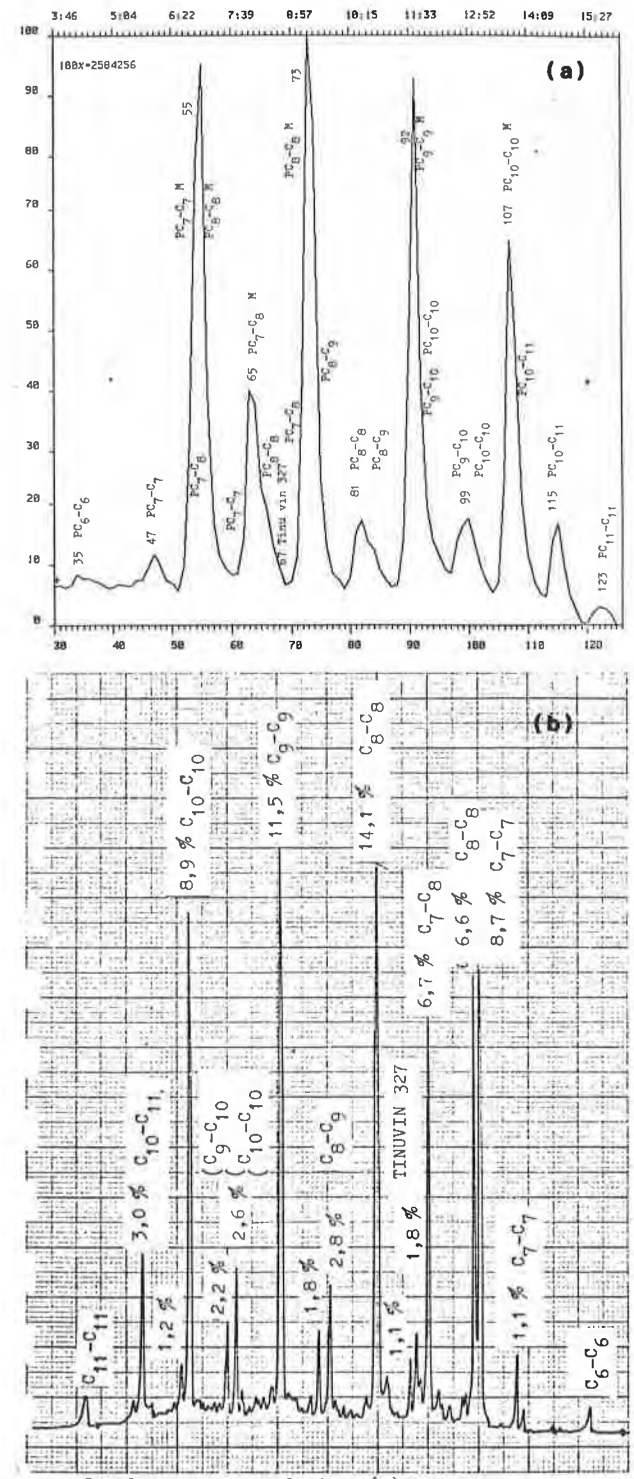

Fig. 5 Plasticizer analysis: (a) Mass spectrometry diagram on a sample from pond 1. Each peak is related to a constituent of the plasticizer. (b) Gas chromatography of the same sample. The area under each peak is approximately proportional to the percentage of this constituent.

<u>Resin Analysis</u>. Almost identical molecular weights were obtained for a control sample and for samples exhibiting a plasticizer loss of 20%, 29.6% and 52.3% respectively (and, consequently a certain loss of stabilizers as discussed above). Although the test method is not precise, these results tend to indicate that the resin remains intact even after a large plasticizer and stabilizer loss.

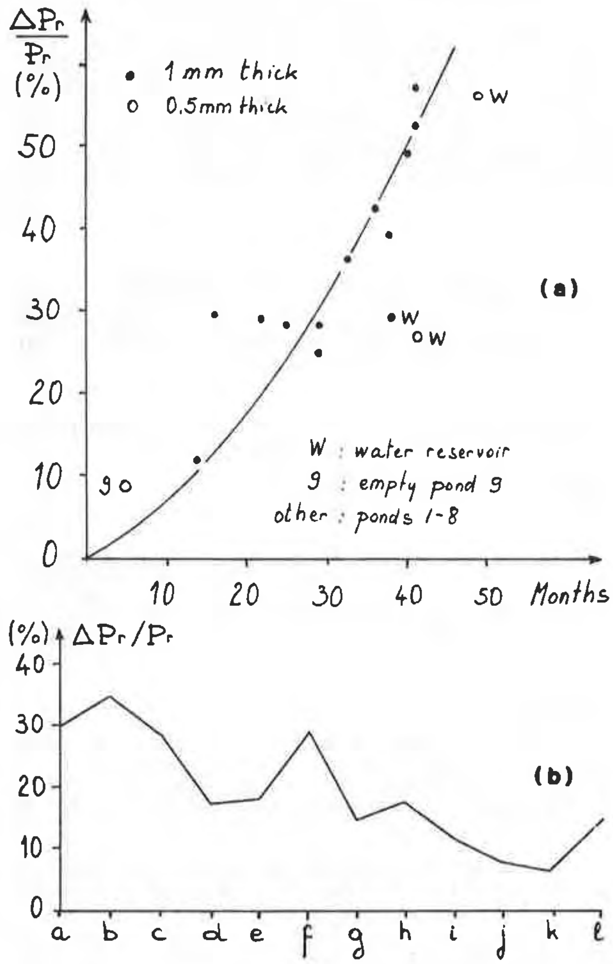

Fig. 6 Plasticizer loss: (a) as a function of elapsed time from installation date, for samples permanently exposed; (b) as a function of the position of the sample on the slope of pond 5 (see Fig. 4).

Table 2 Evaluation of stearate stabilizer stability.

Pond No. or Reservoir	Time to Extract Stearate Stabilizer (min.)	Percentage of Plasticizer Pr
Control Sample	45	52
No. 9	26	48
Industrial Water	11	37
No. 1	9	37
No. 2	8.5	37
Acid	6.5	26
No. 1	7	22

2.2 Mechanical Tests

Classical tensile tests were conducted. The geomembrane properties measured were stress and elongation at failure, and secant modulus at 100% elongation. Values of elongation at failure are presented in Fig. 7 as a function of the plasticizer loss. A clear relationship appears: the elongation at failure decreases when the plasticizer loss increases. The decrease in elongation at failure is particularly marked when the plasticizer loss exceeds 30%. The simple relationship between elongation at failure and plasticizer loss represents well the progressive deterioration and increasing stiffness of the geomembrane. Relationships between stress at failure or modulus and plasticizer loss are not simple: the stress at failure and the modulus first tend to decrease when plasticizer loss increases from 0 to 10% approximately, then they both increase as a result of increasing stiffness of the geomembrane.

In addition, a few tests conducted on samples with seams showed that seams were not affected by ageing of the geomembrane, which is in good agreement with observations (Figs. 1 and 2).

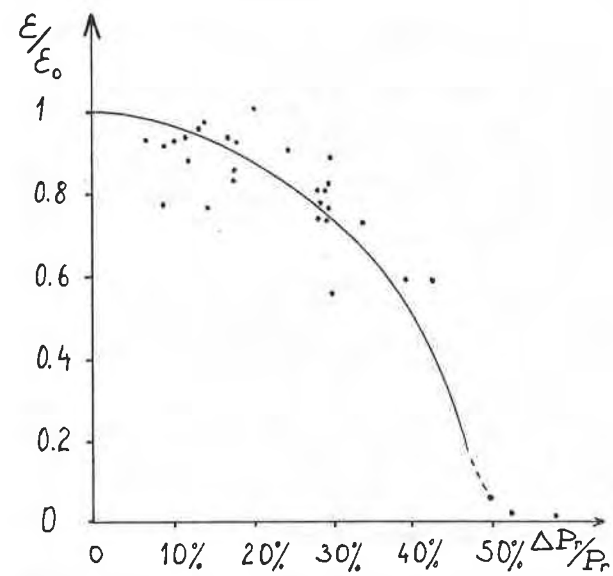

Fig. 7 Relative elongation at failure as a function of the plasticizer loss (ε is the elongation at failure of the sample exhibiting plasticizer loss and ε_0 is the elongation at failure of a control sample).

2.3 Test Result Interpretation

Chemical tests (section 1.1) suggest that the PVC resin is not affected even after significant plasticizer and stabilizer loss. The main change undergone by the PVC geomembrane with time is the progressive plasticizer loss shown in Fig. 6a. The progressive increase of the geomembrane stiffness observed at the site is clearly linked to the plasticizer loss as shown in Fig. 7. Therefore, factors influencing plasticizer loss should be discussed.

Plasticizers can leave the geomembrane either by volatility, i.e., evaporation under the action of heat, or can be extracted by liquids or soil in contact.

Influence of Heat. Fig. 6b shows that plasticizer loss is less for geomembranes protected during a certain period of time from sun-generated heat by liquid. Also, plasticizer loss is less in the anchor trench, where the geomembrane is protected by earth, than on the crest, where the geomembrane is exposed. Plasticizer loss is less under the liquid than in the anchor trench, possibly due to a lesser temperature in the liquid (however, tests on samples taken from anchor trenches in two other ponds showed plasticizer losses equivalent to that of submerged samples). The rapid ageing of the sample taken at the bottom of pond 9 (Fig. 6a) can be explained by the fact that pond 9 had never been filled when the sample was taken. The temperature in the bottom of an empty pond is higher than at the edge of a pond filled with liquid, as mentioned in section 1.1. Also, observations made at ore treatment pads (Figs. 2 and 3) located far from any body of liquid, showed that ageing of the geomembrane was much faster than in the ponds. Typically a PVC geomembrane would become brittle after one year at ore treatment pads as opposed to three years in ponds.

In conclusion, Fig. 6 and observations show that the action of sun-generated heat with time is the main factor governing ageing of the geomembrane.

Influence of Liquid. Fig. 6a shows that samples taken just above water level in three water reservoirs exhibit less plasticizer loss after a given period of time than samples taken above acid level in the ponds. Also, the three peaks of the curve in Fig. 6b appear at levels where the acid has stayed for long periods of time. As mentioned in section 1.1, the ponds are in a windy area and waves are frequent. As a result, the geomembrane just above liquid level is leached by waves and, at the same time, its temperature is very high because of sun exposure.

Acceleration of plasticizer loss by acid also appears in ore treatment pads where the geomembrane is exposed to acid spray. As mentioned above, apparent ageing of the geomembrane is approximately three times faster in ore treatment pads than in ponds.

In conclusion, a PVC geomembrane, heated by the sun, loses its plasticizers more rapidly if it is sprayed with acid or leached by waves of acidic liquid. On the other hand, total immersion in acid is more beneficial than detrimental, since submerged portions of geomembrane, being protected from direct sun exposure, lose less plasticizers than exposed portions.

No influence of kerosene (of which there were traces in a few ponds) on ageing has been identified.

Influence of Soil in Contact. Ageing of the geomembrane in anchor trenches can be explained by the combined action of plasticizer evaporation caused by heat and plasticizer absorption by the dry clayey soil used to backfill the anchor trench (just as a spot remover powder absorbs grease from cloth). However, extraction of plasticizers by soil appears to be much less significant than evaporation caused by sun-generated heat because no difference in plasticizer loss was identified between: (i) exposed samples in contact with the clayey soil used to construct the dikes (in six ponds); and (ii) exposed samples separated from the clayey soil by a geotextile (in three ponds).

Influence of Geomembrane Thickness. Most samples tested were 1 mm (40 mil) thick. Only four 0.5 mm (20 mil) thick samples were tested. Results related to three of these samples are presented in Fig. 6a. The fourth one was covered with tailings and exhibited about one third the loss of plasticizers it would have had if exposed. No conclusion can be drawn from the results shown in Fig 6a because of the different conditions to which these samples were exposed: water for two of them, and high heat in an empty reservoir for the other.

2.4 Geomembrane Evaluation

To perform satisfactorily, a PVC geomembrane must not become stiff as a result of ageing. Fig. 7 shows a clear relationship between stiffness and plasticizer loss. The discussion presented above shows that plasticizer loss is the key factor governing ageing. Consequently, the criteria presented in Table 3 have been proposed to evaluate the geomembrane in the various ponds, and the curve presented in Fig. 6a is used to predict time when maintenance or repair will become necessary depending on the date of construction of each pond.

Table 3 Proposed PVC geomembrane evaluation as a function of plasticizer loss.(*) Note: Weight loss and plasticizer content have been calculated from plasticizer loss with an initial plasticizer content of 35%.

Plasticizer Loss	Weight Loss (*)	Plasticizer Content (*)		Evaluation
$\Delta Pr/Pr$ (%)	$\Delta W/W$ (%)	Pg (%)	Pr (%)	
0	0	35	53.8	Intact
0 - 10	0 - 3.5	32.6 - 35	48.5 - 53.8	Excellent
10 - 20	3.5 - 7.0	30.1 - 32.6	43.0 - 48.5	Good
20 - 30	7.0 - 10.5	27.3 - 30.1	37.7 - 43.0	Fair
>30	>10.5	<27.3	<37.7	Unacceptable

CONCLUSION

As shown by chemical analyses, the PVC geomembrane used in the ponds discussed in this paper was of good quality. Therefore, the study presented is a representative outdoor-ageing test, accelerated by an exceptional climate and, to some extent, by acid.

The study shows that the appropriate parameter to use in evaluating the progress of ageing is plasticizer loss and the appropriate parameter to use in evaluating consequences of plasticizer loss is elongation at failure. The study shows that there is a clear relationship between plasticizer loss and elongation at failure which is related to flexibility of the geomembrane.

To ensure that a PVC geomembrane, under any climate, has the required flexibility for satisfactory performance, the criterion proposed in Table 3 can be used.

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An Assessment of HDPE Liner Durability: A Report on Selected Installations

"How long will it last?" is one of the most obvious and pressing questions asked about any geomembrane proposed for a lining system, yet it is one of the most difficult to answer with exactitude. Not only are the influences on the performance of a geomembrane almost too many to predict with certainty, but more importantly, the experiential base with many such materials in such applications is at this time still limited.

This paper is intended as a contribution to that increasing base of experience by reporting the results of tensile testing of samples of a high density polyethylene geomembrane, taken from an actual in-service installation and from exposed test specimens. This data is augmented by similar but unpublished research from West Germany.

MINIMUM VALUES

The high density polyethylene geomembrane that is discussed throughout this paper is SCHLEGEL[R] sheet, manufactured and installed by Schlegel Lining Technology, Inc., Houston, Texas. The particular resin used in the manufacture of the geomembrane has a relatively high molecular weight and a narrow molecular weight distribution. The geomembrane contains two percent carbon black (by weight) as stabilization against UV attack. The liner does not contain plasticizers or other additives. Minimum properties for this geomembrane are given in Table I.

TABLE I
Minimum Specifications For Schlegel® Sheet Polyethylene

PROPERTY	TEST METHOD	VALUE
Density	ASTM D792	0.930 gm/cc
Tensile strength @ yield	ASTM D638	1500 psi
Tensile strength @ break	ASTM D638	1500 psi
Elongation @ yield	ASTM D638	10%
Elongation @ break	ASTM D638	500%
Stress crack	ASTM D1693	> 500 hours
Low temperature	ASTM D746	- 40 °C
120 day soil burial	ASTM D3083	± 10% of orignial tensile
Bonded seam strength	ASTM D3083	90% of material breaking factor
Dimensional stability	ASTM D1204	± 3%

CASE #1 - NEW MEXICO

Sets of samples were obtained from a process waste evaporation pond, installed in Farmington, New Mexico for Public Service Company of New Mexico. The contents of the pond from which the samples were taken are as follows:

Ca	314 mg/L	
Mg	619 mg/L	
SiO_2	200 mg/L	
SO_4	164,842 mg/L	
Cl	13,605 mg/L	
SO_3	1,342 mg/L	
Na/K	86,114 mg/L	
HCO_3	150 mg/L	
T.D.S.	266,928 mg/L	(about 25% solids)
pH	6.0	

Installation took place in early 1981; the samples were taken in December 1983.

The quality control program employed by the manufacturer/installer permits a comparison of values for samples from the installation with earlier records for the identical roll of material as it came from the production line. Thus, for example, samples from roll #8476 would undergo a series of tensile tests after it had been produced, but before it was shipped. Field recordkeeping is sucn that any roll of geomembrane can be identified in the "as built" drawings by number.

Similar records are kept for welded seam samples. The joining technique used by the installer is extrusion welding. In this process, liner panels are overlapped, preheated, and then a bead of molten extrudate, of the same resin as the parent sheet, is introduced into the overlap area. The two sections of liner are then immediately compressed using a pressure roller system. Seam samples are cut in the field at regular specified intervals, checked in the field for bond strength and then forwarded to the installer's laboratory for determination of tensile values and corroboration of bond strength. All samples are retained after they have been tested. It is these samples that are here compared to the seam samples obtained in 1983.

It is also worth noting that the liner installed in 1981 was 2.0mm thick (80 mil).

All testing for physical properties was performed in accordance with ASTM D638, except where noted. The results are shown below in Tables II, III, IV and V.

TABLE II
Comparison Of Values For Samples From Below Effluent Level

	Values Before Service (Original Values – Averaged From 10 Samples)	Values After 3 Years In Service – Samples From Below Effluent Level	% Deviation From Original Value
Tensile Yield Strength	3304 psi	3285 psi	-0.57
Elongation at Yield	17.5%	20%	+ 14.00
Tensile Break Strength	4176 psi	4152 psi	-0.55
Elongation at Break	795%	858%	+ 7.92
Tear Resistance	82 lb-f (ASTM D1004)	81.3 lb-f	-0.76
Environmental Stress Crack	> 1000 hours (ASTM D1693)	no failures to date	
Carbon Content	2.38% (ASTM D1603)	2.378%	-0.08
Tensile Impact Resistance	500 mJ/mm² (ASTM D1822)	488 mJ/mm²	-2.40

TABLE III
Comparison Of Values For Samples From Above Effluent Level

	Values Before Service (Original Values – Averaged From 10 Samples)	Values After 3 Years In Service – Samples From Above Effluent Level	% Deviation From Original Value
Tensile Yield Strength	3304 psi	3288 psi	-0.48
Elongation at Yield	17.5%	17.3%	-1.14
Tensile Break Strength	4176 psi	4170 psi	-0.12
Elongation at Break	795%	760%	-4.40
Tear Resistance	82 lb-f (ASTM D1004)	81.8 lb-f	-0.2
Environmental Stress Crack	> 1000 hours (ASTM D1693)	no failures to date	
Carbon Content	2.38% (ASTM D1603)	2.362%	-0.75

TABLE IV
Comparison Of Values For Welded Seam Area Below Effluent Level

	Original Values Before Service (Averaged from 10 Samples)	Values After 3 Years In Service – Samples From Below Effluent Level	% Deviation From Original Value
Weld Tensile Strength	2965 psi	2959 psi	-0.20
% Bond Strength	> 100%	> 100%	——
Weld Peel Value	2640 psi	2608 psi	-1.20
% Bond Strength	> 100%	> 100%	——

TABLE V
Comparison Of Values For Welded Seam Area Above Effluent Level

	Original Values Before Service (Averaged from 10 Samples)	Values After 3 Years In Service – Samples From Above Effluent Level	% Deviation From Original Value
Weld Tensile Strength	2965 psi	2960 psi	-0.16
% Bond Strength	> 100%	> 100%	——
Weld Peel Value	2640 psi	2610 psi	+ 0.07
% Bond Strength	> 100%	> 100%	——

All values recorded would seem to indicate that the liner and the seam areas have not undergone noticeable degradation. The material above the effluent level, directly exposed to ultraviolet radiation, does not show any significant loss in mechanical properties. The presence of acceptable carbon content, even after three years exposure in a geographic region with high sun loadings is an indication of proper carbon black dispersion at the time of liner manufacture.

After three years, the loss in tensile properties for the liner samples exposed to effluent is -0.57% T_Y (tensile yield strength). In order to extrapolate from this deviation so as to obtain an estimated service life for the liner in this application and this environment, the assumption is made that the material exhibits equal loss every three years. And thus:

$$0.57\% \text{ of } 3304 \text{ psi} = 18.83 \text{ psi}$$

If 18.83 psi is lost every three years, then the loss over 20 years is:

$$\frac{20}{3} \times 18.83 \text{ psi} = 125.5 \text{ psi}$$

And thus the tensile yield value for the liner after 20 years is 3278.5 psi, well above the typical value of 2500 psi and the minimally acceptable value of 1500 psi. Similar calculations can be performed for all the physical properties and deviations listed, with all indicating an expected service life exceeding 20 years.

CASE #2 - WARREN, PENNSYLVANIA

A 10 x 200m test roll of 2.5mm (100 mil) HDPE liner was placed over an existing asphalt liner in early 1982 in order to assess the cold weathering capability of the HDPE geomembrane. The thickness for the test roll was selected because of its previous performance record in installations where cold weather and icing conditions are encountered. Samples from this test roll were taken in January 1984, after the liner had undergone almost two full winters in this environment.

The geomembrane properties considered critical in this climate are its puncture and tear resistance. No damage to the liner in the forms of puncture and/or tear was observed at the time the samples were cut.

The original values for the test roll and values for the samples are shown below in Tables VI and VII.

TABLE VI
Tensile Strength Of Control Samples

Samples	Tensile Strength (psi)	Ultimate Elongation (%)
1	3402	600
2	3304	564
3	3486	600
4	3521	628
5	3471	611
Average	3437	600.6

TABLE VII
Tensile Strength Of Exposed Samples

Samples	Tensile Strength (psi)	Ultimate Elongation (%)
1	3311	605
2	3289	555
3	3371	534
4	3688	566
5	3347	595
Average	3401	571

For the exposed samples, the average change in tensile strength ($\%\Delta T$) is 1.047%, from:

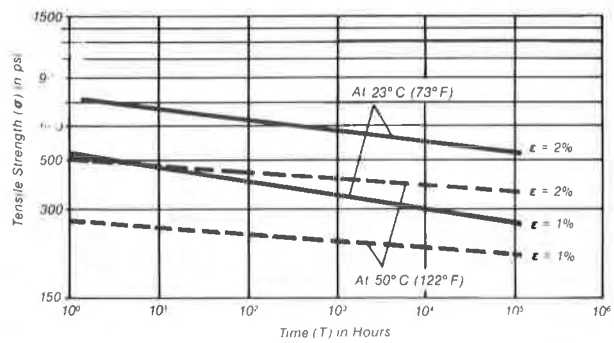
$$\frac{3437 - 3401}{3437} \times 100$$

The average change in ultimate elongation ($\%\Delta E$) is 4.92%, from:

$$\frac{600.6 - 571}{600.6} \times 100$$

These results indicate that the climate of Warren, Pennsylvania has not affected the test roll. The 4.92% loss in ultimate elongation is not considered a severe loss. The actual average value of 571% ultimate elongation after exposure still exceeds the manufacturer's minimum specification of 500%. Additionally, this loss will eventually be compensated for by the high density polyethylene's relaxation property. (The crystalline structure of the material will, over a long period of time, realign itself to relieve stresses below its yield point.)

Laboratory testing of the HDPE liner from the same manufacturer has shown this to be so. In relaxation testing, the specimen was subjected to a constant deformation, and the stress determined as a function of time. If the deformation is small enough, or the relaxation time long enough, relaxation will be complete, i.e., the specimen will return to the unstressed state. This is shown in Figure 1, but the caution must be made that this data cannot by itself be used for dimensioning calculations, as the exact stress levels present in field applications are generally not available.



Typical tensile stress (ε) vs time (T) under constant load (σ).

FIGURE 1

CASE #3 - HOUSTON, TEXAS

A roll of the HDPE geomembrane was exposed at the manufacturer's facility in Houston, Texas, from February 1981 until February 1984. The thickness of the test roll was 2.0mm (80 mil). Before the material was placed for the exposure test, the following tests were conducted and values recorded as follows:

Tensile Yield Strength (T_Y)	3487 psi	ASTM D638
Yield Elongation (E_Y)	16.25%	ASTM D638
Tensile Break Strength (T_B)	4592 psi	ASTM D638
Break Elongation (E_B)	676%	ASTM D638
Tensile Impact Energy (I)	572 mJ/mm^2	ASTM D1822
Stress Crack Resistance	No failures 1000 hours	ASTM D1693
Percent Carbon Content	2.4%	ASTM D1603

Samples from the exposed roll of liner were removed in February 1984. The values are shown below in Table VIII, along with the manufacturer's minimum specifications.

TABLE VIII
Retention Of Tensile Strength For Geomembrane Exposed At Houston, Texas

	Values For Exposed Samples	Manufacturer's Minimum Specification
T_y	3484 psi	2500 psi
E_y	15%	10%
T_B	4585 psi	3500 psi
E_B	673%	500%
I	569 mJ/mm^2	400 mJ/mm^2
Stress Crack Resistance	> 800 hours (test still in progress)	> 500 hours
Percent Carbon	2.4%	2–2.5%

Comparing the values for the samples taken after three years of exposure with the values for the original, unexposed liner, the percent changes in values are:

% T_Y	-0.08
% E_Y	-7.6
% T_B	-0.15
% E_B	-0.44
% I	-0.52
Stress Crack Resistance	No Failures To Date

It can be seen from these values that no severe loss or degradation of mechanical properties was encountered after three years of exposure in this climatic area of high humidity and severe UV loadings.

WEATHERING RESISTANCE EXPERIENCE IN EUROPE AND THE MIDDLE EAST

Weathering and other environmental effects that cause lasting changes in a geomembrane are generally known under the term "ageing." Material changes can be seen through changes in the mechanical properties, and under certain conditions, a change in mechanical characteristics can permit an estimation of the material's life span.

Increased temperature and UV radiation are, with polyolefins, the primary causes of ageing. The effect of increased temperatures is termed thermal-oxidative, and that of UV radiation is photochemical deterioration. Both modes of attack can be simulated in laboratory tests, and/or evaluated from already installed projects.

A number of laboratory tests for long term ageing have been performed by the raw material supplier. In tests for normal photochemical attack (Xenon test 450, according to DIN 53387), no deterioration could be determined in the high density polyethylene geomembrane's mechanical properties after 5000 hours. This period of testing is considered roughly equivalent to 25 years under mid-European climatic conditions. The results of thermal-oxidative tests allow an extrapolation, by purely thermal attack of 50°C, of a life span of more than 50 years.

Testing was also performed for combined attack modes of thermal-oxidative with increased photochemical attack (Xenon test 150). After a test period of 10,000 hours, no change could be determined up to the yield point in the behavior of the material. The elongation at break value was, however, reduced to 50 percent of its original value.

The installer has also performed weathering resistance testing on samples taken from already installed projects. Two projects from arid climates (Basrah, Iraq and Sar Chesmeh, Iran) and three projects in Europe (two in Germany and one from Austria) were used for this purpose.

Samples were taken from all five of the installations and short term tensile tests, according to DIN 53455, were performed at the manufacturer's laboratory in Hamburg, West Germany. The values are shown below in TABLE IX.

TABLE IX
Mechanical Properties From Five European And Middle Eastern Projects

Project	Age of Sample (In Years)	Yield Tensile Stress (psi)	Yield Elongation (%)	Break Tensile Stress (psi)	Break Elongation (%)
Basrah	1	2745	15.5	4394	880
Sar Chesmeh	1	2531	15.0	3200	760
Burghausen	3	3157	13.8	4550	860
Galing	3	3228	13.0	4181	830
Vienna	2	2588	15.4	3967	850
	4	2616	15.0	3854	855
Virgin Material	0	2560	14.0	3413	800

When comparing the typical values for virgin material, as shown in the bottom row, with the values obtained from the short-term tensile tests, one may conclude that no significant material changes have occurred. All values for exposed samples are within the ranges of test discrepancies and fluctuations in the base material values. The manufacturer concludes that in a mid-European or Middle Eastern climate the life span of this geomembrane is more than 20 years.

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Performance of Plastic Canal Linings

A study was conducted to determine the performance of buried plastic membrane linings, primarily 0.25-mm-thick PVC, used for seepage control in Bureau irrigation canals. Samples from nine canal installations ranging in service life from 1 to 19 years were evaluated. Results of the study indicate that plastic linings are providing satisfactory service for seepage control and are viable alternatives in areas not suitable for concrete or compacted earth linings. Results of the study indicated that some stiffening of the PVC has occurred with time. This stiffening or aging is caused by the loss of plasticicizer, the agent used in the manufacturing of the lining to impart flexibility. The rate of this aging is primarily dependent on three factors: (1) Source - linings originally manufactured with a high plasticizer content exhibited less aging, (2) Location - samples obtained from within the water prism exhibited less aging than those obtained outside the prism, and (3) Subgrade preparation.

INTRODUCTION

Under the LCCL (Lower Cost Canal Lining) Program which began in 1946, the Bureau of Reclamation conducted laboratory research and field experimentation with virtually every material thought to have value as a lining material to control seepage from canals. Development of new or improved construction methods and maintenance techniques was also part of the LCCL Program. In addition to plastic membrane linings, Bureau engineers working in cooperation with contractors and manufacturers developed economical reinforced and unreinforced concrete and compacted earth linings. These linings are now in service in more than 8000 km of the Bureau's irrigation systems.

Beginning in 1967, because of the growing emphasis to place more of its water conveyance systems underground in pipelines, the Bureau terminated the LCCL Program and initiated the OCCS (Open and Closed Conduit Systems) Program which includes study of closed systems and continuing studies of lower cost canal linings. Work under both programs produced plastic membrane linings for seepage control in irrigation canals [1].

Plastic linings have generally been used in conjunction with the rehabilitation of old, unlined canals. However, plastic linings are now being specified for new construction on the Closed Basin Division, San Luis Project, Colorado, and the Garrison Unit, Pick-Sloan Missouri Basin Project, North Dakota.

With the increased use of plastic linings in Bureau work, a study was conducted to determine the performance of these materials [2]. As part of the study, samples ranging in service life from 1 to 19 years were obtained from the following canal installation for evaluation:

Canal Installation	Age of samples evaluated, years
Helena Valley Canal, Helena Valley Unit, Montana	5, 9
East Bench Canal, East Bench Unit, Montana	4, 5, 8
Wyoming Canal, Riverton Unit, Wyoming	2, 4
Pilot Canal, Riverton Unit, Wyoming	1
Bugg Lateral, Tucumcari Project, New Mexico	4, 9, 14, 19
McCaskey Lateral, Tucumcari Project, New Mexico	18
Lateral H, Sun River Project, Montana	5, 13
Amarillo Canal, Navajo Indian Irrigation Project, New Mexico	1, 2, 3
Mirage Flats, Mirage Flats Irrigation District, Nebraska	14

This paper summarizes this study conducted from 1973 to 1983. Also presented is a brief description regarding plastic lining construction.

CONSTRUCTION

General

As mentioned in the Introduction, much of the Bureau's plastic membrane lining work has been accomplished under its R&B (rehabilitation and betterment) program. Generally, this R&B work is restricted to the nonirrigation season and often involves wintertime construction. The installation of plastic membrane linings both in R&B work and new construction involves four procedures: excavation, subgrade preparation, installation of the plastic lining, and placement of the earth cover material.

Because of the earth cover requirement, buried membrane linings are restricted to canals having a low velocity flow. Maximum allowable velocity may range between 0.3 to 0.9 m/s, depending on the soil cover.

Excavation

Overexcavation of the existing canal prism, a minimum of 300 mm, is required to allow for placement of the protective earth cover. The side slopes are made sufficiently flat to ensure that the cover material remains on the slopes under operating conditions. The proper side slopes will depend upon the type of earth cover material used, but even with the best conditions the slopes should never be steeper than 2H:1V.

Subgrade Preparation

After the rough excavation is completed, the subgrade is prepared to a firm, relatively even surface. Sharp rocks, roots, and other objects that might puncture the membrane are either removed or padded by covering with 75 to 100 mm of sand or fine-textured soil.

Dragging the subgrade with a heavy machine-type chain or an old tractor truck is a rapid and effective method for smoothing the canal subgrade. In earlier plastic lining construction, there was a specification requirement for rolling to obtain a smooth subgrade. This particular requirement was a carryover from work with the hot, spray-applied asphalt membrane material. However, subsequent experience with plastic linings indicates that this procedure is generally not required.

If weed growth is considered to be a likely problem, an approved soil sterilant should be applied to the subgrade. Care should be taken not to apply sterilants outside the lined area if landscaping or grasses are to be established later.

Plastic Lining Installation

PVC plastic lining is supplied to the jobsite in large, shop-fabricated sheets wide enough to cover the canal prism and to any length practical for handling, to minimize the amount of field joining required. The lining is generally packaged accordion-folded in both directions and is simply unfolded and pulled into place.

For most R&B work, sheets of PVC lining can be joined simply by lapping the downstream end of one sheet 1 meter over the upstream end of the adjacent sheet. PVC has a tendency of adhering to itself and with the weight of the earth cover, a sufficiently bonded joint is obtained where 100 percent seepage control is not required. Where a more positive seal is required, the PVC should be overlapped a minimum of 300 mm and a solvent cement recommended by the manufacturer of the PVC lining applied to a minimum width of 50 mm.

The upper edges of the plastic lining are buried in an anchor berm at the top of the slope at the time of installation to prevent removal of the lining by unexpected gusts of wind.

It is important that the plastic lining is placed in the canal in a slack condition so that the weight of the earth cover will not cause severe stressing.

Placement of the Protective Earth Cover

The earth cover is placed soon after the plastic membrane is installed to eliminate possible wind or other damage. An earth cover is an essential part of a buried membrane lining system because its function is to protect the membrane from the elements, animal traffic, vandalism, and mechanical damage during canal cleaning operations.

The depth of cover depends on size of canal, water velocity, canal side slopes, and soil material available. The minimum recommended total cover depth is 250 mm plus 25 mm for each 0.3 meter of water depth.

To minimize costs, approved excavated material should be used for one-half (bottom layer) the cover requirement. For erosion protection, the upper layer should be a sand and gravel mixture conforming to Bureau specifications.

Protective cover placement should begin as soon as the plastic lining is judged to be properly positioned, jointed, and repaired if necessary. The cover material can be placed with draglines, conveyors, trucks, or by other means, preferably starting at the toe and working upslope. During the initial stages, careful inspection should be made to ensure that the cover placement operations are not causing damage to the plastic lining. It is not necessary to compact the cover material by rolling; however, dragging is usually required to attain the required finished and uniform thickness.

PERFORMANCE STUDY

The study involved both field and laboratory investigation. The field portion included the following:

1. Obtaining samples of in-service linings for laboratory evaluation.

2. Making visual observations concerning the condition of in-place lining, including protective earth cover material and providing comments regarding the performance of the plastic linings.

In addition, seepage tests were conducted on the Helena Valley Canal, East Bench Canal, Lateral H, and Fivemile Lateral. Results are tabulated in the following:

Canal	Location	Date of ponding tests	Seepage loss $(L/m^2/)/d$
Lateral H, Sun River Project, Montana	-	9-10-65 (before lining)	152.5
		10-10-66 (after lining)	0.0
		9- 29-30 -70	3.1
Helena Valley Canal, Montana	Sta. 1321+93 to Sta. 1377+50	10- 7-8 -73	14.3
		4- 25-26 -74	16.8
East Bench Canal, Montana	Sta. 878+00 to Sta. 933+61	10- 17-18 -73	11.6
		4- 10-11 -74	17.1
Fivemile Lateral, Riverton Unit, Wyoming	Sta. 200+00 to Sta. 207+00	10-82	1.8

The laboratory evaluation of the retrieved samples involved the following:

1. Visual examination

2. Physical properties testing

3. Conducting chemical extraction tests to determine plasticizer content of the PVC lining

Changes in physical properties of the PVC lining on the Bugg Lateral, Tucumcari Project, New Mexico, after 4, 9, 14, and 19 years of service are summarized in table 1. Also listed is the plasticizer content of the lining at the various ages.

Results of the studies indicate that buried plastic membrane linings generally are providing satisfactory service for seepage control. Minor problems have been encountered with animal traffic damaging the protective earth cover, necessitating some repair work. To protect the linings from animal traffic and cleaning operations, a minimum cover depth of 400 mm is recommended.

Several projects reported some problems with weed growths, which create more maintenance activities (cleaning). It appears that the linings in combination with the earth cover provide a receptive environment for establishing and maintaining vegetation. Damage to the linings from root penetration has been minimal. When damage has occurred, as noted on the Tucumcari Project, New Mexico, it has been in areas primarily above the waterline.

Laboratory studies and field observations indicate that some stiffening of the PVC lining has occurred with time. This stiffening or aging of PVC is caused by the loss of plasticizer, the agent used in the manufacturing of PVC to impart flexibility. With aging, there is generally a reduction in elongation, an increase in modulus at 100-percent elongation, and a decrease in resistance to impact damage at low temperatures. In regard to the latter, PVC linings are now being used more in cold climates and often must be installed in the wintertime. Consequently, good low temperature properties are important to ensure that the lining is not damaged when unfolding and installing and also during the placement of the protective earth cover. After the lining has been installed and in service, the requirement for good resistance to impact damage at low temperatures is not as critical.

The rate of aging on PVC lining is primarily dependent upon the following factors:

- Source of PVC lining. - The linings originally manufactured with a high plasticizer content exhibited less aging.

- Location in canal. - Samples obtained from within the water prism exhibited less aging than those obtained outside the prism. This is probably because the lining within the water prism is subjected to fewer wetting and drying cycles and more uniform temperature conditions.

- Condition of subgrade. - Samples obtained from areas where the lining had been placed over a fairly smooth subgrade exhibited less aging than those installed over a coarser base.

Results of field studies on polyolefin-type linings such as polyethylene (Amarillo Canal) and ethylene vinyl acetate copolymers (Mirage Flats Canal) indicate they have excellent aging characteristics. However, additional work is needed with these materials to develop better field seaming techniques. Because of their inertness, they are very difficult to seam. If suitable methods can be developed, such materials would be good candidates for the bottom-only lining studies being conducted by the Bureau's Grand Island, Nebraska Office [3]

As a result of the performance study, the Bureau is now specifying 0.5-mm-thick PVC rather than 0.25-mm-thick PVC. The additional cost of the heavier gage material will be minimal. With a 100-percent increase in the thickness of the membrane, the overall cost of construction will only increase by about 15 percent. Also, the Bureau has been working with the NSF (National Sanitation Foundation) since 1978 on developing material standards for geomembranes used in various containment systems including irrigation canals. The standards were approved in November 1983, and the Bureau plans to incorporate them into its seepage control work.

REFERENCES

[1] Hickey, M. E., Investigations of Plastic Films for Canal Linings, Research Report No. 19, Bureau of Reclamation, Denver, Colorado, 1969.

[2] Morrison, W. R., and Starbuck, J. G., Performance of Plastic Canal Linings, REC-ERC-84-1, Bureau of Reclamation, Denver, Colorado, 1984.

[3] Kutz, R. D., Special Membrane Canal Lining, Farwell Main and Lower Main Canals, Nebraska, Bureau of Reclamation, Grand Island, Nebraska, 1982.

Table 1. - Physical properties test results for PVC membrane linings - Bugg lateral, Tucumcari Project, New Mexico - installed spring of 1961

Physical property	Specifications requirements	Sample No. B-3526 (typical original results)	Sample No. B-4343 (4 years of service, BWL)	Sample No. B-6094 (9 years of service, BWL)
1. Thickness, mm (mils) percent change	0.25 (10) ± 10%	0.28 (11.2)	0.26 (10.2) -8.9	0.25 (9.8) -12.5
2. Tensile strength, N/mm (lbf/in^2) percent change	3.0 (17)*	4.5 (25.6) L 4.0 (22.8) T	4.8 (27.4) L 4.1 (23.7) T +7.0 L +3.9 T	4.4 (25.2) L 4.0 (22.6) T -1.6 L -0.9 T
3. Elongation percent percent change	225*	412 L 462 T	322 L 336 T -21.8 L -27.3 T	260 L 433 T -36.9 L -6.3 T
4. Modulus @ 100 percent elongation, N/mm (lbf/in) percent change	Not required	2.0 (11.4) L 1.8 (10.3) T	2.3 (13.3) L 2.0 (11.7) T +16.7 L +13.6 T	2.8 (15.8) L 2.0 (11.7) T +38.6 L +13.6 T
5. Elmendorf Tear, grams percent change	1500*	1830 L 2290 T	2430 L +32.8 L	2900 L 2840 T +58.6 L +24.0 T
6. Impact resistance	Not more than 2 specimens out of 10 shall fail @ -18 °C (0 °F)	10 tested 1 failure	10 tested 0 failure	10 tested 0 failure
7. Plasticizer content, % percent change	Not required	39.8	Not determined	35.2 -11.6
8. Bonded seam strength, percent of parent material	65	100	97.9	94.4

L denotes longitudinal direction.
T denotes transverse direction.
* minimum, each direction.
AWL denotes above normal waterline.
BWL denotes below normal waterline.

Table 1. - Physical properties test results for PVC membrane linings - Bugg lateral, Tucumcari Project, New Mexico - installed spring of 1961 - Continued;

Physical property	Sample No. B-6763 (14 years of service, AWL)	Sample No. B-6764 (14 years of service, BWL)	Sample No. B-7022 (19 years of service, BWL)	Sample No. B-7023 (19 years of service, AWL)
1. Thickness, mm (mils)	0.24 (9.3)	0.24 (9.6)	0.24 (9.6)	0.21 (8.2)
percent change	-17.0	-14.3	-14.3	-26.8
2. Tensile strength, N/mm (lbf/in)	4.6 (26.0) L 5.8 (33.3) T	4.2 (24.2) L 4.6 (26.1) T	4.6 (26.4) L 5.2 (29.8) T	5.0 (28.6) L 4.7 (26.9) T
percent change	+1.6 L +46.0 T	-5.5 L +14.5 T	+3.1 L +30.7 T	+11.7 L +18.0 T
3. Elongation percent	169 L 225 T	268 L 274 T	211 L 188 T	151 L 188 T
percent change	-59.0 L -51.3 T	-35.0 L -40.7 T	-48.9 L -59.3 T	-63.3 L -59.3 T
4. Modulus @ 100 percent elongation, N/mm (lbf/in)	3.9 (22.4) L 4.3 (24.8) T	2.4 (13.8) L 2.4 (13.8) T	3.6 (20.5) L 4.2 (23.9) T	4.6 (26.0) L 4.2 (23.9) T
percent change	+96.5 L +140.8 T	+21.1 L +34.0 T	+79.8 L +132.0 T	+128.1 L +132.0 T
5. Elmendorf Tear, grams	1660 L 2210 T	3000 L 2865 T	3000 L 2200 T	450 L 1300 T
percent change	-9.3 L -3.5 T	+63.9 L +25.1 T	+63.9 L -3.9 T	-75.4 L -43.2 T
6. Impact resistance	5 tested 5 failures	5 tested 5 failures	Not determined	Not determined
7. Plasticizer content, %	23.1	34.1	27.0	21.6
percent change	-42.0	-14.3	-32.2	-45.7

L denotes longitudinal direction.
T denotes transverse direction.
* minimum, each direction.
AWL denotes above normal waterline.
BWL denotes below normal waterline.

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Establishing Acceptable Standards for Use of Plastic Membranes in Canal Construction Worldwide

This year, guidelines have been completed by the International Commission on Irrigation & Drainage (ICID) to help standardize construction methods and material testing for the application and use of membrane linings for watertight canal construction. These guidelines, developed by a working committee of 10 nations, are performance oriented, requiring the membrane manufacturer to meet the jobsite conditions and tests required for a successful project. The abbreviated guidelines are reviewed in detail and are offered for consideration of this conference.

INTRODUCTION

The International Commission on Irrigation and Drainage represents technical people from 74 countries, all with a common interest in furthering irrigation and land reclamation within their countries. The International Commission not only serves as a sounding board where representatives of the various nations can meet yearly, but also provides for an update and circulation of technical information. Within the Commission is a standing committee concerned with the various aspects of construction of irrigation projects. This committee known as the Committee for Construction and Operation is composed of members from Canada, U.S., USSR, India, England, France, Egypt, Iraq, Syria and Spain. Here we have a United Nations type organization trying to establish a common understanding and agreement on standards or guidelines for construction processes worldwide. Anyone having worked on international committees knows the frustration of bringing a group of countries to a common understanding of what is right or wrong, good or bad, acceptable or not acceptable. Each country has its own biases, but how interesting it is to ask yourself why your country does what it does, why it follows the practices it does and then try to support your position against other countries who have totally different practices.

Because of an apparent increase in the use of plastic membranes in canal construction brought on by construction problems in difficult soils and the increasing need to conserve water and increase the efficiency of distribution, the Committee on Construction was assigned the task of developing a set of standards or guidelines covering the use of geotechnical membranes to construct watertight canals. At the outset it was proposed that standards had been established in the U.S. by the American Society of Agricultural Engineers, (ASAE EP3401) and that this work could suffice. After a serious review by the committee, it was decided that the American document as written would not serve the needs of other countries and that what was needed was a result oriented document that would allow specifications to be written outlining the end result required by the client and the testing that would be used to determine the adequacy of the material being supplied. This document was developed in 1981 and 1982, reviewed by the Committee at their annual meeting in Melbourne, Australia, in October, 1983, and has now been finalized by the 12th International Congress held at Fort Collins, Colorado in May, 1984.

Starting as a set of standards, the document has ended as a guideline for consultants and clients. The further the subject matter has been studied, the more apparent it has become that adequate test data is still not available to establish absolutes in the field of standards. William McCready of Sir M. McDonald of Cambridge, England, Chairman of the Working Group on Construction, has brought together a good document, acceptable by the various countries taking part in the effort. The general consensus of what is important and what is not highlights this document and provides the reader with a good insight into the usage of membranes for canal construction.

ICID GUIDELINE 108
Use of Flexible Membranes for Irrigation, Canal and Reservoir Lining
(Abbreviated Draft)

Purpose

The purpose of this document is to provide the designer of a membrane-lined irrigation canal or reservoir with guidelines for its design and specification.
This guideline covers preformed flexible membranes of rubber, plastic or similar material, used as watertight linings for canals, reservoirs and associated structures in irrigation systems. It distinguishes different types of application and gives guidelines for appropriate designs, installation methods and specifications.
The main applications considered are the lining of new canals and open pond reservoirs, with exposed or buried membrane liners. The rehabilitation of deteriorated canals or reservoirs, and the use of membranes in double-watertightness systems, are also mentioned but the Guideline does not replace specialist knowledge and experience.

Application

Buried Liners

Some liners are covered with soil or rigid materials, for one or more of the following reasons:

- Protection from solar radiation, particularly ultra-violet
- Protection from mechanical damage by maintenance machines and workers, by animals or by vandals
- Reduction of range of temperature variation
- Protection from lifting by wind
- Restraint against uplift by groundwater

Because of the protected installation, the liner can be of a relatively cheap material. Disadvantages are that bank slope is limited by the cover material, and that the liner cannot be easily inspected or repaired. Cover materials include soil and concrete (precast, slipformed or sprayed).

Exposed Liners

Exposed liners must either be made of materials resisting deterioration due to radiation and weathering, or a relatively short useful life must be expected. They must be adequately protected from machines, animals and vandals, and measures must be taken to prevent lifting by wind and groundwater. In canals, where prevention of public access is difficult or impossible and where operations to remove sediment or weeds present a danger of mechnical damage, it is very rare that an exposed liner can be considered. For reservoirs, however, mechanical damage can often be guarded against and exposed liners then offer cost savings and ease of inspection and maintenance.

Double Watertightness

In special circumstances, such as canals on loess or gypsum soils, a particularly high degree of watertightness and reliability may be needed and this may justify a double watertightness system. This consists of two impermeable layers with a permeable drained layer between them. Any water leaking through the first impermeable layer flows away in the porous layer and cannot build up pressure to make it leak through the second impermeable layer. Either or both the impermeable layers can be of flexible membrane, the drained layer can be sand or gravel or a synthetic fabric.

Rehabilitation of Deteriorated Linings

Flexible membrane liners are sometimes placed over rigid linings which are unsatisfactory because of cracking or deterioration.

Materials

The main constituent of a flexible membrane liner is the plastic or elastomer that provides watertightness. Historically, some materials were called plastics and some were called rubbers, but there are materials which cannot be classified in these terms. An unsupported or unreinforced membrane consists merely of a sheet of such material, and typically has an elongation at fracture of more than 250%. A supported or reinforced membrane includes also a fibrous fabric or scrim which adds strength and reduces elongation.

Most materials are subject to deterioration when exposed to heat, ozone, and/or ultra-violet radiation, and it is important to test resistance to such deterioration. The effects of aging and exposure include loss of strength, loss of flexibility and elasticity and shrinkage. Some materials are liable to be attacked by animals and insects, particularly rodents and termites seeking water.

Membranes are made in strips and these are joined together to form liners, overlap joints being made in a factory or on site. Some materials can be joined by heat welding without the addition of other material; some are joined by vulcanizing; some are joined by placing an adhesive-coated strip between the layers of membrane; and some are joined by adhesives.

Failure of linings is usually associated with shrinkage, loss of strength or loss of flexibility. Specifications should require tests of these properties after artificial exposure to heat or ultra-violet radiation, as well as tests at temperatures typical of actual service, and a test for ozone resistance.

Membranes are available over a wide range of prices, from very strong, stable and long-lasting liners to cheap ones suitable for temporary works or some of the less demanding permanent applications. Suggested specifications takes account of this by considering various expected durations of useful life, as well as various climates and degrees of exposure.

Design for Particular Applications

Buried Liners

When a liner is covered by soil or other granular material, the side slope is limited by soil stability. The slope of the liner may be steeper than that of the soil surface so that the thickness of the soil cover increases with depth. Soil slope must be decided on the basis of the soil's characteristics and attainable compaction.

Cover thickness is determined by practical considerations, construction being the deciding factor rather than service. A minimum cover thickness of 250mm is recommended and, if heavy plant is to operate, 300mm is more appropriate. Permissible velocity and hydraulic roughness should be estimated from the covering soil's characteristics.

After placing and compaction is completed, the cover should not include particles larger than 2mm in the 50mm nearest the liner, unless well-rounded gravel is used with an appropriate liner material. (To achieve this stone-free zone, an initial sand layer up to 150mm thick may be needed, depending on construction method). Alternatively, or additionally, non-woven matting may be used to protect the membrane. In either case, thickness needed depends on the liner, the fill material, and the method of placing; details should be determined by field trials.

Exposed Liners

Exposed liners in reservoirs can be designed for slopes up to about 1 vertical to 1.5 horizontal with unreinforced liners, steeper with reinforced ones. The design must include measures to limit or prevent access by animals and vandals, and must provide adequate underdrainage and anchorage to avoid lifting by wind. Anchoring and drainage details are discussed on the following pages.

In the rare case where a canal liner can be adequately protected without burial, average water velocity should be limited to 1.5 m/s. In the absence of other information, a Manning's hydraulic roughness factor of n = 0.018 can be used for such exposed liners.

For side slopes more than 8m long, some unreinforced liner materials may prove unsatisfactory. In these cases, a superior unreinforced liner should be used, or a reinforced liner considered.

Rehabilitation of Deteriorated Linings

An old rigid lining should be rendered stable before covering with a membrane liner, and, if any relative movement is likely at underlying joints or cracks, the membrane should be chosen for adequate extensibility. In some instances, felt matting has been used beneath the membrane liner to take up tension forces caused by movements at cracks or joints. This results in membrane stresses remaining low. Fixing the membrane to the old lining with adhesives may be useful against wind uplift or mechanical damage, but should not be carried out within 200mm of a joint or significant crack in the old lining. Access by animals, vandals and machines must be prevented or limited.

Design Aspects Common to All Applications

Freeboard

Figure 1 defines the liner freeboard (FL) and bank freeboard (FB) for both canals and reservoirs, relative in all cases to the highest water level expected. For canals, recommended minimum values of FL and FB are given in Figure 2 as functions of design discharge. For reservoirs the freeboard must be determined by estimating wave height and run up. Suggested minimums for small reservoirs are 0.20m for FL and 0.30m for FB.

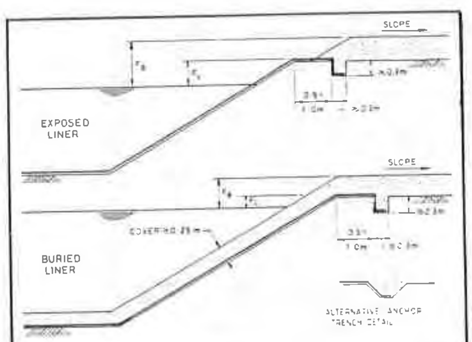

Figure 1 FREEBOARD AND ANCHORAGE DETAILS

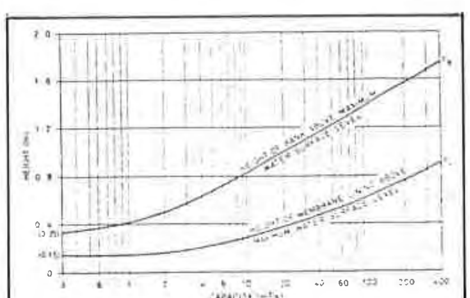

Figure 2 BANK HEIGHT AND FREEBOARD FOR MEMBRANE-LINED CANALS

Anchoring

Liners should be secured at earthen bank tops by means of anchor trenches as shown in Figure 1, or equivalent provisions. Where liners join rigid structures they should be firmly fixed mechanically or by building in. A good fixing to concrete can be made by clamping the liner to the concrete with metal battens 50mm wide by 7 to 10mm thick, fixed by bolts at 250mm centers, with mastic adhesive or a 50 x 7mm neoprene strip between liner and concrete.

Underdrainage

Canals and reservoirs more than 10m wide or more than 2m deep may need provision for relief of water and gas pressure under the liner. Gas may derive from biological decay in the subsoil or from groundwater.

Pore water under the liner should be collected by a continuous drainage layer, or by strip drains not more than 3m apart. The resulting flow can be discharged to low ground or sumps. Gas pressure can be relieved by "air drains" terminating near bank top level, with similar permeable layers and/or collectors, and the two functions can be performed by one system. Effective pressure relief valves for membrane liners are difficult or impossible to make.

Joints

Joints or splices between strips and pieces of liner are made by means of heat welding, vulcanizing, solvents, adhesives and pressure in various combinations. Overlap joints should be used, with minimum overlap depending on strength tests across the joint. For solvent and adhesive methods, minimum overlaps of 50mm for factory made joints and 75mm for site made joints are recommended. These minimums may be reduced in accordance with the manufacturer's recommendations provided the strength across the joint is satisfactory. Upstanding butt or peel joints should be avoided. The choice of joint type should usually be left to the liner supplier, provided tests across the joints are included in the specification.

Preparation of Subgrade

The soil on which the membrane is to be laid must be free of objects capable of puncturing it (sharp stones, sticks, roots), and of objects liable to decompose and leave a void. Rolling is usually beneficial. If necessary the original soils may be covered with a layer of sandy material, nominally at least 25mm thick, which must not be disturbed during liner installation. Alternatively, or additionally, a synthetic non-woven fabric can provide some protection. This may be adhered to the back of the membrane during manufacture. The subgrade should be uniformly compacted to avoid uneven settlement and difficulty during laying, but the soil does not need to be as compact as for rigid linings. Site trials should be used to select an effective and economic combination of protective layers and installation methods.

To avoid subsequent root growth, the subgrade should be treated with a good non-selective soil sterilant, care being taken not to contaminate adjacent areas. Visible roots should be cut off 50 to 100mm below the subgrade surface.

Placing the Membrane Liner

The liner should not be stretched, but no slack or folds should be left. The laying procedure must be capable of avoiding undue disturbance of the subgrade. If the ambient temperature is high, special care may be needed to avoid puncturing the liner.

In the case of buried liners, the cover material must be placed in such a way as to avoid moving, puncturing or stretching the liner. If a sandy layer is used to protect the liner from stones in the main cover material, it should be laid in a moist condition on slopes.

Any areas of lining damaged during installation should be cut out and replaced using an approved jointing method.

Joints

The two pieces to be joined must lie flat on each other. One section should not be drawn tight over another that is wrinkled. The sensitivity of the field jointing method to moisture, dust and temperature must be determined in advance and adequate measures taken to obtain consistently good joints under actual field conditions. All field joints must be inspected for soundness and continuity, both visually and by feeling under the overlap. Samples cut at random from the installed liner, across the field joints, should be tested.

SPECIFICATIONS

General

The designer of a membrane-lined structure will generally need to write specifications for the subgrade preparation, the installation procedure, the membrane itself, and the joints. Those for the preparation and the installation can be drawn from information given thus far. This section provides guidelines for the material and joint specification.

Specification by Performance

This guideline suggests performance limits for various liner applications. A specification should depend on the application, not the material, so that different materials competing for a particular job can be objectively compared. A specification for one of the more demanding applications is likely to exclude some type of material altogether.

This chapter classifies the range of liner applications into 16 cases by quantifying three aspects:

- Exposure - either "exposed" or "buried" under soil, concrete or similar materials;
- Climate - in four categories described below;
- Design life - two categories indicating the approximate period for which the liner is expected to maintain its performance, "short" implying less than approximately 10 years, "long" over 10 years and up to 20 or 30 years.

A performance-related specification for any application can be drawn up from the suggested requirements and the test methods set out in the document. The guiding principle is that the specification should not directly limit the thickness of the liner but should give minimum performance criteria in terms of tensile strength (force per unit width), tear strength (force on standard specimen) and other parameters. Suppliers can then choose to meet these requirements with a thin liner at high stress or a thicker liner at low stress. The distinction between more and less demanding applications is made by varying the artificial aging and weathering and the testing temperature, leaving the strength criteria constant.

Tests after artificial aging and weathering can only give an approximate guide to the long-term performance of liner materials. The procedures described here should be supplemented by consideration of the service history of a particular material, particularly when the design is long life. In considering such history, designers and users should examine carefully all examples of prior application with the following questions in mind:

(a) Was the material exactly the same as the one being proposed for the new application?

(b) How relevant are the site conditions to the new case?

Classification of Climates

The four climate categories are defined in terms of average monthly temperatures. The classification is a special one for the present purpose. The categories "very hot" and "hot" are defined by the temperature that is exceeded by the three highest monthly averages (i.e., the temperature of the third-hottest month, which is not the same as the average for the three hottest months). Similarly, the "very cold" category is defined by the temperature exceeded by the averages of nine months (i.e., the third-coldest monthly average, and also by the coldest month). A site would be classified as "very cold" even if only one of these conditions is fulfilled. The "moderate" category is a residual class representing sites which do not fall into any of the other three categories.

Routine Tests

Once a liner material and its thickness have been chosen, a few tests need to be applied regularly to ensure that membrane and joint properties are reasonably consistent. The arrangements should take into account the circumstances of each job, but the following recommendations can be used as a guide:

Frequency - one set of samples from each 5,000 to 10,000m2 of liner, including samples of joint made under factory and (if appropriate) field conditions.

Thickness - Should not vary by more than 30% from the normal thickness.
Unaged Properties - Tensile strength and elongation at break should not vary by more than 30% from the average values obtained from unaged and unweathered samples tested at the stage of liner selection. Tests should be made with and without joints.

Other Properties - The above should usually suffice for routine tests, but if the tests at liner choice stage indicate that a particular property is likely to be critical for the liner chosen, additional routine tests should be devised to check that property.

Testing Methods

This section sets out the test methods, generally international (ISO) standards, intended to be used for the quantitative specifications. It includes some indications of the national standards (often more widely known than international ones) which are precisely or approximately equivalent or can be used as acceptable alternatives. In this context, it is intended that "equivalent" tests can be regarded as giving the same results.

Figure 3

SUMMARY OF TEST REQUIREMENTS

Climate:	"very hot"		"hot"		"moderate"		"very cold"	
Design Life:	Long	Short	Long	Short	Long	Short	Long	Short
TESTS WITHOUT AGING OR WEATHERING								
Uniformity	No pinholes, cracks, blemishes, etc., unless proved insignificant.							
Ozone Resistance	One test for all cases - 7 days, 50 pphm.							
HEAT AGING (weeks)	15	8	6	4	2			2
WEATHERING (weeks)								
- If exposed	6	2	6	2	6	2	6	2
- If buried	1 week for all cases (unless no sunlight before burial)							
TESTS AFTER AGING OR WEATHERING (SEPARATELY)								
Tensile properties (values at break) on plain liner or across a joint								
Testing temp. Degrees C	80		60		40 and 0		23 and -10	
Strength kN/m	min. 6 (unless low-modulus*, then 3)							
Elongation %	min. 100 (unless reinforced, then 10)							
Tear, N								
- Low-modulus*	min. 8							
- High-modulus*	min. 12							
- Reinforced	min. 20							
Brittleness: Max. Impact b. temp. Degrees C	No requirement				Max. -10		Max. -35	
Burst strength Pressure kPa	To be tested, but no minimum requirement							
Shrinkage %	Max. 3% after heat aging (no weathering)							
Application Code								
- If exposed	EVL	EVS	EHL	EHS	EML	EMS	ECL	ECS
- If buried	BVL	BVS	BHL	BHS	BML	BMS	BCL	BCS

* "Low-modulus" means nominal stress at 100% strain in tensile test is less than 7 MPa; "High-modulus" means all other unreinforced liners.

Figure 4

SUMMARY OF TESTING METHODS

	ISO	BS	ASTM	DIN
Dimension & Shrinkage Thickness	4648 4593	903:A38 2782 Method 512B 1970	D1042.51 D374-79 Method C D751-74	-
Ozone Resistance	1431	903 Part A23 1963	D1149-64	53509
Ultra-violet Exposure - Weathering				
Xenon Arc	105-B2	1006-1971	G 26-77	53387(1982)
Carbon Arc	4892 4892	-	D 2565-79 G 23-69 D 1499-64 D 750-68	
Heat Aging	188	903 Part A19 1975	D 573-67 D 3045-75 D 756-78	50011
Tensile Properties Unreinforced over 0.7mm thick	R 527	2782 Method 320A 1976-903 Part A2-1971 Type 1	D 412-68 Die C D 638-77a Type IV	53455 Specimen #4 53504 Specimen S1
Unreinforced less than 1.0mm thick	R 1134 1421	2782 Method 326- C - 1977	D 882-79 D 751-79 Method B	
Tear Resistance	34 Method C	903 Part A3 - 1972	D 624-73 Die B	
Impact Brittleness Temperatures	R 812 4646	903 A 25-1968 Amended 73	D 746-79 D 2137-75	
Burst Strength	1420 etnod A2		D 751-79 Method B Proced. 2	

These guidelines and tests have been developed by the Construction Committee of ICID under the chairmanship of William McCready and Gordon Hawkins. This committee will gladly review all recommendations for improvement on the guidelines which eventually will be re-defined as standards. Contact regarding these guidelines can be made at any of the three locations below:

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ICID
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Chanakyapuri
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Copies of the complete guidelines can be obtained from the same sources.

COURTESY OF U.S. BUREAU OF RECLAMATION

STARBUCK, JOHN G. and MORRISON, WILLIAM R.
U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, USA

Flexible Membrane Lining for Closed Basin Conveyance Channel, San Luis Valley Project, Colorado

The Closed Basin Conveyance Channel is located in the San Luis Valley near Alamosa, Colorado. The channel will be used to transport salvaged ground water to the Rio Grande River as a supplemental source.

In 1983, two specifications were issued by the Bureau of Reclamation to construct 34.6 km (21.6 mi) of channel of which 27.0 km (16.9 mi) will be lined with 20-mil PVC (polyvinyl chloride) plastic film. Future work calls for an additional 32.0 km (20 mi) of channel to be constructed and lined.

Under the first two specifications, approximately 652,000 m^2 (780,000 yd^2) of 20-mil PVC will be installed, representing the largest use to date in the United States of a flexible membrane lining in canal construction.

INTRODUCTION

The San Luis Valley Project is located in Colorado in the uppermost part of the Rio Grande drainage basin. The San Luis Valley, a high (elevation 2250 m), flat, semiarid plain about 80 km (50 mi) wide and 200 km (125 mi) long, once was the bed of an ancient lake. Irrigated by the Rio Grande and artesian wells, it has become one of the most productive farming areas in Colorado.

The area in the center of the Valley (Closed Basin) is not drained by the Rio Grande River, but instead, both surface and subsurface water moves toward the Closed Basin where it is nonbeneficially lost by evaporation and exapotranspiration. This water will be pumped from many wells located in the Basin and conveyed to the Rio Grande River near Alamosa, Colorado. The conveyance channel, when completed, will be 64 km (40 mi) in length. The typical cross section of the channel is shown in figure 1.

Two specifications, [1] and [2], have been awarded to date for the construction of the conveyance channel which has a maximum capacity of 4.53 m^3/s (160 ft^3/s). Reach A [1] is 17.1 km (10.6 mi) in length and incorporates 316,900 m^2 (379,000 yd^2) of 20-mil PVC. Reach B [2] is 17.0 km (10.5 mi) in length with 286,400 m^2 (342,500 yd^3) of 20-mil PVC.

The following three lining materials were under consideration during the design formulation stage for the conveyance channel:

a. Compacted impervious clay lining.
b. Unreinforced concrete lining, 64 mm (2.5 in) thick.
c. PVC liner, 20 mils thick protected with an earth, sand, and gravel cover.

In the design stage, samples of clay lining material were analyzed and found to be dispersive, which eliminated the use of impervious lining. The concrete lining was not used because of cold winter temperatures of -40 °C (-53 °F), and frost would have caused problems due to existing high ground water. The one design that proved to be adequate for our use was a 20-mil PVC liner with earth, sand, and gravel cover.

Our final design for the main Conveyance Channel as shown in [1] Bureau of Reclamation Solicitation/Specifications 3-5B-5D-00260/DC-7553 was a channel in the first reach 4.27 m (14.0 ft) wide with 2:1 side slopes and a water depth of 1.72 m (5.63 ft). In the second reach, the channel was 3.66 m (12.0 ft) wide with 2.5:1 side slopes and a water depth of 1.72 m (5.63 ft). Both sections had a freeboard of 24.4 cm (0.80 ft) to the top of the PVC liner and were designed for a slope of 0.000052 with a Mannings "n" value of 0.0225. The PVC liner was protected with 25.4 cm (10 in) of earth cover with a 15.24-cm (6-in) layer of sand and gravel on top. The specifications requirements for the PVC lining is shown in table 1.

The soil in the first reach had greater stability than that in the second reach, which crosses an extensive area of poorly graded sand and gravel. Because of this, side slopes of 2:1 were used in the first reach and 2.5:1 in the second. The total depth of cover used by the Bureau of Reclamation is based on the total depth equal to 25.4 cm (10 in) + 2.54 cm (1 in) of cover depth for every foot of water depth. The protective-cover sand and gravel was placed at the minimum thickness of 15.24 cm (6 in). The minimum thickness was based on the low channel velocity of 30.4 cm/s (1.13 ft/s) and less.

It is our experience that in order to hold construction costs down, a source for the sand and gravel cover material must be approved prior to construction. If the sand and gravel cover material needs to be blended in order to meet specifications, the cost of furnishing and placing can be twice that for sand and gravel that does not require blending. The typical gradation of the sand and gravel cover placed according to the two specifications is shown in figure 2 along with the upper and lower limits for recommended cover.

Underdrains were needed along the conveyance channel in both specifications to lower the ground water in the event the channel needed to be dewatered once the channel was placed in operation. In the first specifications [1], Reach A required 12.2 km (7.6 mi) of

Table 1. - Specification requirements for PVC lining

Property	Required	Test method	Property	Required	Test method
1. Thickness, minimum.	0.019 inch.	ASTM: D 1593, paragraph 8.1.3.	6. Directional stability, each direction, maximum.	5.0 percent.	ASTM: D 1204, 15 minutes at 212 °F.
2. Specific gravity, minimum.	1.20	ASTM: D 792 method A.	7. Plasticizer:		
3. Tensile properties:		ASTM: D 882	a. Water extraction, maximum percent weight loss.	0.35 percent.	ASTM: D 3083, immersion in 122 °F distilled water for 24 hours.
a. Breaking factor, each direction minimum.	46.0 lbf/in		b. Volatile loss, maximum.	0.9 percent.	ASTM: D 1203, method A.
b. Elongation at break, each direction, minimum.	300 percent		c. Resistance to soil burial, increase in modulus at 100 percent elongation each direction, maximum.	10.0 percent.	ASTM: D 3083, 30-day soil burial.
c. Modulus at 100 percent elongation, each direction, minimum.	18.0 lbf/in				
4. Tear resistance (Graves), each direction, minimum.	6.0 lbf	ASTM: D 1004	*8. Bonded seam strength, tensile, each direction, minimum, percent of breaking factor.	80 percent.	ASTM: D 882
5. Low temperature impact.	Not more than 5 specimens out of 10 shall fail at -15 °F.	ASTM: D 1790	*9. Bonded shear strength, peel adhesion.	Film tearing bond (FTB).	ASTM: D 413, strip specimen, type A, 180° peel, 2-inch width, 2 in/min.

* Tests conducted on factory seams.

underdrains and [2] Reach B, in the second specifications, required 14.2 km (8.8 mi). The underdrains were placed in a trench near the outside edge of the operating road on both sides of the channel. Each drain was independent of the other, starting with a 15.4-cm (6-in) and increasing in size to a 35.6-cm (14-in) perforated, corrugated plastic pipe which drained into a concrete pipe sump. The length of each drain was approximately 701 m (2,300 ft). The pipe trench is excavated with the pipe placed in a gravel envelope and backfilled in one operation. The trenches were located away from the channel prism so that the channel area could be dewatered during construction operations.

The total length of the conveyance channel is 64.4 km (40 mi). Reach A and Reach B are 33.9 km (21.1 mi) in length with 30.4 km (18.9 mi) for future construction. Some typical costs for Reach A and B are as follows:

The work for Reach A was awarded in February 1983 and will be completed in January 1985. Reach B will be awarded in March 1984 and will completed in August 1985.

		UNITS	Reach A	Reach B
1.	Excavation for conveyance channel	0.76 m^3 (yd^3)	$ 1.55	$ 1.35
2.	Excavation for borrow	0.76 m^3 (yd^3)	1.20	1.50
3.	Overhaul	Mile yd	0.72	1.00
4.	Furnishing and constructing corrugated plastic drainpipe			
	15.2 cm (6 in)	30.4 cm (ft)	13.60	5.80
	20.3 cm (8 in)	30.4 cm (ft)	14.20	9.30
	25.4 cm (10 in)	30.4 cm (ft)	15.20	11.90
	30.5 cm (12 in)	30.4 cm (ft)	17.00	15.10
	35.6 cm (14 in)	30.4 cm (ft)	20.80	18.00
5.	Furnishing and placing 48-inch-diameter precast manholes for pipe drains	each	6000.00	5500.00
6.	Preparing subgrade in earth material for PVC lining	0.836 m^2 (yd^2)	1.02	0.25
7.	Furnishing and placing 20-mil PVC lining	0.836 m^2 (yd^2)	1.13	1.12
8.	Placing earth cover material on lining	0.836 m^2 (yd^2)	0.95	1.00
9.	Furnishing and placing sand and gravel cover material	0.836 m^2 (yd^2)	6.10	5.75

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1. Bureau of Reclamation Solicitation/Specifications 3-SB-5D-00260/DC-7553, Conveyance Channel, Stage 1-2, Reach A, San Luis Valley Project, Colorado.

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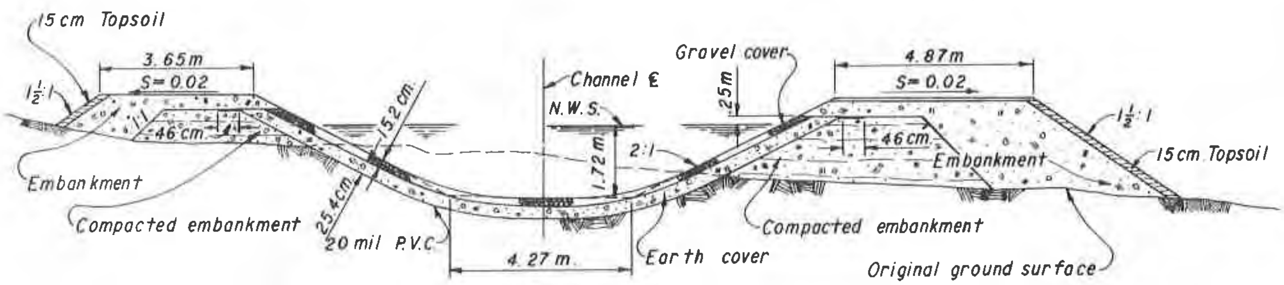
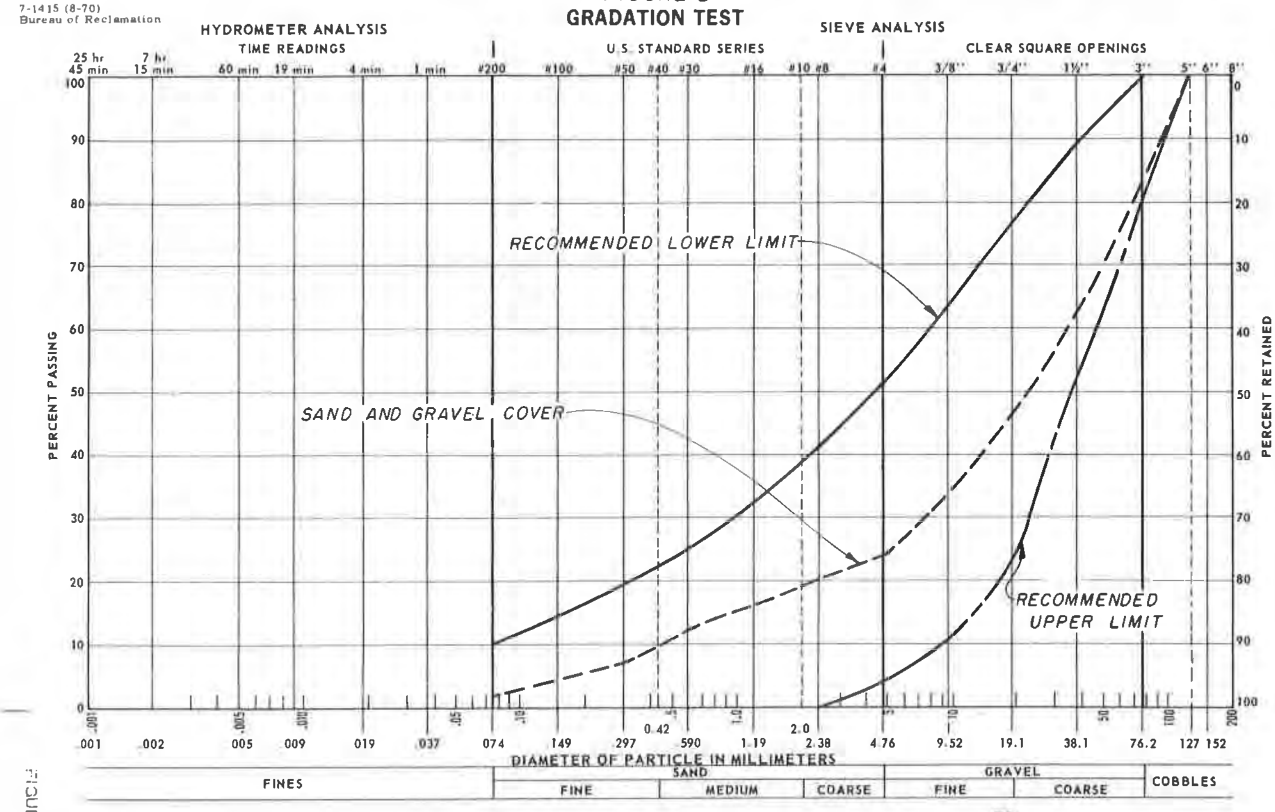


TYPICAL SECTION
FIGURE 1

FIGURE 2
GRADATION TEST

7-1415 (8-70)
Bureau of Reclamation



CLASSIFICATION SYMBOL ________ ATTERBERG LIMITS SPECIFIC GRAVITY NOTES________

Gravel 74 % Liquid Limit ______% Minus No. 4 ______

Sand 24 % Plasticity Index ______% Plus No. 4

Fines 2 % Shrinkage Limit ______% Bulk ______ Apparent ______

SAMPLE NO.____________ HOLE NO. T. P. 7 ________ DEPTH ________ ft (________ m)

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Joint U.S./U.S.S.R. Field Studies on Canal Linings at Ukrainian Field Test Station, Black Sea Canal, U.S.S.R.

As part of the joint U.S./U.S.S.R. program on Plastic Films and Soil Stabilizers, a field study was conducted on various canal lining systems in experimental sections. Nine test sections were installed. These included PVC, PE, and polyolefin membranes with earth, concrete, and shotcrete cover; two synthetic soil sealants were studied. For this study, the U.S. designed and furnished all materials. The Soviets constructed the ponds according to U.S. specifications and collected data which was jointly analyzed. Seepage tests were conducted for two irrigation seasons, 1977 and 1978. A seepage control effectiveness factor was determined for each lining system. Results indicated that 20-mil polyolefin plastic film with concrete cover followed by 10-mil PVC plastic film with concrete cover were the most effective lining systems. Both systems have been selected for further joint evaluation.

INTRODUCTION

The joint study in the use of plastic films for canal linings and polymers for the stabilization of construction slopes against erosion was made possible by the Soviet-American Joint Commission on Scientific and Technical Cooperation Program, signed at Moscow, May 1972. The main objective of this cooperation was to provide broad opportunities for both countries to combine the efforts of their scientists and specialists in working on major problems whose solution will promote the progress of science and technology for the benefit of the world.

Under this broad program of science and technology cooperation, several areas of joint study were identified. One was Water Resources, which included the joint study on Plastics in Hydrotechnical Construction, with H. G. Arthur, Director of Design and Construction, Bureau of Reclamation, as U.S. coordinator, and P. B. Sviklis as coordinator for the U.S.S.R. The work was completed in 1982 [1], [2], [3], [4].

Specific objectives of the joint program were:

1. Improvement of long-term communications between the Soviets and the U.S. for continued transfer of technology on this topic

2. Joint preparation of improved specifications and testing methods of plastic films for canal and reservoir lining

3. Joint preparation of improved design and construction procedures and methods for canal lining with plastic films

4. Promotion of improved manufacture and fabrication of plastic films for lining canals and reservoirs

5. Joint preparation of improved field methods and selection of materials for the use of polymers for soil stabilization in construction of water resource projects.

To achieve these objectives, a U.S. team composed of representatives from Federal agencies and private industry was formed to carry out specific activities with the U.S.S.R. The U.S. team provided expertise in laboratory and field research, design and construction engineering, and the manufacture, fabrication, and installation of plastic film materials.

During the visit by the U.S. team in 1975, Dr. P. I. Kovalenko, Director, UKrNIIGim (Ukrainian Research Institute of Hydraulic Engineering and Land Reclamation) suggested a joint study on various canal lining systems. It was agreed that the U.S. team would furnish design specifications and materials necessary for the construction of nine test ponds. The U.S.S.R. agreed to construct these ponds to U.S. specifications using materials produced in the United States. The U.S.S.R. team furnished detailed construction data and later established seepage data for each pond.

The southern area of the Ukrainian S.S.R., where intensive construction of large irrigation systems is underway, was selected as the test site for the suggested designs. The test ponds were constructed at an experimental field station near the Black Sea Branch of the North Crimean Canal.

For this study, three plastic lining materials were selected. These included:

1. 10-mil PVC film. This plastic was the most widely used material in the Bureau of Reclamation's membrane lining work. The Bureau is now specifying 20-mil PVC.

2. 10-mil PE film. Plastic linings are more widely used for canal and reservoir linings in the Soviet Union than in the United States. For seepage control in their canals the Soviets use PE.

3. 20-mil PO (polyolefin) film. Preliminary laboratory tests conducted on this newly developed lining (circa 1975) indicated that it had merit for use in seepage control work and was selected for this study to further evaluate its effectiveness.

Since plastic linings require some type of cover to protect them from the elements and physical damage, this study provided an opportunity to evaluate several different protective cover systems. These included concrete, shotcrete, and the conventional sand and gravel cover.

The concrete/plastic lining system was of interest for the following reasons: as a method to avoid overexcavation in expansive shales or gypsiferous soils and loessial soils, for elimination of joint sealant materials required for use with unreinforced concrete lining, and for avoidance of the need for sealing of extensive random cracks in unreinforced concrete linings. Also, during this period of time, UKrNIIGiM was working on developing equipment to place concrete and PE film in one operation. A movie documenting the developmental work was furnished to the U.S. team.

The shotcrete cover was selected because of potential application on steep canal slopes where concrete or earth materials could not be used.

In addition to plastic linings, two sealant materials were evaluated. These included a synthetic polymer sealant supplied in dry powder form which was mixed with the soil to form a compacted lining [5], and a spray applied urethane sealant. Preliminary studies conducted by the Bureau of Reclamation indicated that this sealant provided high compressive strength properties, excellent resistance to wave action, and good weathering characteristics. [6]

TEST PONDS

Nine test ponds were constructed by UKrNIIGiM in the spring of 1977 as follows:

Pond 1 - 10-mil polyvinyl chloride film having 380-mm (15-in) earth cover

Pond 2 - 10-mil polyvinyl chloride film having 150-mm (6-in) thick sand cover protected with an 80-mm thick unreinforced concrete lining

Pond 3 - 10-mil polyvinyl chloride film having 150-mm (6-in) thick sand cover protected with 80 mm of shortcrete

Pond 4 - 10-mil polyethylene film having 380-mm (15-in) earth cover

Pond 5 - 10-mil polyethylene film having 150-mm (6-in) thick sand cover protected with an 80-mm thick unreinforced concrete lining

Pond 6 - 20-mil polyolefin film having 380 mm (15-in) earth cover

Pond 7 - 20-mil polyolefin film having 150-mm (6-in) thick sand cover protected with an 80-mm thick unreinforced concrete lining

Pond 8 - Synthetic polymer sealant mixed with soil to form a 150 mm (6-in) compacted layer

Pond 9 - A 150 mm (6-in) thick compacted earth lining sprayed with a urethane sealant

The top measurements of the nine ponds were 12 by 17 meters at a depth of 2 m (39.4 by 55.8 at 6.6 feet) and side slopes of 2:1. Water depth in each pond was maintained at 1.47 m (4.8 ft). A view of the test ponds is shown in Figure 1. Drawings showing typical sections are presented in Figures 2 through 6.

Figure 1.- View of U.S. experimental ponds at Black Sea Canal Test Station

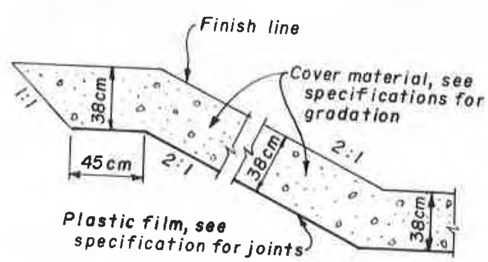

Figure 2.- Typical section for Ponds 1, 4, and 6.

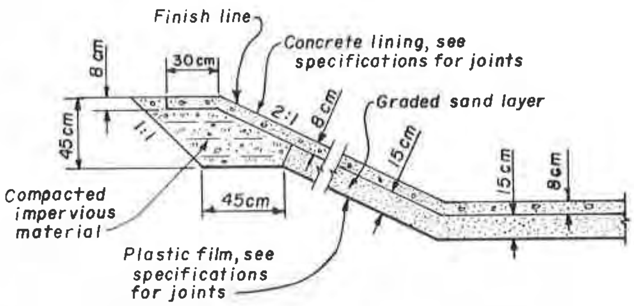

Figure 3.- Typical section for Ponds 2, 5, and 7.

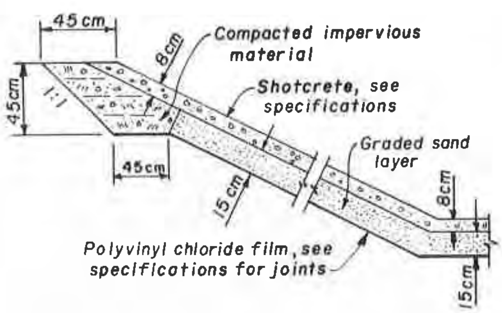

Figure 4.- Typical section for Pond 3.

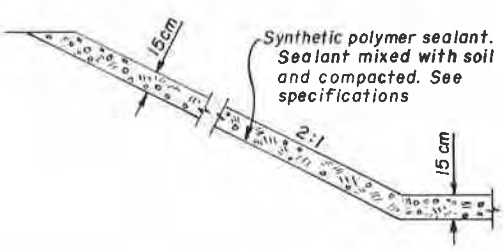

Figure 5.- Typical section for Pond 8.

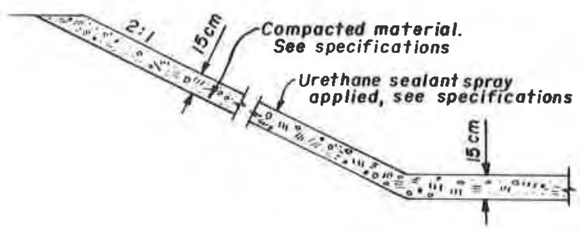

Figure 6.- Typical section for Pond 9.

SEEPAGE TESTS

Seepage tests were conducted by UKrNIIGiM for two irrigation seasons, 1977 and 1978. Ponds were equipped with water level recorders and the field station had a hydrometeorological unit for measuring evaporation and precipitation. Details on the methods used by UKrNIIGiM to calculate the seepage losses are given in reference 3.

The seepage control effectiveness of the nine lining systems for the two irrigation seasons is summarized in table 1. The effectiveness was computed according to the following relation:

$$E = \frac{V_s - V_1}{V_s} \times 100$$

where E = seepage control effectiveness in percent
V_s = seepage velocity through soil subgrade, m/day, and
V_1 = seepage velocity through the lining, m/day

Table 1 - Seepage control effectiveness of U.S. lining materials installed at Black Sea Test Station

Pond No.	1977 Seepage velocity, m/day			1978 Seepage velocity, m/day		
	through lining	through soil bed	E, %	through lining	through soil bed	E, %
1	0.0048	0.0109	56.0	0.0043	0.0107	60
2	0.0022	0.0122	82.0	0.0023	0.0123	81
3	0.0041	0.0124	67.0	0.0041	0.0124	70
4	0.0101	0.0140	28.0	0.0100	0.0143	30
5	0.0033	0.0153	78.0	0.0032	0.0157	80
6	0.0064	0.0151	58.0	0.0068	0.0156	56
7	0.0021	0.0178	88.0	0.0017	0.0188	91
8	0.0063	0.0225	72.0	0.0282	0.0242	0
9	0.0045	0.0240	81.0	0.0254	0.0261	0

Note: E is the seepage-control effectiveness, percent

Results of the study indicate that plastic films with concrete covers were the most effective lining systems. Of the plastic films, polyolefin and polyvinyl chloride were the best performers and were selected for further evaluation in a test reach on Lateral R-5-1, Kakhovka Project, Ukraine S.S.R.

After two years of service, the concrete lining was in good condition with only minor hairline cracking noted in several isolated areas. The shotcrete lining exhibited some shrinkage cracking, with the longest crack up to 1 mm deep.

Of the three protective covering systems, the sand and gravel cover performed the poorest. Some sloughing and deformation of the cover was observed. The gradation of the cover material, both specified and actually used, is shown in Figure 7. As a result of this study, the Bureau of Reclamation is now specifying a coarser gradation [7].

The urethane coating and synthetic sealant performed satisfactorily during the first season with a seepage control effectiveness of 81 and 72 percent respectively. However, during the second season these linings lost 100 percent of their seepage control effectiveness. This was primarily due to damage from wetting and drying and freezing and thawing action that occurred during the winter time. The urethane coating suffered the most damage with breakings and buckling occurring to the lining.

The results are summarized in Tables 2 and 3.

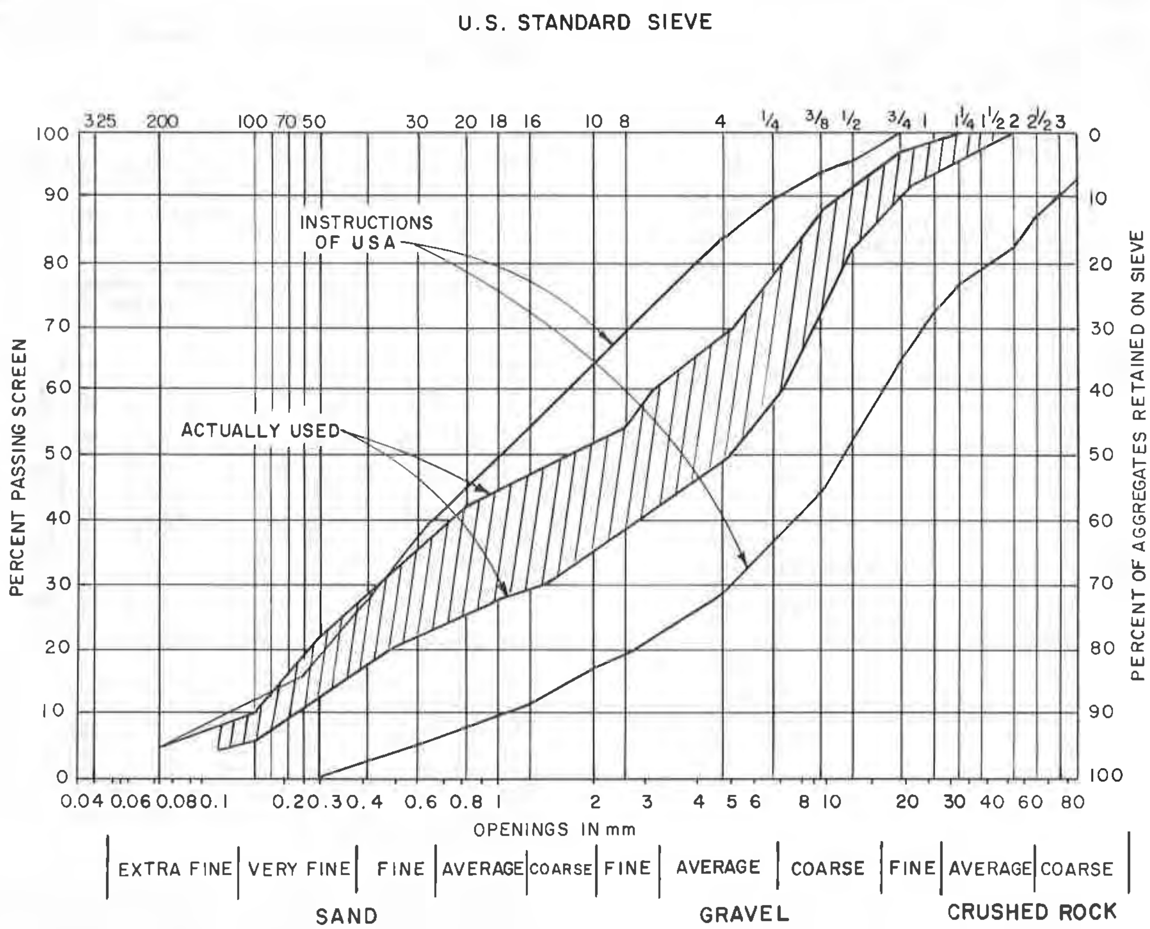

Figure 7.- Gradation of sand and gravel cover for plastics linings.

Table 2 - Seepage control effectiveness - 1977

Order No.	Lining design	Seepage-control effectiveness, %
1	Polyolefin film under concrete coating	88
2	PVC film under concrete coating	82
3	Urethane coating	81
4	Polyethylene film under concrete coating	78
5	Synthetic sealant M-179 mixed with soil	72
6	PVC film under shotcrete coating	67
7	Polyolefin film under sand and gravel backfill	58
8	PVC film under sand and gravel backfill	56
9	Polyethylene film under sand and gravel backfill	28

Table 3 - Seepage control effectiveness - 1978

Order No.	Lining design	Seepage-control effectiveness, %
1	Polyolefin film under concrete coating	91
2	PVC film under concrete coating	81
3	Polyethylene film under concrete coating	80
4	PVC film under shotcrete coating	70
5	PVC film under sand and gravel backfill	60
6	Polyolefin film under sand and gravel backfill	56
7	Polyethylene film under sand and gravel backfill	30
8	Urethane coating	0
9	Synthetic sealant M-179 mixed with soil	0

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[7] Morrison, W. R., and Starbuck, J. G., Performance of Plastic Canal Linings, REC-ERC-84-1, Bureau of Reclamation, Denver, Colorado, 1984

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Case Study of an In Situ, Uninterrupted Flow Repair of a Concrete Sluice Channel

A reinforced concrete conveyance channel at a paper mill in South Africa suffered progressive attack and degradation from the mill's bisulphite rayon pulp process effluent. The deterioration eventually reached such a degree of severity that the structural integrity of the concrete channel was threatened, raising the specter of a costly plant shutdown for an equally expensive channel reconstruction.

Because a plant shutdown was considered only a last resort, a proposal from Schlegel Lining Technology GmbH, Hamburg, West Germany, for lining the existing channel with a geomembrane installed during normal effluent flow, and without imposing additional loads on it, was made and accepted. Not only did the geomembrane provide an economic and environmentally safe solution, it also improved flow characteristics to the extent that today the channel handles greater volume of effluent than before it was lined. The geomembrane has provided, in effect, a new channel, at a fraction of the cost of rebuilding it in its original form.

PROBLEM/SOLUTION

The 3.5 kilometer long channel carries process effluent from the paper mill to a pumping station from where it is discharged into the Indian Ocean through a sea outfall plastic pipeline.

Before it was restored, the channel had been subjected to aggressive effluent attack for more than 25 years, and a thorough assessment of its structural and surface conditions became necessary. In recent years, it had become obvious that major repairs to the channel were essential if it was to be saved at all. A viable solution was, however, lacking. Meanwhile, the channel had degraded in places to a condition not normally permitted, requiring structural support and spot restoration.

The owner and his engineers imposed strict criteria for a channel restoration project. Specifically, the solution had to provide a channel that was:

1. Leakproof,
2. Environmentally safe,
3. Vapor tight,
4. Of equal, and preferably greater, capacity to the existing channel,
5. Restored in its existing site, the only position available for a gravity line to the ocean,
6. Maintenance free over the long term,
7. Repaired in as short a period of time as possible,
8. Capable of handling the increased effluent volume that would be produced after a plant extension,
9. Easy to clean, with no build-up of deposits that might adversely affect capacity,
10. Safe for pedestrian traffic where necessary (the channel is normally covered in several areas), and
11. Restored economically.

From another standpoint, rehabilitating the channel could not:

1. Cause any plant disruption,
2. Impose further structural loads on the channel,
3. Require substantial alteration of the existing precast concrete trough, now or in the future,
4. Rely on pumping equipment to handle the effluent while the channel was restored, or
5. Present a health or safety risk to personnel employed in rehabilitating the channel.

All proposed repair solutions were evaluated in light of these requirements and prohibitions. After careful study, the geomembrane relining proposal described below was selected and implemented.

The technology used was new. To the author's knowledge, an in-situ, uninterrupted flow repair of a channel of this nature with a geomembrane has never before been performed.

THE MATERIAL

It is this author's conviction that large area lining should always be carried out with the largest available single sheets, as this minimizes risk through minimizing seam lengths. The geomembrane used in this application was manufactured in sheets 4.5 m wide by 9.5 m long, and installed in this size after being formed on site to the U-shape of the channel.

The material that was used was polypropylene, manufactured to a thickness of 5.0 mm. Such a thickness was considered necessary because the degraded rough surface on the inside of the channel would accept only a lining that would support itself with little deformation under effluent loading. The geomembrane is not in any way bonded to the concrete inside the channel. The strength of the polypropylene in this thickness also negated the need for additional structural support to the concrete channel, beyond some elementary restoration work and backfilling. The mechanical properties of the polypropylene resin that was used to manufacture the geomembrane are shown in Table I.

TABLE I
Selected Mechanical Properties

PROPERTY	TEST METHOD	VALUE/UNITS (SI)
Impact Resistance 23° (Charpy impact flexural test)	ISO/R 179 Test specimen acc. to fig. 2	No Break
Impact Resistance 23° (notched) (Charpy impact flexural test)	ISO/R 179 Test specimen acc. to fig. 2	25mJ/mm²
Impact Resistance (notched) (Izod impact flexural test)	ISO/R 180 Method A ASTM D 256 Method B	250 J/m of notch
Flexural Stress at Conventional Deflection	ISO/ 178 Standard Test specimen 3.1	20 N/mm²
Tensile Stress at Yield	ISO/R 527 Speed C Test specimen acc. to fig. 2	21 N/mm²
Tensile Stress at break		40 N/mm²
Percentage Elongation at Break	ASTM D 638 Speed C Test specimen Type 4	800%

The chemical resistance of the material was taken from industrial experience; the raw material supplier also confirmed the material's resistance to the effluent at a constant temperature of 60°C and projected a service life under these conditions of more than 20 years.

PREPARATION

Starting at the plant end of the channel, the inside of the trough was first cleaned of decades of accumulated debris. A high pressure water jet cleaning system, mounted on a frame shaped to the profile of the channel, scraped deposits and loose and degraded concrete. To ensure that the action of the water jets on the concrete was not too aggressive, the jets were angled acutely to the concrete, to produce a spading action. A debris screen was placed downstream of the cleaning system to trap deposits and debris; if they had been carried away by the effluent stream, they might have damaged equipment at the ocean pumping station. After they were cleaned, the upper side walls of the channel were recapped with concrete to provide a smooth surface to which the geomembrane would later be fixed.

INSTALLATION OF THE GEOMEMBRANE

A specially engineered gantry was designed and built as the primary tool for lowering the liner into the flow and positioning it correctly in the channel. The gantry moved along tracks that had been installed on top of the channel walls. It was designed so that the geomembrane could be handled and submerged automatically without the need for manpower to force it into position below the effluent. The gantry provided accurate horizontal positioning of the geomembrane; effluent flowed through the shaped section during this installation procedure.

The polypropylene sheeting was trimmed to slightly more than the girth of the channel. Enough material extended above the channel walls so that covering pieces could later be joined to it.

Once the geomembrane had been bent and shaped, a specially designed stainless steel coupling was fixed to one end. When the sheet was lowered into the channel by the gantry, it met and was fastened to the previously placed section of geomembrane with a system of gasketing and specially designed bolts. Thus it was possible to join geomembranes under water; welding of plastic in such conditions is not yet feasible. The bolts were tightened with another piece of custom-designed equipment that traveled under water from bolt to bolt, locating the studs and guiding sockets and nuts on to them. The device tracked along a rib in the sheet coupling frames and was adjustable to specific torque settings.

When the geomembrane was installed in a curved section of channel, adjustments were made to the gantry to accommodate the radius of the curve. Angles were also cut in the geomembrane to negotiate the curve.

In the lower corners of the channel, a supporting pipe was inserted beneath the sheet to guarantee a specific calculated radius for the geomembrane, which accepts the full load of effluent flow at that spot. A leak detection system, in the form of small control pipes, was also insatalled at regular intervals behind the steel U-frames. The owner is thus able to test the water-tightness of the lining system at any time.

Computer-modeled loading calculations were made and presented to the owner. This mathematical modeling showed that the geomembrane would be subjected to the following loads:

1. Bending by forming (cold)
2. Bending under elevated temperature
3. Bending under the load of the effluent.

Calculations of static loads were made based on computer programs for finite elements. The programs also made allowances for the bending which takes place in the corners of the channel and where the sheet bearing on the concrete is gripped by the coupling frames. The results of this modeling showed that all related stresses were within the material's allowable limits for long-term stress-strain behavior. The calculations also showed that the correct placement of the supporting piping was in the lower corners of the channel; this positioning was designed into the system.

COVERING

From an environmental point of view, there were numerous arguments to be made in favor of a completely covered channel: it would prevent animals or people from falling into it; a cover would contain noxious fumes; and the cover would protect the channel from vandalism and accidents. For those lengths of the channel where human and animal traffic was expected, precast concrete slabs were laid progressively along the top of the channel as the lining operation was completed, offering a permanently trafficable surface. The slabs were placed on top of the lengths of covering geomembrane, which protect the slabs from vapor corrosion. The geomembrane was, in effect, fabricated into an envelope through which process waste flows. The precast concrete cover panels were manufactured in accordance with the design of the engineer and the civil works contractor.

INSTALLATION CHALLENGES

Access to the channel was extremely limited in some areas. A railway runs parallel to the channel at a distance of only six feet; all fencing could not always be removed. In other spots, a railway siding had to be underrun, and some road crossing had to be accepted. Completion of the project took approximately five months and required intimate cooperation among Schlegel Lining Technology, GmbH, the geomembrane system supplier, Empro Ltd., Schlegel's South African distributor, the customer and its engineers and civil works contractors.

CONCLUSION

Rehabilitating the concrete channel by lining it with a geomembrane was approximately a third of the cost of building another system. Of even more economic benefit to the owner, however, the improved flow characteristics obtained with the geomembrane was a key factor in the capacity expansion of the paper mill. The facility is able to release nearly double the volume of effluent, through the same discharge system. Without the relining, a plant extension would have been unthinkable.

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Design Aspects of Flexible Revetments Constructed on Impermeable Synthetic Linings

This paper describes some principles of design for impermeable lining systems for rivers or canals. In particular, it addresses the problems anticipated where large scale mining subsidence is expected to occur in the vicinity of a river, and suggests design detailing to cope with it.

The author describes a five-layer composite lining suitable for major undertakings, together with the functions and laying/jointing recommendations for each layer.

1. INTRODUCTION

This paper attempts to approach some of the principles of design for an impermeable lining system for rivers over areas potentially subject to catastrophic mining subsidence.

Such a situation might arise where, for example, very thick coal workings are taking place beneath a faulted zone with thin soils cover to rockhead. It might also arise where large-volume extraction of metalliferous ore bodies is to take place, with the consequent possibility of pillar failure and catastrophic surface collapse.

2. LINING OBJECTIVES

In the case of a lining scheme specifically required for a mining situation, the loss of water per se would not be important, but the consequences of the disappearance of that water into the active mine workings below would be catastrophic. Furthermore, the consequences of the river flow disappearing (even if only temporarily) would be substantial on the downstream stretches of the river.

In view of the economic nature of mining projects, and the psychological factors associated with the world-wide investment in the mining industry, it is considered that a lining proposal carries not only the requirement that the river should be effectively lined to prevent the possibility of water ingress during mining activities, but also that any scheme should be so well engineered and so substantial that it can clearly be seen and arguably so, that the possibility of catastrophic water entry into the mine does not exist. It is therefore to be expected that, if a lining system be commissioned, it would be of the highest technical quality and best material quality available. We also would expect it to be thoroughly researched and properly designed.

3. ASSESSMENT OF HYDROLOGICAL SITUATION

It is important to obtain at least one year's information on the ground-water regime, together with a minimum of one year's information on the flow pattern of the river in question. One year is the minimum time with which it is possible to consider working, and usually several years or many years of information should be made available for this kind of work.

In particular, of course, the regularity of cut-off peaks and summer drought flows are important for the assessment of the practicalities of installation.

4. HYDROLOGICAL SITUATION IF LEAKAGE OCCURS INTO A MINE

It is often the case that general/regional ground-water levels have fallen as a result of general mining activities in any mining area. This establishes a primer facie case that, mining could induce a situation such as shown in Fig. 1. In this diagram, it can be seen for the author is postulating the opening-up of joint systems above the mine workings. In this event, as water passes downwards into the mine void, the water table surfaces become depressed. Note that the river acts as a water source. Although there is a slight danger of some soil piping taking place as a result of natural ground-waters entering the mine workings, there is a considerably higher danger of piping occurring if large quantities of river-water start to pass downwards into rock fissure systems connected with the mine void.

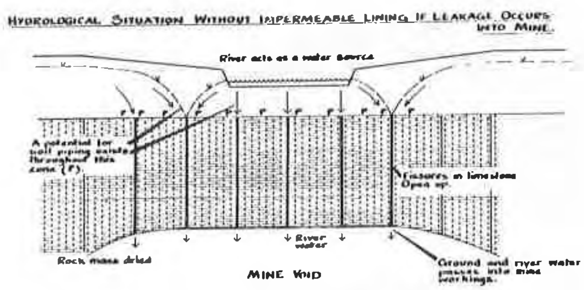
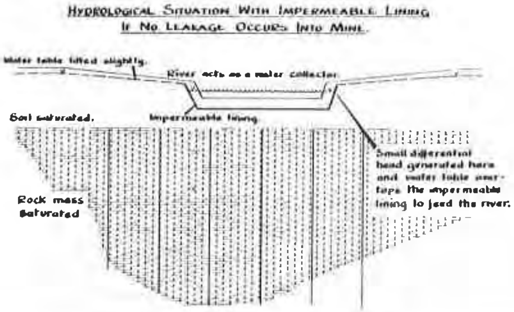


FIG. 1

5. HYDROLOGICAL SITUATION WITH IMPERMEABLE LINING IF NO LEAKAGE OCCURS INTO A MINE

Figure 2 shows the situation envisaged, where the introduction of an impermeable lining will act as an obstruction to the collecting function of the river. Therefore, a) hydrostatic inbalances must be catered for at the river edges. b) Hydrostatic uplift forces must be catered for on the river bed. c) Water-flow nets must be checked and a suitable sized collector designed with vertical inflow capability and horizontal transporter capability to allow 'over-topping' in a limited vertical space.

FIG. 2

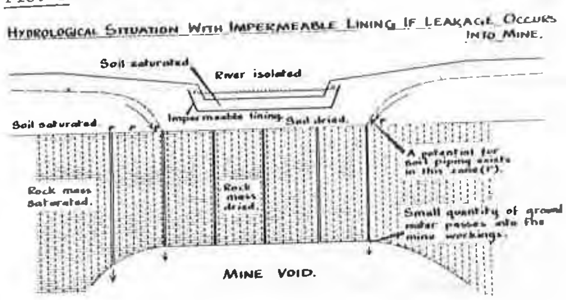


6. HYDROLOGICAL SITUATION WITH IMPERMEABLE LINING IF LEAKAGE OCCURS INTO A MINE

This situation is described in Fig. 3. This shows the author's envisagement of the hydrological situation where the rock mass above the mine void has become dry as a result of the opening of fissures through subsidence.

It is to be expected under this situation that the impermeable lining will effectively contain the river, but that the local ground-water table will fall as ground-water passes down into the mine workings. Consequently, there remains the possibility of soil piping as a result of ground-waters transporting soil particles down into the mine workings, but the limited potential source of ground-water volume, once the river has been isolated, is very small and the risk is no greater therefore, than in any other normal mining situation as exists in any other part of a mining operation.

FIG. 3



7. SPECIFIC DESIGN PARAMETERS

7.1 Land Ownership

One of the fundamental premises of being able to install an impermeable design, is that access to the river bed and to each bank is available. Designs can be made, to be handled from one side only, but these become more-complex and expensive.

Land boundaries often run down the middle of rivers, and considerable care should be taken before the preparation of any lining scheme, to ensure that matters of land ownership and access are clearly and suitably clarified.

7.2 Positions of Geological Faults

The position of rockhead outcrop of any major geological faults cutting the line of the river, must be known in detail. It is along such lines of weakness that postulated ground movements will take place, for which the supporting membrane stresses will have to be calculated.

In practice, the detailed positions of faulting may never be located, and the design approach may well revert to the consideration of the worst single fault movement event, and the consequent design of resitivity capability for such an event, be incorporated into the full length of any scheme.

7.3 Amount of Subsidence to be Catered For

A full, and detailed study of the future mining proposals beneath the river, is essential to predict the expected stress/strain history, of the river channel, and this should be compared with as detailed an understanding as possible of the local geology, to consider the possible modes and extent of failure which might take place in the mine workings. This comparison will allow an estimate to be made of the likely magnitude of failure together with the up-to-surface propagation characteristics of each particular failure mode. For example, a long-wall coal mining procedure will involve the certain generation of some moderate strains and ground movments, and will therefore almost certainly represent a low level of ground failure. Nonetheless, the overall possible amount of ground failure will be small.

Alternatively, stoping processes allow controlled mine movement to take place, with little surface expression. Under certain circumstances, one might expect the surface expression of a metalliferous stoped deposit to be considerably less than that of a long-wall planar deposit. Nonetheless, the potential for large-scale catastrophic failure with substantial surface expression is much higher for the former than for the latter.

7.4 Downstream Property

There are always downstream properties to be considered, and which may cause some direct limitations on the required design parameters. Considerable legal, social, and technical care must be exercised in the consideration of the consequences of the proposed work design on the owners or users of downstream property and the property itself.

7.5 Period Available for the Works

As mentioned earlier, a limited period for the works will have to be assumed, when water levels are at their regular lowest. Schemes can be put together for re-lining a river within the river whilst it is actually flowing. These are expensive and difficult to design. The easier system is to operate a nearby sheet lined diversion. This diversion is cheapest and easiest to control during the dry weather period of river flow. Such a diversion need not necessarily be large, especially if it is only to cater for the summer flow, and might not even involve excavation, being temporarily impermeably lined to prevent erosion and water loss. The returning flow would be diverted slowly into the main channel, to check it's efficacy, but it would be recommended that the diversion channel be maintained through the first winter and the reclaiming of the sheet piling, backfilling and landscaping be undertaken during the following summer.

8. PRELIMINARY OPERATIONS

8.1 Land Preparation

On both sides of the river, topsoil will have to be stripped off. This will involve the allocation of stockpile areas, and stockpile should not exceed two metres in height, in order not to damage the biological functioning of the internal soil organisms.

8.2 Other Requirements

In any operation of this kind, there will be a requirement for constant river crossing to take place by both equipment and for the purpose of transporting materials.

8.3 Feed-in Tributaries

The detailing of any junctions necessary within the lining works, of any feed-in tributaries or pipes, should be made at an early stage.

8.4 Construction of End Units

Once the river diversion has been executed, then the isolated section of the river can be pumped dry and the construction of two concrete end units can take place. The end unit has several functions, including:-

a) To prevent under-cutting of the lining by the river once re-established.

b) To hold the impermeable lining to the river bed at both ends.

c) To allow the dissipation of any excess pore pressures which may attempt to establish themselves behind the lining subsequent to construction.

Rivers are often subject to re-grading activity, and the presence of downstream weirs which should be investigated, or the alternative effect of the downstream concrete end of the design should be borne in mind in terms of it's own possible effect as an erosion-preventing weir. Considerations of bank protection works in the form of sheet-piling or revetments should be made in view of the possibility of re-grading erosion activities on the downstream end of the structure.

9. DESCRIPTION OF COMPOSITE SYNTHETIC LINING

9.1 Layered Functions

The layered functions of a suitable lining are described on Fig. 4.

The design concept is that the use of a multi-layer composite will meet correctly the many different requirements of the application environment. In brief, it is not possible to find any one membrane that will perform all the required design functions.

9.2 Selection of Environmental Properties

Given the existing range of synthetic fabrics available today, it is unlikely that a normal river environment in which the proposed structure would be built, would be considered as particularly aggressive. It would be expected that the products which the author envisages as being used in such works, by virtue of their thickness and high-strength, should work satisfactorily for a period of at least 20 years.

FIG. 4

PROPOSED SPECIFICATION RANGES

(Final Specification To Be Defined)

VERY STRONG PERMEABLE MEMBRANE OR WEBBING MAT
YOUNG'S MODULUS AND ULTIMATE TENSILE STRENGTH TO BE CONSIDERABLY HIGHER THAN FOR THE UF AND LF BUT LOWER THAN THOSE OF LPM PROBABLY ABOUT 25 TONNES/M JOINTING SYSTEM MUST DEMONSTRATE ADEQUATE STRENGTH TRANSFERENCE HIGH UV RESISTANCE HIGH ABRASION RESISTANCE DETERIORATION RESISTANCE DEGRADABILITY TO BE TAKEN INTO ACCOUNT IN STRENGTH CALCULATIONS

A THICK FELT WITH YOUNG S MODULUS AND ULTIMATE TENSILE STRENGTH TO BE CONSIDERABLY HIGHER THAN FOR THE CM PROBABLY A POLYESTER OR POLYPROPYLENE NEEDLE-PUNCHED MATERIAL APPROX 5-10 mm THICK

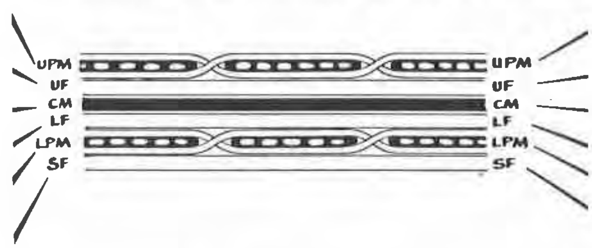
AN IMPERMEABLE MEMBRANE
PROBABLY <1 mm THICK WITH EXTENSIBILITY >100 % AND HIGH ULTIMATE TENSILE STRENTH MUST HAVE EXCELLENT AND PROVED JOINTING SYSTEMS WITH HIGHER BOND STRENGTH THAN THE SINGLE MEMBRANE PROBABLY BUTYL RUBBER OR POLYETHYLENE

THICK FELT PROPERTIES AS FOR UF

PERMEABLE MEMBRANE PROPERTIES AS FOR UPM, BUT UV RESISTANCE NOT CRITICAL YOUNG S MODULUS AND ULTIMATE TENSILE STRENGTH TO BE CONSIDERABLY HIGHER THAN UPM IN ORDER TO ABSORB MAIN STRUCTURAL STRESSES PROBABLY OF THE ORDER OF 50 100 TONNES/LIN METRE IN BOTH DIRECTIONS WITH LOW CREEP CHARACTERISTICS

THICK FELT TO ACT AS A SACRIFICIAL ABRASION PROTECTION BELOW THE LPM WHERE LPM IS PLACED DIRECTLY ON ROCKHEAD AND DRAG WOULD BE EXPECTED UNDER DIFFERENTIAL MOVEMENT CONDITIONS

PROPOSED CROSS-SECTION OF COMPOSITE IMPERMEABLE LINING MEMBRANE



PROPOSED FUNCTIONS

UPPER PROTECTIVE MAT (STRONG)
TO TAKE THE DIRECT PLACEMENT OF BOULDERS AND ROLLING OF BOULDERS, TO RESIST ANY ACCIDENTAL SURFACE DAMAGE (MAN MADE), TO ABSORB THE EXPECTED SUBSIDENCE-INDUCED TENSION AWAY FROM THE FELT AND IMPERMEABLE MEMBRANE, PERMEABLE

UPPER FELT (THICK)
TO RE-DISTRIBUTE POINT LOADS FROM ABOVE, TO ACT AS A SMOOTH 'LUBRICATOR' BETWEEN THE MAT ABOVE AND THE CENTRAL MEMBRANE, PERMEABLE TO ALLOW WATER THROUGH TO ASSIST IN LUBRICATION AND REDUCE SUBSIDENCE-GENERATED PRESSURE DIFFERENTIALS, TO ACT AS A SECOND SAFETY BARRIER IN THE EVENT OF FAILURE OF THE IMPERMEABLE MEMBRANE, SINCE IT'S PERMEABILITY IS LOW, AND IT WILL CLOG FURTHER AND BECOME SELF-SEALING IN THE EVENT OF PIN-HOLE LEAKS, TO ACT AS A CUSHION LAYER BETWEEN THE JOINTS IN THE UPPER MAT AND THE CENTRAL MEMBRANE

CENTRAL MEMBRANE (IMPERMEABLE)
TO HAVE EXCELLENT JOINTING CHARACTERISTICS, TO HAVE HIGH STRENGTH AND EXTENSIBILITY, TO LAST THE REQUIRED LENGTH OF TIME WITHOUT DETERIORATION

LOWER FELT (THICK)
TO RE-DISTRIBUTE POINT LOADS FROM BELOW, OTHER FUNCTIONS AS FOR UPPER FELT

LOWER PROTECTIVE MAT (STRONG)
TO ABSORB POINT LOADS FROM BOULDERS BELOW OR FROM DIRECT ROCKHEAD PLACEMENT, TO ABSORB SUBSIDENCE INDUCED TENSION AWAY FROM THE UPM, THE FELTS AND CENTRAL MEMBRANE, TO RESIST DAMAGE FROM BEING DRAGGED OVER UNDERLYING MATERIAL, PERMEABLE TO ACT AS PORE WATER PRESSURE DISSIPATOR

SLIDING FELT (THICK)
REQUIRED BENEATH LPM TO ALLOW THE LATERAL MOVEMENT OF THE LPM WITH MINIMUM ABRASION ie TO ACT AS A LUBRICATOR BETWEEN THE MAIN STRUCTURAL ELEMENT AND THE ROCK BELOW, AND TO ACT AS A FINE FILTER ELEMENT

It is possible that the 5 underlayers would be exposed to sunlight for short periods of time, but the author does not expect this to have any significant effect on the membranes whatsoever provided that such periods were of the order of less than two weeks. Furthermore, the uppermost 'upper protective membrane' might probably be exposed to sunlight for a period of several months. In this event we would expect that any product chosen for this work would be such that this kind of U.V. exposure would have no significant effect on the products tensile strength performance.

9.3 Selection of Tensile Properties

Consideration of the basic principles of composite layered structures brings to author to the conclusion that the prime tensile function of the design will be performed by the fifth layer down in the composite, being the lower protective membrane. In order to ensure correct inter-layer geometry during defomration, this lower protective membrane must be in principle, stronger than any of the other layers, and it is suggested that the strength of twice that of the upper protective membrane, would be suitable. The relative order of strength which would be required in the design are shown in Fig. 4, in fact, it is not just the ultimate strength which must be greater, but the Young's Modulus of each layer must also be in the same proportion.

Considering that the fundamental tensile work would be undertaken by the lower protective membrane, and that this type of material would have an ultimate extension at failure of about 10 - 12%, then one only has to envisage the order of magnitude of stresses to appreciate that this membrane layer will have to be exceptionally strong. For example, the unit square metre weight bearing down directly on the membrane during winter flow conditions will approximate in a relatively small river to 6 tons/sq.metre. If, for example, support is removed from beneath a 10 metre long strentch of this membrane, then catenary stresses of the order of 70 tons tensile/metre width, could easily be generated within that membrane. It is not impossible that either a single step fault, or a small graben-type rock movement could result in such a situation, and if this order of stresses could be imposed on the fabric, it seems prudent that a lower protective membrane should be selected with a 50% higher ultimate strength, in order to provide a satisfactory safety margin.

Normally, when considering working stresses for reinforcing geotextiles, a considerably higher requirement of safety is expected, above working stresses. However, in this kind of design, the working function is intended as a remedial temporary protection measure, which would be subject to on-going monitoring and civil engineering assistance if necessary.

The ultimate tensile strength of the central impermeable membrane is not as critical as the fact that it should have a much higher extensibility at failure than all of the other membranes within the structure. This will allow it to distort internally within the two felt 'sandwich' layers, whilst following the movment of the underlying lower protective membrane.

The lower protective membrane, in taking the main structural stresses of any movement, will have to physically move laterally or lengthwise in relation to the underlying ground. In this regard, it is assisted by the lower free-lying felt layer over which the stress absorbing membrane can move. this is intended to minimise any abrasion on the lower protective membrane, thus ensuring its integrity.

10. GROUND MONITORING

10.1 Contour Maps

A detailed topographical survey, with the production of accurate contour maps with contour intervals of less than 0.5 metres, is essential before design work of this type can be undertaken.

10.2 Water Tables

It will be necessary to monitor, using piezometers, the effect of construction works on phreatic surfaces. This is done to ensure the supply of information on sub-membrane pore-water pressures. These should be checked at an early stage after installation of the impermeable lining, to ensure that first assumptions on the requirements for counter-weighting, are correct.

10.3 Level Stations

In order to monitor the effects of mining subsidence on the structure, it is recommended that a number of accurate levelling and positioning points be paced on the upper banks of the area surrounding the proposed re-lined channel.

10.4 Tension Gauges

It is quite feasible to attach strain gauges to the tensile members of the composite geotextile, in order that on-going stresses can be monitored to ensure a fieldback of functional information for comparison with design criteria.

10.5 Tilt Monitoring

Buried beneath the impermeable lining, on horizontal plates, a linear array of ground tilt sensing devices can be sacrificially installed, to provide an on-going detailed account of ground movements along the river channel, subsequent to construction. The rates of change of tilt can be correlated with actual level changes measured by the point survey mentioned previously.

11 PRINCIPLES OF DESIGN

11.1 Designed Response to Subsidence

The first step to ensuring minimisation of tensile stresses, is to allow for excess material to be built-in in the form of 10% slack, to be laid as a 'ripple' between each one-metre section of boulders. With a vertical loading of, say, approximately 6 tonnes/square metre, and a coefficient of friction approximating 0.3, then there would be a horizontal resistance to sliding of some 2 tonnes/linear metre. Step deformation would generate tensional stresses which would in turn allow the ripple to be drawn out of the fabric at incremental loading of 2 tonnes/linear metre down the length of the structure. A composite membrane having an upper protective layer with an ultimate tensile strength of 30 tonnes/linear metre would therefore be able to draw the 'ripple' out of the fabric for a distance of some 15 metres before rupturing. This would provide an extra 1.5 metres of fabric for absorption of subsidence-induced stresses.

11.2 Response to Undesigned Subsidence

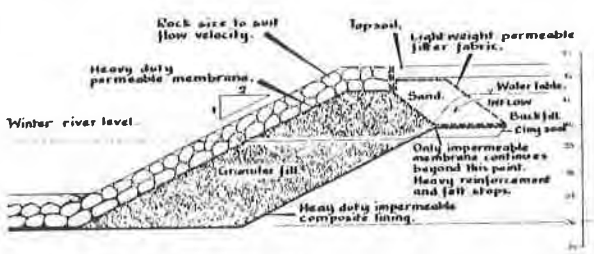
If the strength of both the lower protective membrane and the upper protective membrane exceed 30 tonnes/linear metre, then at least 1.5 metres of excess fabric can be drawn-out from any one side of a relative movement situation across a fault plane. Since in fact, it is likely that draw-out will take place from both sides of such a relative movement, then there is theoretically some 3 metres of excess fabric at any one point to cater for differential movement. Additionally, there would be the safety factor of any excess strength in the fabric beyond that suggested above. These are items which can be quantified and designed with specific safety factors. The design therefore has the capability of a positive and safe response to mining subsidence phenomena which are not part of the designed mining operation but which nonetheless can be assessed within certain specified limits.

11.3 Controllability during Unexpected Subsidence Events

As shown in Fig. 5, it is advisable to design a lateral extension to the impermeable channel lining. This horizontal extension should be constructed at such a level that requires no alteration in the event of subsidence lowering the ground by say, a designed 1 metre. However, if subsidence should continue beyond this point either slowly or dramatically, then it is possible for simple civil engineering works to be undertaken through the sand blanket above, in order to raise this impermeable sector and reconstruct the upper layers of the channel bank without having to purchase further quantities of special impermeable material and without having to undertake the specialised waterproof bonding processes necessary. It cn be seen that the lateral extension of this lining will relate to the specified limits of possible undesigned subsidence from the mine working activities.

FIG. 5

WINTER FLOW
BEFORE SUBSIDENCE



11.4 Design of Bank Geometry

The chosen bank geometry which the author considers to be suitable for most general purposes is shown in Fig. 5. There are three fundamental aspects to the slope geometry, being the slope angle; the bank height relative to the river bed and the size of rocks necessary to rsist the rolling effect of the river flow.

The criteria affecting the slope choice are practicality of laying with regard to the membranes, and secondly the friction coefficients between membrane and soil and between each membrane. If the slope is too steep, then it would be difficult to lay the fabrics and they will tend to slide down the slope.

The height of the bank is chosen in relation to the possible peak flow, and will be modified in terms of a final design, both in view of the ultimate peak flow expected and the centrifugal force of the water causing an upsurge on the outr sides of bends. The height of the slope is also vertically extended by the appropriate amount, in order to cater for future calculated mining subsidence at any point along the channel.

The rock size will also be finally chosen in relation to the ultimate expected edge velocities.

11.5 Repairability

The object of making the structure out of granular material is to ensure its flexibility under extreme subsidence conditions. In this regard, the overall structure should be substantially repairable after subsidence has taken place. Simple stone infill and re-levelling should suffice to repair this damage and no work should necessary beneath the structure unless it were felt that a particular void had been generated which could be suitably backfilled by grout. It is apparent that once the mine workings are abandoned and flooded, then there would be no problem generated even if the impermeability of the membrane became impaired subsequently.

11.6 Design against Potential Hydrostatic Imbalance Beneath the Channel Bank Lining

As can be seen in the given figures the upper edge of the impermeable membrane is bedded into a clay seal. This in order to prevent direct hydrostatic connection developing between the ground surface and the back of the impermeable membrane. Even a small connection could generate high hydrostatic heads which should be avoided.

11.7 Design Against Potential Hydrostatic Imbalance beneath the River Bed Lining

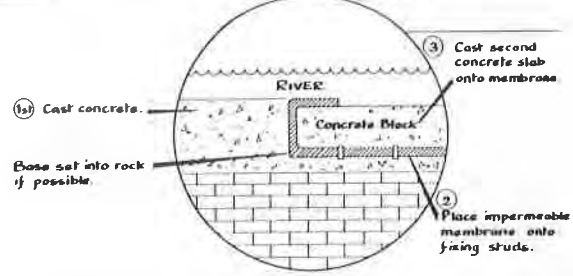
It is expected that hydrostatic pressures could build up beneath the impermeable membrane if allowance is not made for the dissipation. In this regard it can be appreciated that the strong lower protective membrane has been selected with a high in-plane permeability. Beneath this is a thick filter felt which will allow ground-water to enter through it at a rate in excess of the percolation of the natural soils. Once having passed through the external filter felt, ground-water will travel laterally along the interwoven voids of the lower protective membrane, and will dissipate outward at each end of the structure. It can be seen on the lower left-hand part of Fig. 6 that the design brings the entire composite membrane up through the concrete structure to the surface of the river bed. Pore water pressures can dissipate at these points, and the exact permeabilities and hydraulic gradients are calculated as part of the final design.

11.8 Collector Overflow Principle

The drawings illustrate the principle of overtopping, by which the river can continue to act as a collector. This is done by lifting the local ground-water table and allowing it to overtop through a sand filter into the granular fill.

FIG. 6

DETAIL (Not to Scale)



11.9 Source Overflow Principle

The attached drawings illustrate that, as an example, after one metre of subsidence, a nominal outflow of water is permitted above the top of the impermeable lining. The use of a two-metre thickness of sand guarantees a minimum hydraulic gradient for the outflow, and a concomitant minimal water-flow into the surrounding ground. Furthermore, it should be recognised that such outflow would only be possible within the more extreme areas of mining subsidence which could be very localised, and therefore the outflow effect would tend to be insignificant. It must also be borne in mind, that as part of the design, piezometers would be installed along the length of the channel, and if these suggested it necessary, then the extra length of impermeable lining can be raised to eliminate overflow and to establish satisfactory conditions.

11.10 Requirement for Piezometers

As mentioned above, it is recommended that a number of piezometers be installed along the length of any construction works. These piezometers would monitor ground-water pressures immediately beneath the impermeable lining under the river, and within the soils adjacent to the channel banks.
A part of the design concept is that the heavy-duty lower protective membrane forms a pore water pressure dissipator beneath the impermeable membrane.

12. LINING INSTALLATION

12.1 Laying of Sliding Felt

The sliding felt would be based along the river floor and bank with a 10% ripple which would be repeated on all the overlying layers. The felt would not be joined, but would be overlapped by at least a one metre overlap laterally across the river direction and with a three metre overlap in the downstream direction. It would be necessary to use temporary block weights to hold the ripple in place or some kind of clip on the fabric to maintain the ripple until the next layer was placed.

12.2 Lower Protective Membrane

This lower protective membrane would be laid on to the sliding felt with a 10% ripple and would be joined together in such a way as may have been determined by laboratory experiment to transfer the strain from one sheet to the next. If the membrane is of a substantial webbing type, each individual web strip would have to be mechanically joined to its corresponding neighbour to ensure full stress transference. During the course of placing the lower protective membrane, sensors and connecting tubes would be inserted for piezometers and water-presence detectors. Similarly other instrumentation such as tilt meters could be placed at this time. These are presently envisaged as being placed between the lower protective membrane and the sliding felt below.

12.3 Lower Felt

The lower felt would be laid on top of the lower protective membrane and would be joined mechanically together adjacent roll to adjacent roll. The joining method would probably be sewing, although if the fabric were capable of it, a combination of flame-welding followed by sewing would prove more effective.

12.4 Central Impermeable Membrane

Depending upon whether this membrane proves ultimately to be butyl rubber or polyethylene, the joining methods will be different. With polyethylene the method finally adopted will probably be continuous roll welding with 100% inspection, whereas butyl rubbers are often joined by adhesive bonding techniques. Butyl rubber certainly has the longest experience of any impermeable lining material, and has a very well-proved performance life. However, since the requirements of such a project may not be extensive in terms of years, it is possible that polyethylene could be considered as a suitable lining material.

12.5 Upper Felt

The upper felt would be laid in the same way as the lower felt, but on top of the central impermeable membrane. Like all the other membranes it would be laid with a 10% excess ripple. Special care would have to be taken during the laying of this upper felt to prevent accidental damage to the central impermeable membrane below.

12.6 Upper Protective Membrane

The upper protective membrane would be laid on to the upper felt with a 10% excess ripple and would probably be joined in a similar method to the lower protective membrane.

12.7 Placement of Boulders

During each of the laying processes of the underlying membranes, it is envisaged that artificial weights or clips would have been used to hold the ripple effect in place. Subsequent to the laying of the upper protective membrane, it would be necessary to import the final boulder layers immediately to hold the whole structure in place on the river bed. On the bank slopes, at the upper edge, the membranes can each have been fastened or clipped in order to take up the 10% surplus. Following the placement of the boulders on the river bed, then further boulders can be placed up the slopes of the bank and would then hold the surplus ripple in place.

12.8 Replacement of the River Soils

Subsequent to the placement of the boulders on the river bed, the design incorporates the transport and replacement of the original river soils back on to the boulders. These materials would be placed in a wet condition so that they fill interstices between the boulders, and would restore the river level to its nominal original height. River soils would be stored in simple supported fence systems, and would have been kept moist during that period.

12.9 Re-Establishment of River Flow

Subsequent to the completion of lining installation and refurbishment of river soils, the design envisages the slow release of river-water into the system from upstream.

13. CONCLUSIONS

It is concluded that modern geotextile and webbing membranes can be manufactured with sufficient strength to allow the design of impermeable lining systems for rivers or canals over areas of future mine workings.

In order to meet all of the varying demands of envisaged ground conditions, it is recommended that a multiple-layer system is used forming a composite structure. Each of the individual layers is chosen to contribute towards the overall performance of the composite lining, giving the engineer considerable scope for design specification.

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Sealing and Waterproofing with Geotextiles and Plastic Sheeting in Tunnel Constructions

This report should show the technical demands on a durable and secure sealing system in tunnelling, the different kinds of stress and strain during installation and after completion of construction work as well as its range of application in the fields of underground construction in rock and soil. The sealing system must guarantee a lasting imperviousness and thus the functioning of the structure. The high standards set out in this report for an effective sealing system are based on emperical data of numerous road-tunnels, underground railway sites as well as pressure tunnels and shafts. The protective- and drainage function of a non-woven fabric intermediate layer should be a topic open to particular discussions. As a special case of application a report is presented on the "non-woven fabric and plastic sheet - sealing system" for the pressure tunnels and shafts of the Sell-rain-Silz hydroelectric power plant with extremely high head (1).

Introduction

Tunnelling activity has shown an unexpectedly high rise the last few years, particularly in Germany, due to the development of underground traffic-networks in large towns, the development of high speed tracks for railways and the construction of road tunnels in the course of road improvement and development of freeways and highways. Also the current annual construction work of supply-lines, such as potable water tunnels, sewer tunnels and pressure tunnels, is expected to treble (2). This unsteady increase of underground construction activity is deemed to be responsible for the development of a technically matured sealing system. That is a system which can resist mechanical stress and deformation and so ensure permanent tunnel waterproofing, thus avoiding future lengthy and expensive re-sealing work. Because of the many advantages, plastic sheet-sealing with a non-woven protective layer has been prevailing worldwide in tunnelling, over conventionel sealing-methods, such as asphalt membranes, sprayed insulations with glass fibre-reinforced plastic or bitumen-latex based insulations (3). A sealing system has been developed particularly, together with the New Austrian Tunnelling Method (NAT). This system meets not only the demands of a high tunnelling rate but also the demands of rough construction treatment caused by the need for speed of installations. A sealing system's effectiveness depends on a great number of construction details which, due to the significance of possible damage, must be adhered to in a meticulous way. The requirement for absolute imperviousness makes high technical demands on a tunnel-sealing system in which the protection from damage of the single loose-laid plastic sheet is of paramount importance. These technical requirements in a permanent waterproofing system must be applied to each individual part of the sealing system which comprises the rock or shotcrete surface, the non-woven fabric protective layer, the fastener plates and the waterproofing membrane. These demands meet the actual standard of knowledge and include the following points:

1. Demands on the waterproof membrane

- The membrane must provide a continuous enclosure in the critical areas of the construction
- The imperviousness of the membrane must remain permanently
- The membrane must be able to adapt to the discontinuity of the surface
- The membrane must not loose its waterproofing effect in spite of structural caused movements by shrinking, creeping, temperature variation, settlements and vibration; it also must be unaffected by convergent movements of the rock bond
- The membrane must be able to absorb discontinuous stress variations without being damaged
- The membrane must be able to absorb, to a certain extent, in plane forces
- The membrane must be able to bridge cracks in the construction surface
- The membrane must be resistant to all types of aggressive water, whether from a natural origin or leached from concrete, mortar or other materials
- The membrane must be resistant to biological attacks
- The membrane must also be suitable for installation on damp or wet areas
- The membrane must be easy to handle and instal , in large quantities, quickly
- It must be possible to install the membrane with the help of a reliable, mechanized welding method
- It must be possible to check all welded seams to see if they are watertight

In modern underground construction of seepage and pressure waterproofing, some thermoplastics have been used predominantly as base materials. These are PVC-soft, PE and ECB.

2. Demands on the rock- and shotcrete surface

The loose laying of a plastic sealing-sheet presupposes a backing of sufficient non-deformability and natural strength.
A shotcrete layer with a thickness of at least 4 cm is essential for the backing, to serve as backing for the fasteners of the fastening plates. The maximum size of the aggregate should not exceed 16 mm and crushed rock generally must not be used. The surface condition of the backing is of special importance at the time of concreting (5). Consideration to this therefore must be given from the time of installation of the plastic sealing-sheets. They should lie against the shotcrete as solidly and as flush as possible. This should be achieved without excessive stressing and damage. So that this is possible the shotcrete must be used to smooth the excavated surface and to cover any metal protuberances.

3. Demands on the fastening-system

The non-woven fabric layer and the plastic sheets are installed with the help of a special scaffold. As a first step the non-woven fabric layer is fastened to the shotcrete (gunite) by means of plates and fasteners. As a second step the actual sheeting which will seal the tunnel is then secured to these plates by means of high-frequency ring welding or hot-air welding. On average 4 fastening points per square meter are required to fasten the protective layer to the tunnel wall. The fastening plate is designed in such a way that if the plastic sheet is overstressed and deformed at a weld, the failure will always occur in a certain plane of weakness inside the fastening plate, never in the plastic sheet itself (3). Thus it is possible to avoid excessive stress and distress of the plastic sheet if such deformations happen when concreting the internal tunnel ring, however, all further protective functions must be carried out by the non-woven fabric layer(s.figure 1-4).

4. Demands on the non-woven fabric layer

The non-woven fabric layer performs in an underground sealing and waterproofing construction, in rock and soil, not only a protective function but also a drainage function. Therefore it is of decisive importance for the effectiveness of the total sealing system (6). The existing technical demands on the non-wovens in tunnelling result from the different kinds of stress and strain during the construction stage as well as in final condition after completion of construction work. The types of stress include those caused mechanically, chemically and hydraulicly. Functional correlations occur between them. It is therefore only possible to claim a permanent mechanical protection function of the non-woven when it is effectively permanently inert to acid and alkali attack. The following technical requirements for non-woven fabrics used in underground construction, in rock and soil, correspond to state of the art geotextile research:

- Chemical resistance to a pH-range 2 - 13 (s.pt.4.1)
- Mechanical resistance by minimum demands (s.pt.4.2)
- Water permeability within the plane (s.pt.4.3)
- Rot resistance (s.pt.4.4)

4.1. Chemical resistance

So that the non-woven fabric can perform a lasting protective function, it is axiomatic that it has a chemical resistance to all types of ground-water. Especially to Calcium Hydroxide $Ca(OH)_2$ (hydrated lime) and other aggressive compounds found in hydraulic binders (e.g. concrete). Examination of the chemical resistance using Na OH (Sodium hydroxide) or K OH (Potassium hydroxide) yields little information. Importantly, the requirement on a non-woven can only be met, provided that there is no doubt about the chemical compound of the base material and so the inert nature of the final product. Polypropylene and Polyethylene are resistant to the required pH-range 2 to 13, whereas Polyester has only a limited resistance. Investigations have shown that non-wovens made from polyester-filaments suffer, during their first 6 months in contact with concrete, a loss of strength of approximately 60 %. Bi-components if they contain, in part, Polyester filaments and non-wovens made from unknown regenerated materials should also be regarded as problematic. The importance of chemical stability cannot be overstated as, long after construction, stress redistribution in the rock bond may lead to convergent movements of the tunnel. It is therefore essential that the non-woven is still intact to protect the waterproof membrane.

4.2. Mechanical resistance

Protective non-wovens must have a certain strength and a certain extensibility to resist the stress and strain caused by the placing operation, concreting pressure and by deformations in the tunnel lining which can occur due to load redistribution and temperature influence. It is also important that the non-woven fabric can resist the potentially damaging effect of high, periodic, hydrostatic pressure. This can occur on a localised basis at fissures. The following minimum mechanical properties for a non-woven have been determined by means of state of the art evaluation circumstances. They recognize that the non-woven must cope with high extension stress and high tear stress when in contact with the partly sharp edged shotcrete surface.

Table 1: Minimum demands on the properties of protective non-wovens in tunnelling

	Standard	Dimension	
thickness under 0,02 bar	DIN 53855/3	mm	3,0
strip tensile strength	DIN 53857/2 ASTM D 1682	N/50 mm	950
minimum extension at 30 % of strip tensile strength	–	%	20
extension at break (Longitudinal,transvers)	DIN 53857 ASTM D 1682	%	45
CBR-puncture resistance	DIN 54307 E	N/50 mm	2800
cone penetration test (puncture resistance)	The Technical Research Centre of Finland	mm	7,5
bursting strength	AS-2002.2.4	N/cm2	350

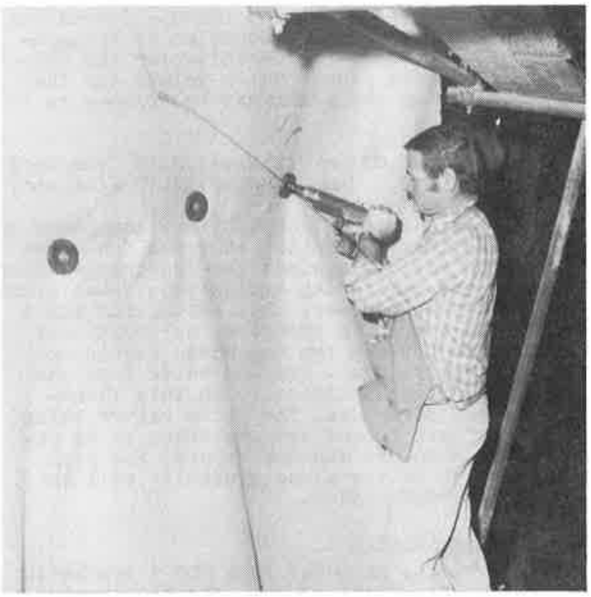

Fig. 1: Fastening the non-woven protective layer to the tunnel wall

Fig. 3: Section of a tunnel showing installed non-woven and the special scaffold to facilitate fastening of the non-woven and plastic sheeting

Photo not available

Fig. 2: Spotwelding between the waterproof membrane and the non-woven fabric

Fig. 4: Section of a tunnel after fastening of the plastic sheeting

Criteria for the selection of non-woven fabric protective layers in tunnelling

- The weight of a non-woven must be rejected as a selection criterion for application in tunnelling. The weight not only depends on the weight of the raw-material itself but also on the manufacturing process, it is not possible therefore to conclude mechanical properties from weight information alone.
- The thickness of a non-woven cannot be taken, without hesitation, as a parameter of the mechanical effectiveness either. A greater thickness need not mean a greater factor of safety against distress. Staplefibre non-wovens have curly fibres and hence they are thicker in an uncompressed state than are continuous filament non-woven fabrics of the same area weight, as there latter have sleek fibres.
  If the non-woven fabric will be subjected to compressive stress, during placement of concrete and during concrete vibration, then the staplefibre non-woven will become thinner than the continuous filament ones. The reason for this is that the continuous filament fabric has a greater original compactness of fibres. Therefore the thickness of the non-woven can only be considered as a selection criterion when taken in conjunction with the type of non-woven, it's quality, the process by which it has been manufactured and it's other mechanical properties.
- The strip tensile strength test can be assumed an index test. This has a great potential power of statement for practical use. It has no direct relation to the field of application in tunnelling. However, you can estimate a very important characteristic index for the protective function - adaptability. This is derived from the strip test's stress-strain curve and from absolute values of the strip tensile strength. The adaptability of non-woven fabric is characterized by the initial strain (minimum extension at 30 % of the strip tensile strength) and by the extension at break. Its importance can be seen especially when concreting the internal ring of the tunnel. This concreting causes the waterproof membrane to be pressed into the hollows on the shotcrete surface.
  The CBR value and the bursting strength demonstrate the reinforcing behaviour of the non-woven fabric which must span the cracks and irregularities of the tunnel wall. The extension of the waterproof membrane into cracks and fissures, during concreting the tunnel lining should be controlled by the non-woven fabric's reinforcement function.
- The cone penetration test results in an average value of diameter - a characteristic index which tries to show the non-woven fabric's puncture resistance relating to the peaked, sharpedged surface condition of the shotcrete lining. This kind of stress not only takes place during concreting the internal ring, but also after completion of construction work. Then the rock-bond is pressed against the internal ring either because of redistribution of stresses or when the internal ring expands because of temperature influence.

4.3. Water permeability within the plane

Local ingress of water is collected in the course of shotcrete lining by means of PVC-drain pipes or central, slotted pipes and conducted to the drainage at the bottom. The residual water which comprises seepagewater and other smaller ingresses of water can dissipate in the plane drainage of the non-woven fabric. It is then conducted in longitudinal collection drains to the tunnel entrance. The three - dimensional structure and the voids-reduction of the non-woven under pressure is decisive for the permeability in the plane. Guide values for the permeability in the plane may be assumed as follows:

k-value under 0,02 bar (2kN/m2): 8.10^{-1} (cm/sec)
2 bar (200kN/m2): 8.10^{-2} (cm/sec)

These coefficients of permeability are based on the theoretical assumption that the non-woven is pressed solidly against the shotcrete-surface and that the dewatering exclusively takes place in it's plane. However, this assumption won't very often happen in practice, as additional voids exist between the non-woven fabric and the shotcrete lining. Seepage water loss can therefore drain additionally in this three-dimensional structure. The guide values being stated in this report are therefore to be regarded as absolute minimum values; the real permeability in the plane generally will be abaout 10 times higher.

4.4. Rot resistance

All non-wovens, produced from 100 % Synthetic fibres are resistant to rot. The requirement of rot resistance is for the same reasons as set out in section 4.1. "Chemical Resistance" and relates to all non-wovens from regenerated material, non-wovens not produced from 100 % synthetic fibres and non-wovens produced from viscose fibres as these may biologically rot, being regenerated cellulosic fibre.

Actual Projects

During the last decade, at home and abroad, quite a number of tunnels, shafts and galleries have been sealed with the durable and secure system - "non-woven fabric/plastic sheet".
The effectiveness of this sealing system results in low maintenance, in a reduction of operating costs and in an immense increase in traffic safety in road tunnels.
Amongst the numerous completed underground traffic facilities and those presently under construction (e.g. the underground railroad, Seoul, Korea, 780 000 m2 sealed area. The underground railroads in Pusan, Korea and Milan, Italy, 500 000 m2 sealed area) there has been an interesting range of Austrian Tunnel-projects. At an international level the most famous of these is the 12 km long Arlberg road tunnel. In this project the protective layer to the plastic sheet is the mechanically bonded, non-woven polypropylene fabric Polyfelt TS.

The following report on a storage scheme with an extremely high head (1) should give an impression about the range of applications of a highly developed sealing and waterproofing technology. 18 000 m2 of a patented sealing system composed of PVC-sheet and the modified non-woven polypropylene fabric Polyfelt TS 601 were installed in the pressure tunnels and shafts along a total tunnel length of 1 400 m. The 780-MW Sellrain Silz Power Plant built by the TIWAG makes use of a total head of 1 678 m in two stages (Fig. 5). The PSS Kühtai in the upper stage powers the water from the 60 million m3 Finstertal seasonal reservoir through a 1 800 m long pressure shaft without surge chamber to the 400 m lower Längental equalizing reservoir (capacity 3 million m3). The two reversible units with pump turbines in the Kühtai Shaft Power Plant are designed for a maximum capacity of 285 MW using turbines and 250 MW using pumps. The lower stage is a typical high head power plant, characterized by its total head of 1 257 m. From the Längental Reservoir, the water runs through a 4 600-m-long headrace tunnel with throttled twin surge chamber and a 2 400-m-long steel-lines pressure shaft to the Silz Power House. It powers two generating sets with six-nozzle Pelton turbines, having a total capacity of 495 MW.

For both the execution of the prestressed concrete lining with and without plastic sheeting as well as for the design of the steel lining required to share the internal pressure with the rock in the heavily loaded shafts of the two power stages, comprehensive investigations, examinations and rock mechanical test were necessary.

A prototype trial section of the concretelined pressure tunnels sealed with plastic membrane was constructed in the form of a test chamber with 2.0 m internal diameter and 25 m of length in an exploratory adit of the lower stage (Fig. 6). The objectives of the test program were to determine the in-situ casting of the lining with PVC sheeting, the behaviour during prestress grouting with the adapted TIWAG-system and the short- and long-term operating behaviour of lining and rock bedding under internal pressure conditions up to 50 bars. The test chamber was located in an area representative of the weaker rock conditions of the waterway where a radial jacking test had previously been performed to determine the deformability of the rock mass under internal loading.

The prototype lining system consists, from inside to outside, of the following layers:

- concrete lining
- waterproof 3-mm thick PVC sheeting
- non-woven Polypropylene-fabric } *)
- thin plastic sheet } *)
- shotcrete lining

*) non-woven PP-fabric laminated with a film of plastic

The non-woven fabric and the waterproof sheeting were fixed to the rock or shotcrete surface in the same way as at tunnelling, however the non-woven PP-fabric has been laminated with a film of plastic for the special use in the pressure tunnel construction. The non-woven fabric was fixed to the shotcrete in such a way that the plastic-film side of the non-woven faced the shotcrete surface. To allow grout injection through the sheet, special injection collars were welded to the sheet. In order to control grout distribution, rings or spirals of non-woven fabric were attached to the shotcrete prior to placement of the sheeting. The laminated film of fabric prevents the grout from intruding into the non-woven when the circumferential joint between rock and concrete lining is pressure grouted. The grouting can spread easily upon the laminated plastic film and fill the circumferential joint uniformly, so that the waterproof sheeting is embedded entirely flush on both sides. Thus it enables the non-woven to accomplish perfectly well its protective function for the waterproof sheeting, and makes it possible to avoid distress or rupture of the membrane with utmost safety, if internal pressure is put on it.

As can be seen from the test program (Fig. 7), in addition to installing the plastic sheet system, its waterproofness was tested in an external pressure test. Then, the lining was

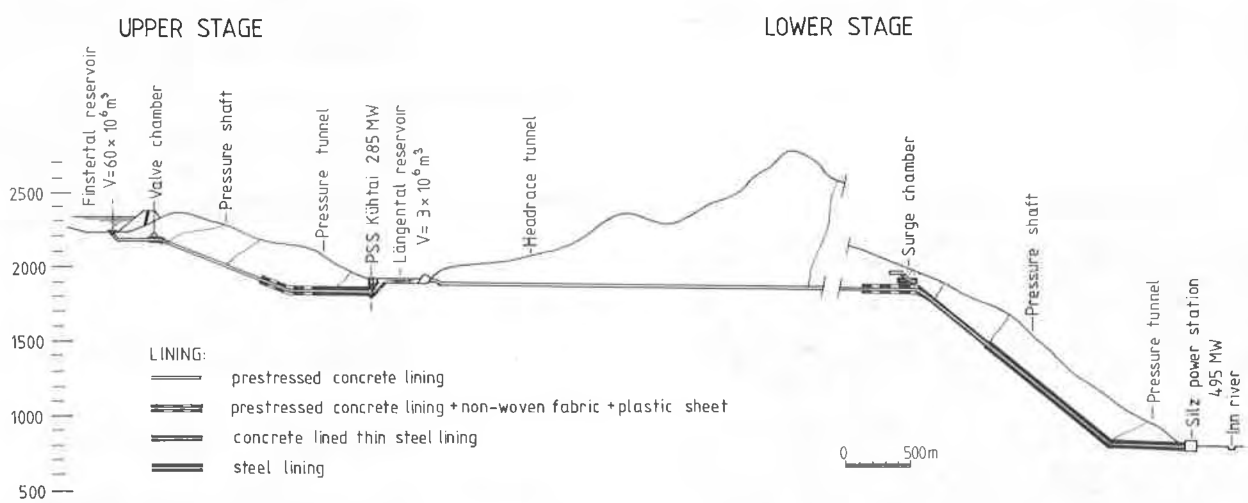

Fig. 5: Cross section showing the lining of the Kühtai upper stage and the Silz pressure shaft

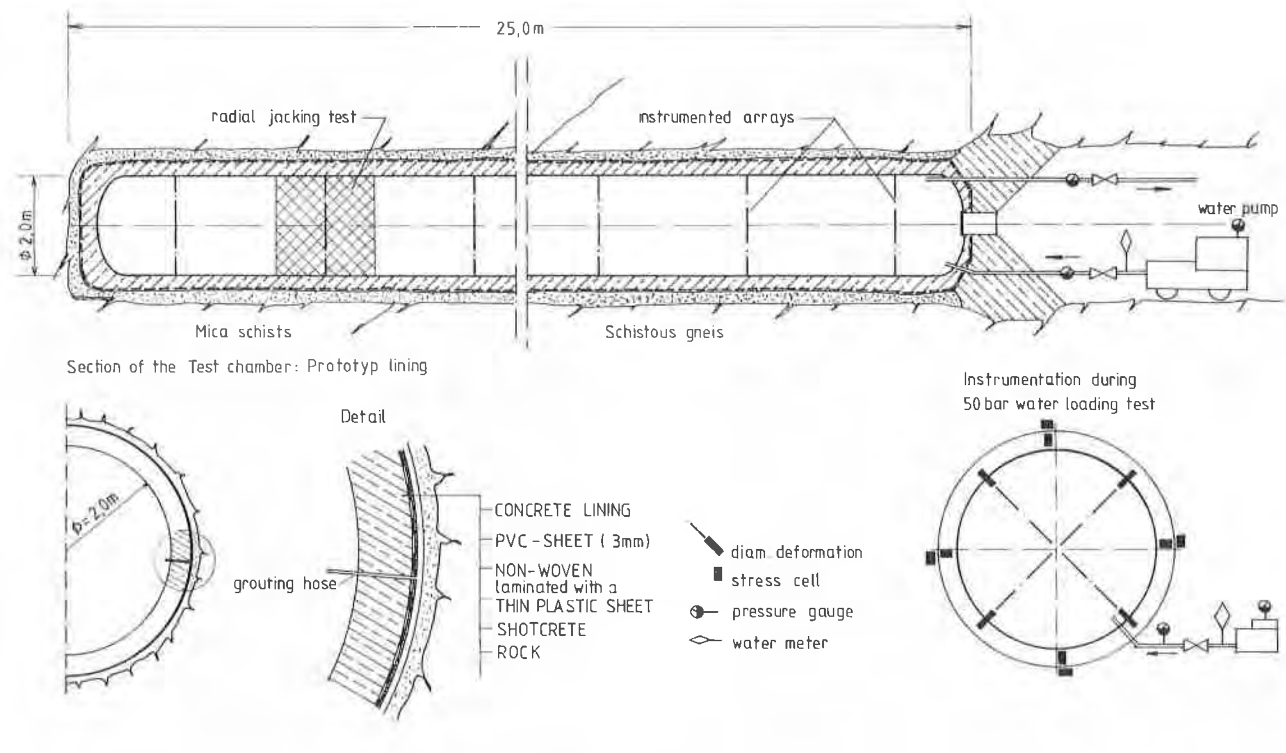

Fig. 6: Test chamber layout and instrumentation arrangement

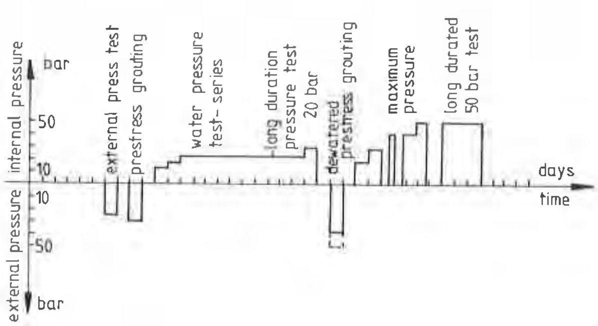

Fig. 7: Test chamber program

prestressed using high-pressure grouting and tested for permeability at 50 bars and longterm loading. The extensive instrumentation in the pressure chamber during prestress grouting and the permeability test supplied information on the lining's behaviour. The plastic sheet was made practically watertight by grouting, so that 50-bar internal pressure test showed a water loss of only 0,8 l/min = 5.10^{-3} l/min/m2. This lining system was installed at the Sellrain Silz Power Plant in the intake structure, in the lower elbow of the pressure shaft and in the tailrace shafts of the upper stage and in the lower stage's headrace tunnel and surge chamber.

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Methods of Constructing and Evaluating Geomembrane Seams

A major concern in the performance of polymeric geomembrane systems is the long-term integrity and strength of their seams. The joined interface where rolls or panels of flexible polymeric sheet materials come together is often the most vulnerable location in a geomembrane system to attacks of wind, water, chemicals, heat, and stress caused by subgrade anomalies, shrinkage, or subgrade consolidation. As such, the methods of joining the polymer sheet materials merit close study as well as the methods available in evaluating the quality of the joining system.

This paper will survey the most common types of seaming methods used to bond the various polymeric geomembranes. Bonding systems from simple tape and solvent bonds to thermal, vulcanized, and fusion welding will be discussed. In addition, the commonly used destructive and nondestructive quality control techniques such as air lance, vacuum tests, electronic methods, and mechanical destructive testing will be described.

INTRODUCTION

Geomembrane seaming methods are dependent upon the polymer composition of the geomembrane system. Some systems can be seamed or bonded together by several different methods while others can be seamed by only one. The most common seaming methods for polymeric geomembranes are the following:

1. Methods involving heat only (thermal methods) such as electronic (dielectric), hot wedge, or hot air.

2. Methods involving solvents or cements including solvent bonding, bodied solvent adhesive, solvent adhesive, and contact adhesive.

3. Methods involving tapes only including polyethylene tape or uncured gum tape.

4. Methods using only mechanical friction connections or mechanical sewing.

5. Methods involving the vulcanization process with uncured gum tape or adhesives.

6. Methods involving the introduction of a hot base product such as the extrusion or fusion welding process.

Thermal seaming methods are applicable only to geomembranes composed of base products sensitive to heat, i.e., thermoplastics, crystalline thermoplastics, and thermoplastic elastomers. Extrusion or fusion welding processes are used exclusively on the crystalline thermoplastic HDPE (High Density Polyethylene) sheet materials usually in field fabrication of a system. Bonding methods with solvents and adhesives are normally used on thermoplastics or thermoplastic elastomers whereas contact adhesives can be used on most polymeric geomembranes. Vulcanization is used on the elastomer type geomembranes. Mechanical methods could be used on many of the thinner sheets but is predominantly used on the thinner polyethylenes and coated fabrics. All seaming methods could be used in the factory and in the field fabrication process with the exception of the thermal dielectric method which is used only in factory fabrication. Tapes and mechanical methods are found predominantly in field installation practice.

Geomembranes serve the primary purpose of controlling the migration of fluids from one point to another, usually in a surface or subsurface impoundment or conveyance system. As such, the geomembrane system is dependent on the factory and field seam performance assuming the base polymer is compatible with the system design and contained fluids. This paper will describe the commonly used joining methods and will discuss available seam test methods presently employed.

DESCRIPTION OF COMMON SEAMING METHODS

The following are general descriptions of methods in use today. Refer to tables 1A and 1B for an overview of seam methods used in factory and field fabrication. Details such as types of solvents, application rates, temperatures, roller pressure, etc., have necessarily been deleted due to the numerous and quite variable methods employed by the many different geomembrane manufacturers, fabricators, and installers. Reference should be made to individual manufacturer's literature for specific seam construction details.

Liquid Applied Solvent or Adhesive Systems

Introducing a cold applied liquid solvent or bodied solvent between two overlapping sheets of a geomembrane followed by pressure produces an acceptable seam system in the thermoplastic and thermoplastic elastomer polymer sheet materials. For solvent bonded seams, a solvent, recommended by the sheet manufacturer, is used to soften the surfaces to be bonded which effects a homogeneous bond between the membranes after application of pressure. The solvent will eventually dissipate and does not remain as part of the bond. Usually, seaming must be accomplished only when sheet and adhesive temperatures are above 15.5 °C (60 °F). If ambient temperatures create conditions where temperatures are lower than 15.5 °C (60 °F), auxilliary means of surface area heating must be used, such as heat guns or portable radiant heaters. A bodied solvent adhesive seam is prepared by dissolving approximately 8 to 12 percent of the parent lining material polymer in a recommended solvent and applying the liquid to the surfaces to be bonded, thus effecting a homogeneous bond by applying pressure and after evaporation of the solvent.

A liquid applied solvent adhesive is used to develop bond strength between two sheets by means of an adherand left after dissipation of the solvent. The adhesive becomes an additional element in the seam system effecting a surface to surface bond but not a homogeneous bond. Sufficient rolling pressure must also be used to produce an acceptable adhesion. Single component contact adhesives can be used on practically all polymer sheet materials with the possible exception of HDPE. Simply, the contact adhesive recommended by the sheet manufacturer is applied to both mating surfaces to be joined usually by brush or roller. After the adhesive has reached proper degree of tackiness, the entire length of overlap for the two sheets is joined together immediately and in one motion. This is followed by applying roller pressure. The adhesive forms the bond but is also an additional element in the seam. Figure 1 shows the application of solvent in forming a PVC solvent welded seam.

Fig. 1. - Making a solvent seam on PVC.

All types of solvent or adhesive systems require the following steps in seam fabrication:

1. Position the sheets with recommended overlap
2. Clean the seam area with appropriate solvents if required
3. Apply solvent or adhesive
4. Effect a tight bond with pressure
5. Inspect and/or test

Vulcanizing Tapes and Adhesives

Because of the density or tightly packed molecular arrangement of vulcanized elastomers such as Butyl and EPDM (ethylene propylene diene monomer), these materials possess excellent impermeability by impeding the movement of moisture or moisture vapor. However, because of the cross linked structure, the polymer will not go into solution with solvent systems to form a bonded joint. In addition, thermal methods that are used on the thermoplastic materials do not provide acceptable acceptable bonding on vulcanized elastomeric sheet materials. To provide a strong bond, an uncured tape or adhesive containing the polymer base of the geomembrane and cross-linking agents (usually sulfur or sulfur-bearing compound) is placed between the overlaps of two cured (vulcanized) elastomer sheets. Upon application of heat and pressure at the seam area, molecular cross linking (vulcanization) of the polymer base in the tape occurs thus achieving a bond strength between the two sheets not attainable with ordinary adhesives. Factory vulcanization of seams is usually accomplished in a large vulcanizing press or an autoclave whereas field seams can be fabricated using the uncured tape and a portable seaming machine that provides the necessary heat and pressure with a bar.

Thermal Methods

Thermal seaming methods are used on thermoplastics, crystalline thermoplastics, and thermoplastic elastomers in which the base polymer is sensitive to heat and readily softens or melts. In practice, the surfaces of the two polymer sheet materials to be joined are first softened or melted and then immediately pressed together to form a "melt" bond that extends into the first few mils of thickness of the sheet (melt depth is dependent on heat source temperature, dwell time, sheet temperature). There are basically three methods of producing a thermal bond: hot air, hot wedge (hot knife), and electronic or dielectric bonding.

- Hot air methods. - Hot air can be used on most of the thermoplastic and thermoplastic elastomer materials as shown in table 1b. A machine consisting of resistance heaters, a blower, and temperature controls directs a blast of hot air or gas usually in excess of 260 °C (500 °F) between two polymer sheet materials, thus melting the surfaces. Immediately following the surface melt, pressure (usually by rollers) is applied to bond the area to form a homogeneous seal between the two membranes at the seam overlap. The heat-producing machine can be either a hand held unit for the field or a large, automated factory machine with preset controls for temperature, movement rate, and roller pressure.

- Hot wedge. - The hot wedge method (also called hot knife) consists of a hot electrically heated resistance element in the shape of a blade or V-shaped wedge that is passed between the two sheets to be sealed and in intimate contact with them thus melting the surfaces. Immediately following the melting, roller pressure is applied thus effecting a homogeneous bond. The hot wedge method is particularly suited for the thicker [greater than 0.76 mm (30 mils)] LDPE and HDPE materials, but could also be adapted for use on the reinforced thermoplastics. Single hot wedge and dual hot wedge systems are presently available. The dual hot wedge weld forms a continuous air channel between two welded seams and is used as a means of testing the bond continuity by injecting air pressure. Welding rate (movement of the machine) as well as temperature and roller pressure are adjustable and continuously monitored. Adjustments are made according to environmental conditions such as ambient temperature and moisture. The hot wedge method can be used in both the factory fabrication of panels and in field installation.

- Dielectric. - Electronic bonding commonly referred to as dielectric is used on thermoplastic and thermoplastic elastomer materials and is carried out only in a fabrication plant. A rapidly reversing electric field is used to effect rapid realinement of molecules at a high rate within the polymer structure. This rapid realinement by reversing field polarity causes internal friction within the compound thus generating heat very rapidly. Electronic bonding machines are large and consist of a top electrode bar [usually 25 mm (1 in) or less in width] a large bottom electrode plate, a high frequency oscillator tube, a pressure system to clamp the sheet being seamed, and a high voltage transformer. Because of their bulk, power requirements, and need for cleanliness, dielectric machines are used only in an enclosed factory production facility. The machines run on 220 V and produce up to 10 kW of power while operating in a 27-MHz radio frequency range. The two polymer sheets to be bonded are placed as a lap joint between the electrode bar and bottom plate and held in position by mechanical pressure from the top electrode. Electrical current is turned on for 1 to 3 seconds transmitting the high frequency through the overlap causing the sheet material to soften and melt together forming a homogeneous bond. The top electrode bar is normally 0.61 to 1.23 m (2 to 4 ft) in length. As the sheets are passed through the electrode, the ends of the bonded areas are overlapped thus forming a continuous seam.

Extrusion or Fusion Welding Methods

Extrusion welding is currently employed in the field joining of HDPE sheets ranging in thickness from 0.6 to 3.1 mm (20 mils to 120 mils). It consists of extruding a ribbon of molten polymer of the same resin as the sheet itself between two mating surfaces or at the edge of two overlapped sheets thus effecting a homogeneous bond under optimum conditions. If extrudite is placed between two overlapping sheets, the technique is referred to as flat welding. This is accomplished by means of a self-propelled extrusion welder which is designed to systematically separate the overlaps, preheat the weld area, extrude a bead of molten HDPE, and pressure roll the weld area. All temperatures and speed of welding are controlled automatically and are very carefully preset according to ambient conditions.

If a bead of molten HDPE is placed at the edge of a seam, the technique is called a fillet weld or hand extrusion weld. The extrusion machine, although smaller, is also carefully preset as to temperature of preheat air and extrudite.

For either type of weld, the following operations are carried out to effect seaming of the HDPE sheets:

1. Position the sheets with manufacturers' recommended overlap

2. Grind weld area to be welded with a hand electric grinder

3. Extrude molten HDPE between or at the edge of the overlapped sheets

4. Provide roller pressure for flat welding

5. Inspect and test the weld

Depending on the manufacturer of the sheeting, the fillet type weld area may or may not be preheated with hot air.

Tapes and Mechanical Seaming

In addition to the more complicated methods of seaming polymer sheets, there are also available relatively simple and quick methods of joining that may be fully acceptable for certain applications.

Tape seams are made by applying an adhesive backed strip between or at the edge of two sheets. The tape provides tensile strength to the joint; however, the tape backing and adhesive may be different than the polymer sheet thus introducing two additional elements to the seam system. The resulting seam will not be considered homogeneous and, thus, its limitations as to long-term integrity must be recognized.

Mechanical methods of seaming use no adhesives or thermal joining but rather rely on the friction between two sheets using some type of pressure or clamping system or mechanical sewing. The simplest friction seam is that employing a 0.3 to 1 meter (1 to 3-foot) overlap of a flexible geomembrane with soil cover material. The soil cover placed over the lining provides overburden pressure to hold the overlapped sheets in place. A more sophisticated mechanical seam method uses an inner rod and an outer "C" clamp of flexible polymer material to hold two sheets together. These methods are commonly used for applications such as water conveyance or storage where less than 100 percent seepage reduction is acceptable. A mechanically sewn seam provides tensile strength but must be waterproofed with a liquid field-applied polymer material.

METHODS USED IN EVALUATING SEAM INTEGRITY

NDT (Nondestructive Test Methods)

There are several available methods for qualitatively testing seams without taking samples from a completed lining system. These nondestructive test methods can be used to measure the continuity of a seam but cannot be used to quantitatively measure the relative strength of the joint nor the projected future performance. These methods should be used in conjunction with destructive testing in a quality assurance program. Table 2 summarizes the available NDT methods which are briefly described below.

Electronic Methods

There are two electronic methods that have been proven to work on detecting unbonded areas in geomembrane seams. These are the Ultrasonic Pulse Echo and the Ultrasonic Impedance Plane Technique.

The Ultrasonic Pulse Echo Technique is presently used in the field for testing continuity of HDPE flatwelds. However, this method works equally well on other nonreinforced geomembranes. Basically, the technique is a thickness measurement using a high frequency pulse of energy transmitted into a seam. The high frequency pulse (5-15 MHz) is transmitted via a surface transducer through both sheet thicknesses and returned to a receiving oscillator after reflecting off the lower sheet bottom surface. If the seam is homogeneous, the pulse echo will indicate a characteristic curve on a time-based CRT (cathode ray tube) monitor. If there is an unbond, the signal will reflect off the discontinuity and indicate the faulty contact on the monitor. To assure good bonding across the seam width, the entire seam surface area must be scanned with the ultrasonic pulse echo transducer. The operator visually marks all unbonded areas for repair.

The UIP (Ultrasonic Impedance Plane) technique works on the principal of characteristic acoustic impedance. A well-bonded joint possesses a certain acoustic impedance after calibration and any other area that is unbonded is different. A continuous wave resonant frequency of 160-185 kHz is transmitted through the overlap seam by means of a surface transducer, and a characteristic dot pattern is established on a CRT monitor. Location of the dot pattern on the screen indicates a good bond, an unbond, or an uncoupled transducer. This method, although not yet widely used, can be applied to reinforced as well as nonreinforced geomembranes as the scrim reinforcement does not seem to affect the low frequency transmission.

Both of the above methods require a surface transducer that is wide enough to be able to scan the full width of a given seam along the entire bonded length. Improvements in method of application for both systems are needed to increase the speed of testing as well as the accuracy. Present speed of testing is only 1 to 2 m/min (3 to 6 ft/min).

Air Lance

This method requires that a jet of air [342.5 kPa (50 psig) regulated and directed through a 4.8 mm- (3/16-inch-) diameter nozzle] be applied to the upper edge of an overlap seam in an effort to detect an unbonded area. When correctly used, the impinging air stream will inflate an unbonded area as shown in figure 2. This area is then marked by the inspector for subsequent repair. The air lance method requires a portable compressor in the field and can effectively test seams at the rate of approximately 4 m/min (13 ft/min).

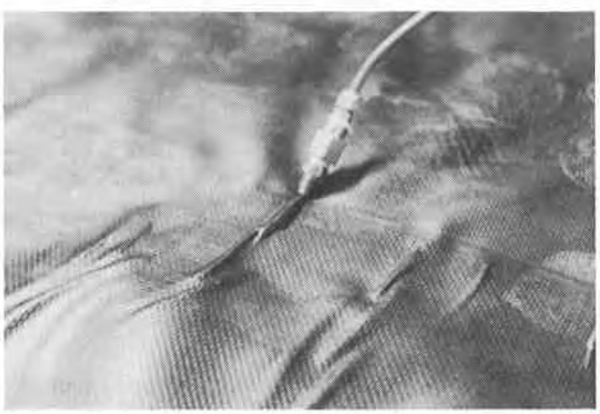

Fig. 2. - Unbonded area detected by air lance.

Vacuum Chamber

The equipment needed for a vacuum seam test consists of a vacuum box capable of being sealed to a seam area with a glass viewport and a portable electric vacuum pump. The rubber seal of the chamber bottom is first soaped to provide a vacuum seal and then the seam edge is covered with a soapy solution. When a vacuum is applied to the chamber [usually less than 17.2 kPa (2.5 psig)], an unbonded area is indicated by the presence of bubbles. The unbonded area is marked for subsequent repair. Speed of testing is usually 1.5 m to 3 m (5 to 6 ft) per minute with one operator.

Pressurized Dual Seam

This test method is utilized on a double wedge thermally welded seam where an air channel exists between two bonded lengths of seam. After the dual wedge seam has been fabricated for a given length [(say 31 m (100 feet)], both ends are sealed. A needle with attached pressure gage and air valve is inserted into the air space between the thermally welded areas, and pressure [usually 207 kPa (30 psig)] is applied to the air channel. The gage is monitored for drop in air pressure over time as an indicator of a seam unbond and leak. Although this method tests long lengths of seam at one time, if a leak is detected, that entire length must be rechecked with a vacuum box or the length must be cap stripped. The pressurized dual seam is presently in use on thermally welded HDPE but could be used on other polymer sheet materials capable of being seamed with a dual wedge welder.

Electrical Sparking

Although rarely used, electrical sparking methods can effectively detect voids, pinholes, or unbonds primarily in the crystalline thermoplastic welds. Basically, a high voltage (15-30 kV) electrical current is applied to a seam area and any leakage to ground can be detected by sparking. A ground wire can be designed into the geomembrane system and attached to the back of a seam for subsequent use of the spark test apparatus.

Mechanical Point Stress

To detect unbonded areas, an inspector can slide a dull tool (such as a blunt screwdriver) along the top edge of a bonded seam. An unbonded area will be easily lifted and then marked for repair. Care must be exercised when using this method so as not to puncture the geomembrane or use excessive force against the seam edge. This method (sometimes called the "pick test") will detect edge unbonds that the air lance method may not detect because of a lightly tacked down edge. Coupled with visual inspection, this method can be a rapid test for seam continuity that requires little operator skill.

Destructive Test Methods

In addition to NDT methods that test for seam continuity, destructive test methods are also needed to quantitatively test for seam strength. Generally, cutouts of inplace seams can be required as well as factory sampling and daily test startup seams in the field. In addition, destructive testing can be accomplished on test coupons of seams placed at the site or retained in a laboratory for aging performance. Once the quality assurance program has been established and the sample size and frequency of taking samples is known, the samples can be subjected to mechanical testing at the owner's field laboratory or a qualified commercial test laboratory. Test results are compared to design specifications to help ensure seam jointing quality.

Shear and Peel Testing

There are only two mechanical test methods that are widely used in testing the bond strength of a polymeric geomembrane seam, tensile shear and peel testing. Tensile shear testing is accomplished by applying a static or dynamic load across the seam width, thus subjecting the bond interface to a shearing force. A static "dead load" at room temperature or at elevated temperatures can be used; however, dynamic loading using a tensile testing machine is most commonly used. The dynamic test method uses 25-mm- (1-in-) wide specimens for nonreinforced materials and 51-mm (2-in-) wide specimens for reinforced materials. Results are recorded as Newtons per meter (pounds force per inch) of seam width at failure. The mode of failure (i.e., failure at seam edge; failure at grip edge; failure in seam interface) must be recorded as well as test parameters such as grip separation, crosshead speed, seam width, temperature, and humidity. A lower strain rate [i.e., slow crosshead movement such as 51 mm/min (2 in/min)] gives a better indication of failure mode and more closely approximates slow tensile movement of an installed geomembrane.

Peel testing is accomplished by applying a load (static or dynamic) such that the bonded interface is subjected to a peeling force that attempts to separate the bonding system from the geomembrane sheet as shown in figure 3. Testing in peel under dynamic loading can be accomplished using 25-mm-(1-inch-) wide strip specimens (for both reinforced and nonreinforced materials) subjected to 180° peel at a test speed of 51 mm/min (2 in/min) in accordance with ASTM: D413. Test results should include peel strength in pounds per inch of width as well as mode of failure (i.e., failure in parent material; failure at interface of bond and geomembrane) and all applicable test parameters.

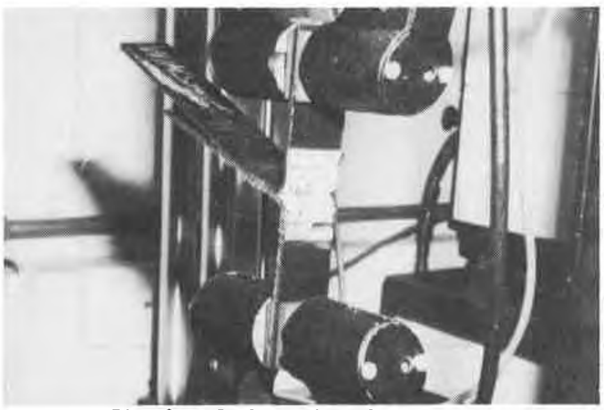
Fig. 3. - Peel testing of a seam.

Peel testing using a static "dead load" at room temperature 23 °C (73.4 °F) and elevated temperatures [usually 50 °C or 70 °C (122 °F or 158 °F)] provides a good indicator of time-dependent weaknesses that will not be observed under dynamic testing. Dead load testing at elevated temperatures can be used as a method of revealing the sensitivity of a seam system to long-term exposures on the geomembranes and the seam jointing system.

In this paper, the author has attempted to describe the most common factory and field methods used in joining today's polymeric geomembrane systems as well as the methods available for testing the seams. As the geomembrane industry grows, new polymer sheet materials will undoubtedly be introduced along with revolutionary new jointing methods. It is apparent that most manufacturer's are developing seaming procedures that will result in homogeneous bonding of their sheet materials both at the factory and in the field. This response to an educated marketplace is an effort to ensure owners that the sheet material installed is not only compatible with the contained material but that the entire system including the seams is designed and guaranteed for the expected life of the project.

Table 1A. - Available geomembrane seaming methods (Adhesive and Tapes)

Base polymer of common geomembrane systems	Solvent		Bodied solvent		Solvent adhesive		Contact adhesive		Vulcanizing tape/adhesive		Tapes
	M	F	M	F	M	F	M	F	M	F	F
Thermoplastics											
Polyvinyl chloride (PVC)	X	X			X	X	X	X			X
Nitrile - PVC (TN-PVC)	X	X			X	X	X	X			X
Ethylene interpolymer alloy (EIA)											
Crystalline thermoplastics											
Low-density polyethylene (LDPE)							X	X			X
High-density ployethylene (HDPE)							X	X			X
Elastomers											
Butyl rubber (IIR)							X	X	X	X	
Ethylene propylene diene monomer (EPDM)							X	X			X
Neoprene (polychloroprene)							X	X			
Epichlorohydrin rubber (CO)							X	X			
Thermoplastic elastomers											
Chlorinated polyethylene (CPE)	X	X	X	X	X	X	X	X			X
Hypalon (chlorosulfonated polyehtylene) (CSPE)	X	X	X	X	X	X	X	X			X
Thermoplastic EPDM (T-EPDM)							X	X			

Note: M = Manufactured or factory seams.
F = Field fabrication.

Table 1B. - Available geomembrane seaming methods (Thermal and Mechanical)

Base polymer of common geomembrane systems	Thermal methods					Extrusion (fusion) welding	Mechanical
	Hot air		Hot wedge		Dielectric		
	M	F	M	F	M	F	F
Thermoplastics							
Polyvinyl chloride (PVC)	X	X			X		X
Nitrile - PVC (TN-PVC)	X	X			X		X
Ethylene interpolymer alloy (EIA)	X	X					
Crystalline thermoplastics							
Low-density polyethylene (LDPE)	X	X	X	X			X
High-density ployethylene (HDPE)		X	X	X		X	X
Elastomers							
Butyl rubber (IIR)							
Ethylene propylene diene monomer (EPDM)							
Neoprene (polychloroprene)							
Epichlorohydrin rubber (CO)							
Thermoplastic elastomers							
Chlorinated polyethylene (CPE)	X	X			X		
Hypalon (chlorosulfonated polyehtylene) (CSPE)	X	X			X		
Thermoplastic EPDM (T-EPDM)	X	X					

Note: M = Manufactured or factory seams.
F = Field fabrication.

Table 2. - Available NDT methods for evaluating seams

Geomembrane system	Electronic		Air lance c/	Vacuum chamber b/	Pressurized dual seam	Electrical sparking	Mechanical point stress
	Ultrasonic pulse echo (5-15 MHz)	Ultrasonic impedance (160-185 kHz)					
Thermoplastics (PVC; TN-PVC; EIA)							
Reinforced a/		X	X	X			X
Nonreinforced	X	X	X	X			X
Crystalline thermoplastics (LDPE; HDPE)							
Nonreinforced	X	X		X	X	X	X
Elastomers (Butyl; EPDM; CR; CO)							
Reinforced			X	X			X
Nonreinforced			X	X			X
Thermoplastic elastomers (CPE; Hypalon; T-EPDM)							
Reinforced		X	X	X			X
Nonreinforced	X	X	X	X			X

a/ Electronic methods do not work on the EIA material.
b/ Vacuum chamber should be restricted to thicknesses of 30 mils and greater due to deformation.
c/ Air lance should be restricted to thicknesses less than 45 mil; this method is not recommended for stiff sheeting.

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The Evaluation of Seam Quality in Geomembrane Systems

This paper will describe several test procedures for evaluating seam joints in geomembrane systems. It will present actual test results utilizing this test criteria on several commonly used geomembrane materials and seam techniques. Finally, a testing protocol will be recommended for the longterm performance evaluation of seam quality in geomembrane systems.

A critical aspect of any geomembrane liner is the seam system. While the membrane selected for an application can be of very high quality, the ability of this membrane to perform in actual application is directly dependent on the quality of the seam joints that are used to construct the total liner system. Factory seam joints are constructed in a controlled environment to fabricate panel sizes that can be adequately handled in a field installation. These large factory fabricated panels are then joined together with field seam joints that are constructed in an uncontrolled outside field environment. The quality of both the factory and the field seams are equally critical to the successful performance of a geomembrane liner system.

There are several seam joint techniques employed today to provide effective factory and field seams. Following is a description of the most common types of seams being used in geomembrane systems.

Solvent Seams: Chemical solvents are used to soften and bond the membrane surfaces, providing a homogeneous bond between the membrane surfaces. Ideally, the chemical solvent ultimately dissipates and does not remain a part of the seam joint.

Dielectric Seams: High frequency dielectric equipment is used to generate heat and pressure on an overlap seam joint resulting in a homogeneous melt of the two membrane surfaces.

Thermal Seams: High temperature air or a hot wedge is directed between an overlap seam joint to melt the membrane surfaces, followed by a pressure system that results in the homogeneous bond of the two membrane surfaces.

Adhesive Seams: A chemical adhesive is used to develop bond strength between two membrane surfaces. The chemical adhesive becomes an additional element to the seam system and must be capable of withstanding all performance criteria to which the flexible membrane liner will be exposed.

Seam joints that are utilized in a geomembrane liner system will be exposed to a wide variety of environmental factors. The seam joint will come into contact with a soil environment, a moisture environment, and a chemical environment. It will have to withstand longterm aging when exposed to these environments. It may well be under constant stress, both at normal temperatures and at elevated temperatures.

The selection of quality control tests and procedures for the evaluation of seam quality is critical. These quality control tests must assure adequate initial seam strength to meet proper performance criteria. More importantly, these tests must simulate, as nearly as possible, the exposure conditions to which the seam will be exposed and the tests must try to verify the longterm performance of any specific seam joint system.

Numerous test methods are available for the evaluation of seam joint systems. Many of these test methods have been used successfully in the evaluation of seam joints utilized in the architectural fabric structure industry. These quality control test methods have proven successful for evaluating the longterm performance of seam joint systems which are under constant stress and are exposed to a wide variety of climatic and other environmental conditions. The successful experience with these quality control criteria in the architectural fabric structures industry makes them appropriate to evaluate the seam joint systems that are utilized in geomembrane liner systems.

Geomembrane systems that utilize a fabric substrate between the film or coating layers can utilize the following test methods as a measure of the adhesion quality of the seams and the adhesion quality of the coating or laminating system.

Ply Adhesion: ASTM D-413, Machine Method Type A - This test method is utilized to determine the quality of the coating bond or the film ply to the substrate layer.

Peel Adhesion: ASTM D-751, or Federal Standard 191, Method 5970 - Both of these test methods are similar in nature and are utilized to determine the strength of the seam joint.

Bonded Seam Strength: ASTM D-751 - This test method puts the seam joint in a sheer force to determine the strength of the seam in comparison to the tensile strength of the geomembrane material itself.

Dead Load Test: There is no ASTM or Federal Standard Test Method which adequately presents a dead load test. There are some military test methods such as Mil-T-52766A, Paragraph 4.6.2.16. This test method describes the procedure for dead load testing. The temperature, weight applied, and time period can vary according to the specific application.

The dead load test procedure is very important for evaluating the load carrying capability of a seam joint. Seam constructions may have excellent initial test values. However, when a sustained load is applied to the seam joint system, either at room temperature or particularly at high temperature, the seam joint can slip and fail. Because of the dark coloration of most geomembrane materials, the seam joint systems can reach temperatures in the range of 150°-160°F./66°C-71°C. Therefore, it is recommended that the dead load test procedure be done at both room temperature, 73°F./23°C. and at high temperature, 160°F./71°C.

To illustrate the sensitivity of the above mentioned test methods to detect factors that will effect the longterm performance of seam joints, a series of tests were run on three commonly used geomembrane liner materials. The materials selected were: Supported

Ethylene Interpolymer Alloy (XR-5®); Supported Chlorinated Polyethylene (CPE); Supported Chlorosulfonated Polyethylene (Hypalon®).

In each case, two types of seam construction were made, a two inch/5.08cm. overlap dielectric seam and a two inch/5.08 cm. overlap adhesive seam. The initial peel adhesion was determined utilizing Federal Standard 191, Method 5970. The initial bonded seam strength was determined utilizing ASTM D-751. A dead load test was performed at room temperature, at 160°F./71°C. and 130°F./54°C. The load applied in the dead load test was determined by the tensile property of the material. In the case of the Ethylene Interpolymer alloy, a 68.0 kg. (150 lb.) load was applied for four hours for the room temperature test and 34.0 kg. (75 lbs.) for the high temperature test. For the Chlorinated Polyethylene and the Chlorosulfonated Polyethylene, 47.6 kg. (105 lbs.) was used for the room temperature test and 22.7 kg. (50 lbs.) for the high temperature test.

After obtaining the initial test results, the seam joint samples were submitted to two aging tests. The first test was to age the seam joint in an oven at 160°F./71°C. for a period of 30 days. The second test was to age the seam joint in a water immersion at 160°F./71°C. for 10 days. These two types of aging tests, have proven to be effective criteria for measuring quality seam joint systems. After each seam joint was exposed to the above aging environment, the seam joint was again tested for peel adhesion and bonded seam strength. The results were then compared with original control samples to determine whether the aging environment had any significant effect on the quality of the seam joint system. This test process provided some indication of which test method was more sensitive to detecting a deterioration of the original seam strength. A summary of the results of these tests are shown in Charts 1, 2, & 3. The bar charts are an average of the two to four samples that were taken for each exposure condition.

CHART 1. SEAM TEST RESULTS
REINFORCED ETHYLENE INTERPOLYMER ALLOY (XR-5®)

	DIELECTRIC SEAM		CEMENTED SEAM	
Peel Adhesion Original	3.57 kg/cm (20+ lbs/in)		2.14 kg/cm (12 lbs/in)	
Bonded Seam Original	167.8 kg (370 lbs)		163.3 kg (360 lbs)	
Dead Load Test 73°F/23°C	68.0 kg (150 lbs)—passed 4 hrs		68.0 kg (150 lbs)—passed 4 hrs	
160°F/71°C	34.0 kg (75 lbs)—passed 4 hrs		34.0 kg (75 lbs)—failed	
130°F/54°C			34.0 kg (75 lbs)—failed	
	Initial	Exposed	Initial	Exposed
Aging Tests 30 Day Oven 160°F/71°C	165.1 kg (364 lbs); 3.57 kg/cm (20+ lbs/in)	161.5 kg (356 lbs); 3.57 kg/cm (20+ lbs/in)	163.3 kg (360 lbs); 2.14 kg/cm (12 lbs/in	159.7 kg (352 lbs); 3.57 kg/cm (20+ lbs/in)
Water Immersion 10 days 160°F/71°C	171.5 kg (378 lbs); 3.57 kg/cm (20+ lbs/in	150.6 kg (332 lbs); 3.57 kg/cm (20+ lbs/in	163.3 kg (360 lbs); 2.32 kg/cm (13 lbs/in	42.6 kg (94 lbs); .71 kg/cm (4 lbs/in)

KEY ■ = Bonded Seam Strength □ = Peel Adhesion

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® Hypalon is a Registered Trademark of The DuPont Company

CHART 2. SEAM TEST RESULTS
REINFORCED CHLORINATED POLYETHYLENE (CPE)

	DIELECTRIC SEAM		CEMENTED SEAM	
Peel Adhesion Original	3.57 kg/cm (20+ lbs/in)		3.21 kg/cm (18 lbs/in)	
Bonded Seam Original	136.1 kg (300 lbs)		136.1 kg (300 lbs)	
Dead Load Test 73°F/23°C	47.6 kg (105 lbs)—passed 4 hrs		47.6 kg (105 lbs)—passed 4 hrs	
160°F/71°C	22.7 kg (50 lbs)—failed		22.7 kg (50 lbs)—failed	
130°F/54°C	22.7 kg (50 lbs)—failed		22.7 kg (50 lbs)—failed	
	Initial	Exposed	Initial	Exposed
Aging Tests 30 Day Oven 160°F/71°C	138.3 kg (305 lbs); 3.57 kg/cm (20+ lbs/in)	136.1 kg (300 lbs); 2.50 kg/cm (14 lbs/in)	141.5 kg (312 lbs); 3.04 kg/cm (17 lbs/in)	136.1 kg (300 lbs); 1.43 kg/cm (8 lbs/in)
Water Immersion 10 days 160°F/71°C	138.3 kg (305 lbs); 3.57 kg/cm (20+ lbs/in)	131.5 kg (290 lbs); 2.14 kg/cm (12 lbs/in)	141.5 kg (312 lbs); 3.57 kg/cm (20 lbs/in)	40.8 kg (90 lbs); .36 kg/cm (2 lbs/in)

CHART 3. SEAM TEST RESULTS
REINFORCED CHLOROSULFONATED POLYETHYLENE (HYPALON®)

	DIELECTRIC SEAM		CEMENTED SEAM	
Peel Adhesion Original	3.57 kg/cm (20+ lbs/in)		3.57 kg/cm (20+ lbs/in)	
Bonded Seam Original	136.1 kg (300 lbs)		136.1 kg (300 lbs)	
Dead Load Test 73°F/23°C	47.6 kg (105 lbs)—passed 4 hrs		47.6 kg (105 lbs)—passed 4 hrs	
160°F/71°C	22.7 kg (50 lbs)—failed		22.7 kg (50 lbs)—failed	
130°F/54°C	22.7 kg (50 lbs)—passed 4 hrs		22.7 kg (50 lbs)—passed 4 hrs	
	Initial	Exposed	Initial	Exposed
Aging Tests 30 Day Oven 160°F/71°C	135.2 kg (298 lbs); 3.57 kg/cm (20+ lbs/in)	138.8 kg (306 lbs); 1.79 kg/cm (10 lbs/in)	134.7 kg (297 lbs); 3.57 kg/cm (20+ lbs/in)	136.1 kg (300 lbs); 1.25 kg/cm (7 lbs/in)
Water Immersion 10 days 160°F/71°C	135.2 kg (298 lbs); 3.57 kg/cm (20+ lbs/in)	122.5 kg (270 lbs); 1.43 kg/cm (8 lbs/in)	143.7 kg (297 lbs); 3.57 kg/cm (20+ lbs/in)	119.3 kg (263 lbs); 1.07 kg/cm (6 lbs/in

A close evaluation of the test results presented in Charts 1, 2, & 3, indicate that seam joint systems can be affected when exposed to different environmental conditions. In each case, the initial test results on the seam indicated a more than adequate seam joint. In some cases the seam joint deteriorated significantly when exposed to simulated environmental conditions. Certain test methods were more sensitive to detecting this deterioration than others. Specifically, the peel adhesion test identified a change in the quality of the seam construction much sooner than the bonded seam strength test method.

To illustrate this sensitivity among different test methods, a simulated test was run between peel adhesion and bonded seam strength. The peel adhesion test method was Federal Standard 191, Method 5970 and the bonded seam strength was ASTM D-751. The material selected for this test was the supported Ethylene Interpolymer Alloy having a tensile property of approximately 62.5 kg./cm. (350 lbs./in.) In this test procedure, the peel adhesion quality of a 2"/5.08 cm. overlap dielectric seam was intentionally varied from a level of 2.50 kg./cm. (14 lbs./in.) down to a level of .36 kg./cm. (2 lbs./in.) At each level of peel adhesion, a bonded seam strength test was made to determine the relationship between a decrease in peel adhesion and its effect on the bonded seam joint strength. The results of this test are presented in Chart 4.

The above test methods verify the quality of the initial seam. Additional tests must be run to verify the longterm environmental exposure performance of the seam strength. Any seam joint technique used for factory or field seams should be submitted to the following exposure tests.

Exposure Condition	Test Method	Time of Exposure
Soil Burial	ASTM 3083	120 days
Bacterial Exposure	ASTM G-22	30 days
Accelerated Aging	160°F./71°C. Oven	30 days
Moisture Exposure	160°F./71°C. Water Submersion	10 days

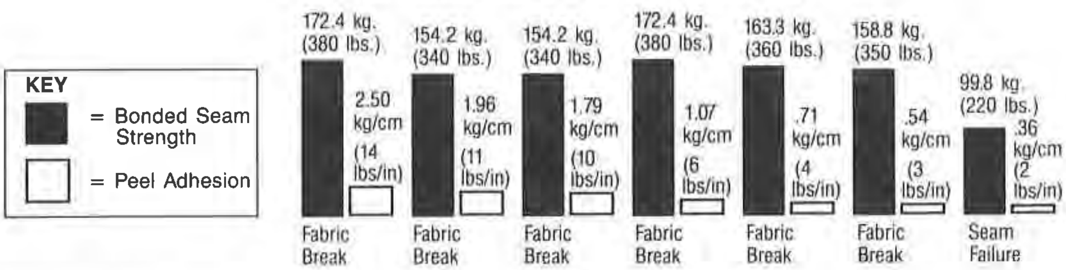
After the above indicated exposures, seam samples will be submitted to the following tests: Ply Adhesion; Bonded Seam Strength; Peel Adhesion. The test values shall not be less than 80% of the **original actual** values when tested to these methods. A value of less than 80% is an indication that the exposure condition may have a longterm adverse effect on the seam joint system.

CHART 4. BONDED SEAM STRENGTH COMPARED TO PEEL ADHESION

MATERIAL: REINFORCED ETHYLENE INTERPOLYMER ALLOY (XR-5®)

SEAM CONSTRUCTION: 2"/5.08 cm. OVERLAP DIELECTRIC

TEST METHODS: BONDED SEAM STRENGTH — ASTM D-751
PEEL ADHESION — FEDERAL STANDARD 191, METHOD 5970



An evaluation of these results illustrate that there can be a significant change in the quality of the seam joint without affecting the bonded seam strength results. The conclusion is that a peel adhesion test method is far more sensitive to adhesion variations than the bonded seam strength method. Therefore, any accelerated test procedure designed to determine the longterm exposure effect on the seam joint construction should include the peel adhesion test.

Because of the critical nature of the performance of a seam joint system to the overall performance of a geomembrane liner system, significant testing criteria should be applied to evaluating both factory and field seam joints. Following is a recommended testing protocol for the evaluation of any seam joint technique that is used in the factory or field seams to assure the longterm performance of the seam joint system. This testing criteria is designed specifically for supported or reinforced geomembrane materials. However, in philosophy, it can also apply to unsupported geomembrane materials with some revision in test methods that are more specifically appropriate to unsupported materials.

Property	Test Method
Ply Adhesion	ASTM D-413
Peel Adhesion	ASTM D-751
Bonded Seam Strength	ASTM D-751
Dead Load Test	Mil-T-52766A, Para.4.6.2.16
73°F./23°C.	Load - 50% of bonded seam strength value for 24 hours
160°F./71°C.	Load - 25% of bonded seam strength value for 4 hours.

When the geomembrane liner is to be exposed to specific chemicals, the seam joint technique used for factory and field seams should be submitted to the following exposure tests:

Chemical Exposure (Representative Chemicals)	Test Method	Time of Exposure
Acid	Immersion Test	10, 20, and 40 days
Alkali		
Aliphatic		
Aromatic		

At the end of each exposure time, seam samples will be submitted to the following tests: Ply Adhesion; Bonded Seam Strength, Peel Adhesion. Test values shall not be less than 80% of the **original actual** values. Any test value less than 80% at any of the exposure intervals is an indication that the exposure conditions may have a longterm adverse effect on the seam joint system and should not be used in that chemical environment.

In addition to the representative chemical exposure tests, the factory and field joint system employed should be submitted to the same chemical immersion tests utilizing the actual chemical fluid to which the geomembrane liner will be exposed. The test procedure and test results should conform to the same criteria indicated above.

The testing protocol identified above will provide excellent information about the quality of seam joint systems used in geomembrane liners. The test procedures identify the initial quality of the seam joint. The various recommended exposure conditions provide confidence that the seam joint systems will perform in longterm environmental exposures.

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Nondestructive Technique for Assessing Field Seam Quality of Prefabricated Geomembranes

Research was conducted to test various nondestructive testing (NDT) methods that are or can be used to assess the quality of field seams in synthetic geomembranes. The objective of the work was to find a universally applicable technique and demonstrate its capabilities for field use on actual field seams.

The research was conducted in two phases: laboratory investigations and a field demonstration. Sixteen different geomembrane materials were considered with assorted seam defects including sand inclusions, gas bubbles, and masking tape spots.

One technique, ultrasonic impedance plane analysis (UIP), was found to work on nearly all of the seams studied, including seams made from materials with woven scrim reinforcement. The UIP technique was demonstrated under simulated field conditions on a variety of materials. Conventional pulse-echo ultrasonic testing (UT) was found to be applicable to most of the nonreinforced materials.

INTRODUCTION

Mill tailings ponds at uranium mills employ synthetic liners to prevent the pond contents from seeping into the ground. These liners, commonly referred to as geomembranes, are made up of large sheets of synthetic polymer material that are seamed in place to form a continuous barrier. Pacific Northwest Laboratory (PNL) contracted with the Low-Level and Uranium Projects Branch of the U.S. Nuclear Regulatory Commission to study certain aspects of mill pond synthetic geomembranes; one task of the study was to investigate and demonstrate nondestructive testing (NDT) techniques that may be used to assess seam quality in the field.[a] The specific objectives of this task were to evaluate current and potential NDT techniques that could be used to assess the quality of field seams and to demonstrate in the field an NDT system that would be more effective than commonly used seam inspection techniques.

After simulated seams with intentional defects were made from four different geomembrane materials, 14 testing techniques were evaluated for effectiveness. This report contains the results of those evaluations as well as the results of demonstrating two NDT techniques which were evaluated under simulated field conditions.

(a) This paper is an account of work performed for the U.S. Nuclear Regulatory Commission under Contract DC-AC06-76RLO 1830 (Fin. B2476). The Pacific Northwest Laboratory is operated for the U.S. Department of Energy by Battelle Memorial Institute.

RESULTS AND CONCLUSIONS

Our research demonstrated that three of the currently used NDT techniques are adequate for finding unbonded areas (unbonds) of seams if an unbond is along the edge of the top piece of a seam or if an unbond extends clear across a seam to form a leak. These techniques (air lance, vacuum chamber, and pick test) will work on any liner material encountered in this research, including nonreinforced and reinforced materials. The pick test is quickest and requires the least operator skill, while the other two are more difficult but yield better information regarding the extent of an unbond. It is important to note that none of these three techniques are capable of detecting an unbond contained wholly within a seam or one that is along the edge of the bottom side of a seam.

Conventional pulse-echo ultrasonic testing (UT) is currently used to assure seam quality by one installer of lap welded high density polyethylene (HDPE) geomembranes. Our research shows that UT can be used to inspect any of the nonreinforced liner materials considered except for fillet welded HDPE. Equipment costs for the pulse-echo technique are significantly higher than costs for the three previous methods. The operator skill required is comparable to the skill required for the vacuum chamber test.

The most significant result of this work was finding that the ultrasonic impedance plane (UIP) technique could be used to find unbonds in seams made from almost all of the materials investigated. This technique is alone among acoustic methods in that unbond detectability is not adversely affected by the reinforcing scrim laminated into the sheet material. The UIP instrument is capable of detecting unbonds as small as 6.4 mm in diameter, regardless of location in the seam. During the field demonstration, most of the indications were cut out of the seams for destructive analysis. These pieces all showed either voids or greatly reduced peel strength in the areas detected by the UIP instrument. When properly calibrated, this instrument requires less operator skill than pulse-echo UT and is somewhat faster. On 76-mm-wide seams, our inspection rate reached one meter of seam per minute using a 9.5-mm-diameter transducer mounted in a transducer holder. The equipment used for this technique is even more expensive than the pulse-echo UT equipment. Even though we found many indications during the field demonstration, very few of the corresponding unbonds would have been considered of rejectable size according to acceptance criteria followed by most liner installers.

SEAM SAMPLES

During the laboratory phase of this research, four types of geomembrane material were studied: polyvinyl chloride (PVC), high density polyethylene (HDPE), chlorinated polyethylene (CPE), and chlorosulfonated polyethylene (CSPE). Of these four materials, the CPE and CSPE were reinforced with loosely woven polyester scrim. Thickness of the materials was between 0.76 and 0.89 mm, except for some of the HDPE, which was 1.5 mm thick.

Commercial geomembrane installers determine seam acceptability by the minimum well-bonded width at any point along a seam's length. For reinforced materials (CPE, CSPE), the minimum acceptable bond width for field seams is typically 41.4 mm at all points along a seam. The seams are usually made with 76 mm of overlap, so any unbond or combination of unbonds is acceptable as long as the unbonded dimension perpendicular to the seam length is less than about 36 mm. For nonreinforced adhesive-bonded materials such as PVC, the minimum acceptable bond width is 51 mm. One installer of HDPE considers 10 mm to be the minimum bond width required for extrusion welded seams. In actual field seams, the most likely causes of unbonds are areas unwetted by adhesive, dirt inclusions, incorrect bonding temperature, bubbles caused by trapped gasses, insufficient seaming pressure, drying of adhesive before bonding, insufficient solvent cleaning of bonding surfaces, and moisture.

Three geomembrane materials were cut up and then fabricated into 17 seam samples, each about 1/2 meter long. The seams were fabricated by a commercial geomembrane installer using the manufacturers' recommended procedures and materials appropriate for field seaming. Some of the seams were made with no intentional defects (unbonds), but most of them were intentionally flawed. The defect types included sand, moisture, lack of adhesive, and masking tape. The masking tape was used to make defect spots of known size ranging from 9.5 to 38 mm in diameter and strips ranging from 3.2 to 6.4 mm wide. All seams were nominally 76 mm wide. High density polyethylene is extrusion welded rather than bonded by adhesive in field installations, so samples of two types of field welds were procured from HDPE installers for this research. The two weld types were lap welds and fillet welds, and some of these also included intentional defects caused by moisture, sand, and poor welding technique.

During the field demonstration phase of this research program, we had the opportunity to perform tests on many seam and material types in addition to the four tested during the laboratory phase. The field demonstration was carried out in cooperation with a U.S. Bureau of Reclamation (USBR) research project sponsored by the Environmental Protection Agency (EPA) Municipal Environmental Research Laboratory, Cincinnati, Ohio. The USBR research program involves 21 field seams and 16 factory seams made out of various polymer materials supplied by 12 manufacturers. We tested 15 seam types which included various polymers, scrim spacings, manufacturers, and bonding systems. The seams tested included ones made from PVC, reinforced and nonreinforced CPE, CSPE (6 x 6, 1000d and 10 x 10, 1000dscrims), polyolefin, and HDPE. Each seam was about 24 meters long, although we only tested about 3 meters of each. As described later in this paper, selected field seams were also evaluated destructively to obtain correlations with the nondestructive examinations.

TECHNIQUES INVESTIGATED

One of the objectives of this research was to investigate various seam quality inspection techniques in an attempt to find one that would work best on the greatest number of materials, seam types, and defect types. Some of the techniques investigated are currently in use by geomembrane installers, while others have never been applied as field inspection techniques. In the course of this research, 14 different inspection techniques were considered. They varied in effectiveness from none at all to nearly universal applicability. Two of the techniques were demonstrated at the U.S. Bureau of Reclamation under field conditions. The most significant techniques are listed below with a general description and qualitative assessment of the usefulness of each. Unless otherwise stated, each of these techniques works on all liner types investigated.

- Visual - This technique is used to some degree by all installers. Large bubbles and "fish mouths" can readily be found by this method.
- Air Lance - A jet of air (345 kPa directed through a 4.76-mm nozzle) is blown at the edge of the top piece of a seam. Any openings along the edge become inflated and easily visible. This technique is very good for determining how far across the seam an unbond extends. The air lance requires a careful operator to detect unbonds, and works only if the defect is open on the top surface of a seam. Figure 1 shows a defect revealed by the air lance.

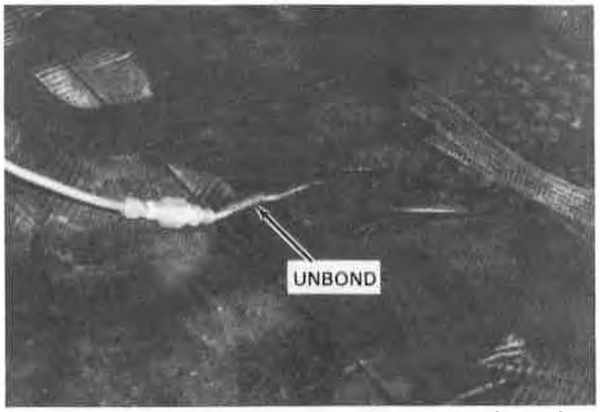

Fig. 1. Air lance detecting an unbond in a chlorosulfonated polyethylene seam.

- Pick Test - This method detects the same defect types as an air lance, but requires less operator skill. To look for unbonds, a dull object (such as a nail with the sharp point ground off) is slid along the edge of the top piece on a seam. Wherever an open unbond exists, the "pick" digs into the opening. Detectability is equal to the air lance, but the pick test does not determine the size of unbonds as well.
- Vacuum Chamber - In the vacuum test, a gasketed chamber is placed over a section of seam that has been coated with a soapy solution. The chamber, which has a transparent top, is then partially evacuated using an air pump. Any unbonds that extend clear across the seam are then indicated by the presence of soap bubbles as shown in Figure 2. This test is very sensitive to small leaks, and has the additional feature of stressing the seam to cause marginally bonded areas to become unbonded and show indications.
- Sample Destructive Tests - At regular intervals during installation, seam samples are made up or cut out and then destructively examined, usually by peel testing. This method is useful for detecting gross problems in the seaming process, such as defective glue, incorrect bonding temperature, etc.
- Monitoring of Welding Parameters - For seams that are thermally joined, certain key parameters need to

Fig. 2. Vacuum chamber detecting a leak in a chlorosulfonated polyethylene seam.

be controlled to permit a good bond to form. These parameters may include extrudate temperature, hot air temperature, seaming rate, sheet temperature, and contact pressure. While this technique cannot assure a good bond, if any of the parameters are violated, then a good bond will not result.

- Pulse/Echo Ultrasonics - This technique involves transmitting a pulse of ultrasonic energy into the seam and interpreting the returned echo on an oscilloscope. A well-bonded area would produce an echo later in time than an unbond because the ultrasonic pulse would have travelled round trip through two sheet thicknesses, whereas at an unbonded area the echo returns after a round trip through only the top sheet thickness (see Figures 3a and 3b). To assure good bonding throughout a seam, the entire seam surface area must be scanned with a transducer. Pulse-echo UT is currently used by one HDPE installer, and our research indicated that the technique works equally well on all the nonreinforced materials that we tested, such as PVC or polyolefin.

The following seam quality testing techniques are not currently employed by geomembrane installers, but were investigated as part of this research.

- Infrared Camera - Using an infrared video camera system allows the detection of seam areas that are at different temperatures. One characteristic that would cause temperature differences is heat transfer differences resulting from areas that are not intimately bonded. An infrared inspection consists of using the infrared camera to view the length of each seam while looking for areas that are cooler than most of the seam. This technique worked only marginally in the lab, and in the field too many effects besides bond quality would tend to mask the temperature differences caused by a poor bond. Two likely masking effects are moist areas under the liner and nonuniform contact between the liner and the substrate.

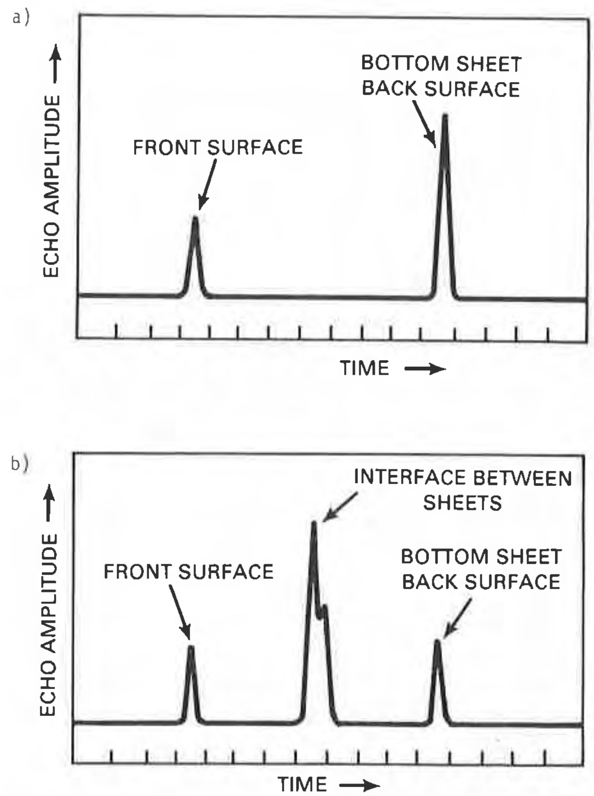

Fig. 3. Sketches of a typical Pulse/Echo UT instrument display for a) a well-bonded seam, and b) a faulty bond in a PVC seam.

- Proprietary Acoustic Instruments - Four different commercial instruments were tried on the seam samples. While each one was unique in some ways, the instruments had several features in common. Each one operated at relatively low frequency, less than 100 kHz. All of the instruments operated without a wet coupling medium between the transducer and the test piece. The instruments were intended primarily for inspecting composite materials in the aerospace industry, and all operate on a similar principle. They all apply acoustic energy to the area under test and then present the reflected energy signal on a CRT screen or a meter. None of the four instruments were capable of determining with any consistency the quality of any of the sample seams.

- Ultrasonic Resonance - In this technique, a tone burst pulse of many cycles of ultrasonic energy is transmitted into a seam by a transducer. The length of the reflected pulse varies as the quality of the bond varies, allowing the operator to determine the quality of the bond in the area under the transducer. Ultrasonic resonance worked well on nonreinforced PVC, but did not work at all on the reinforced materials. The technique is about as effective as pulse-echo ultrasonics, except that interpretation is somewhat more difficult.

- Ultrasonic Impedance Plane Technique - This testing technique was found to work best on the broadest range of materials and is described in detail in the following section.

ULTRASONIC IMPEDANCE PLANE TECHNIQUE

The inspection technique that worked the best on the most materials was the ultrasonic impedance plane (UIP) technique. The principle behind UIP is that a well-bonded area of seam possesses a certain acoustic impedance and any other area that is not as well bonded possesses a different acoustic impedance. The instrument used is capable of digitally "remembering" the characteristic impedance of the well-bonded area and then indicating visually whenever an unbonded area is detected. The primary readout on the instrument is the CRT screen, where upon a digital "flying dot" represents the tip of an acoustic impedance vector corresponding to the characteristic phase and amplitude of a test area's acoustic impedance. The other two visual indications of bond integrity are a probe-mounted LED alarm and a meter that can display either the relative amplitude, phase, or vertical resolved component of an impedance vector.

In order to use the UIP instrument, it must first be calibrated for the particular material and bond type to be inspected. The first step in calibration is to place the transducer (or probe) on a well-bonded area and to auto-null the display so that the dot displayed for a good bond is at the center of the CRT screen. Next, the transducer is placed on an unbonded area and the dot address for unbonds is stored on the screen. Another dot is then likewise stored to represent when the transducer is not acoustically coupled to the piece under test. After all likely dot addresses are stored (up to eight), the CRT display, alarm light, and meter can be adjusted to give the most convenient readouts. A modification of this procedure that we found useful in the field was to squirt some ultrasonic gel couplant between the liner layers (conveniently done by folding over a corner of the sheet) to simulate a well bonded area. Calibration was then carried out as described above while sliding the transducer between the "bonded" area and the unbonded area.

When inspecting a seam with the UIP instrument, the operator scans the transducer over the seam area while watching transducer position and the probe alarm light. If a preset adjustable threshold is exceeded, the light glows, which alerts the operator to look at the CRT screen. He can then determine from the displayed dot position what the condition of the bond is in the area beneath the probe. If the area is unbonded, the flying dot will be in the vicinity of the calibration dot corresponding to an unbond condition. If the probe is somehow not acoustically coupled to the test piece (lack of liquid couplant, rough surface, etc.), the flying dot will be in the vicinity of the stored dot corresponding to an uncoupled transducer. Watching the alarm light while moving the probe, the operator can mark the boundaries of the unbonded area with a grease pencil. Figure 4 shows a calibrated instrument with the probe located over an unbonded area of a reinforced chlorinated polyethylene seam.

The probe used with this instrument is similar to a conventional ultrasonic transducer, except that it operates at a lower frequency than most conventional ultrasonic transducers and has the alarm-triggered LED built in. Several probes are available for the UIP instrument, but we had access to only one for this research. The probe used was 9.5 mm in diameter with a frequency range from 160 kHz to 185 kHz. We operated it at about 167 kHz. A liquid couplant is required with this instrument to get the ultrasound energy through the specimen/transducer interface. We found tap water to be a good couplant and used it in all of our experiments.

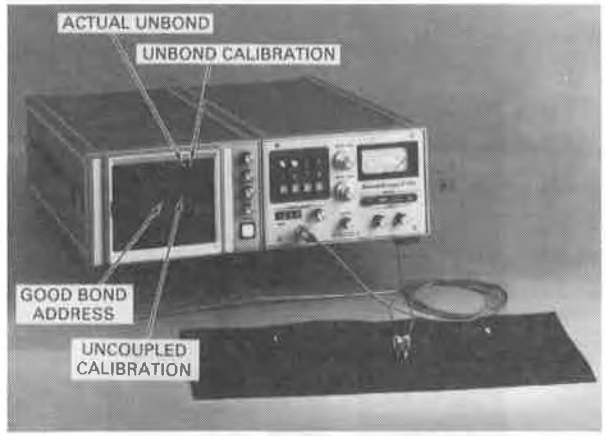

Fig. 4. Calibrated UIP instrument showing an unbond indication in a reinforced chlorinated polyethylene seam.

FIELD DEMONSTRATION

The second phase of this research was to demonstrate an NDT technique in the field on actual field seams. We originally planned to go to a field installation in progress, but we gained several advantages by participating in an EPA-sponsored research program that is being carried out by the Engineering and Research Center, U.S. Bureau of Reclamation in Denver, Colorado. In the USBR research program, a commercial geomembrane installer was contracted to fabricate a wide range of field seams under conditions close to actual field conditions. One major advantage to participating in this experiment was that we were allowed to cut out sections of the seams after NDT so that they could be destructively evaluated in the USBR laboratories. This helped greatly in establishing the validity of the techniques demonstrated. Another advantage was that since the USBR had air lance and vacuum chamber test equipment available, we were able to compare as many as six testing techniques (air lance, vacuum, "pick", visual, pulse-echo ultrasonics, and ultrasonic impedance plane analysis) on certain seams. A third benefit was the large assortment of materials we were able to test, including field seams of the four materials that were considered in the laboratory phase of our research. Our testing was carried out on a loose sand floor over hard packed dirt inside an open metal building, as shown in Figure 5.

We demonstrated two instruments at the USBR laboratories. One was a conventional pulse-echo ultrasonic unit, and the other was an instrument that operates on the ultrasonic impedance plane principle. We demonstrated the pulse-echo instrument at the request of USBR personnel, since they were unaware of the instrument's capabilities on a variety of nonreinforced liner materials. The UIP instrument was demonstrated because it offers increased effectiveness in the NDT of reinforced and nonreinforced geomembrane seams. For these demonstrations, the pulse-echo frequency was between 5 MHz and 15 MHz, and the UIP frequency was 167 kHz. Two types of couplant were used to help transmit ultrasonic energy into the materials: gel couplant for pulse-echo and tap water for UIP testing. The gel couplant was also used between two liner layers to simulate a good bond when calibrating the UIP instrument. The only modification we made to either instrument was to build a transducer holder for the UIP instrument. As shown in Figures 6 and

Fig. 5. Simulated field conditions for NDT technique demonstrations.

7, the holder is a transparent block that has a water supply line attached to it. A cavity around the transducer retains water so that adequate acoustic coupling is assured at all times. The holder allows observation of the alarm LED and is more comfortable for the operator to hold, so it helped increase the UIP inspection rate while preventing false alarms from lack of couplant.

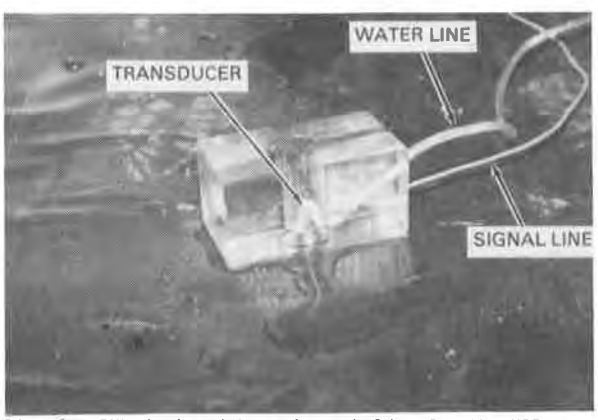

Fig. 6. PNL-designed transducer holder for the UIP instrument.

POTENTIAL AREAS FOR FURTHER DEVELOPMENT

In order for the ultrasonic impedance plane technique to become more useful for the geomembrane industry, several areas should be further investigated.

- Several installers mentioned a desired inspection rate of 2 to 3 meters per minute, which is about twice as fast as we achieved at the field demonstrations. A good way to speed inspection would be to develop a transducer wide enough to test the entire seam width in one pass. The feasibility of this approach was established by other researchers using a predecessor to the UIP instrument they found

Fig. 7. Transducer holder being used with the UIP instrument to inspect chlorosulfonated polyethylene seam.

that a wide transducer worked well for bond inspection.

- Our calibration procedure was convenient, but on a few materials at the field demonstration, it led to ambiguous results because not all dot addresses encountered during inspection had been previously accounted for during calibration. Using carefully made seam samples with known defects similar to our lab samples would be a likely place to start this work.
- Many seams need to be tested both nondestructively and destructively to arrive at acceptance criteria that correlate UIP test results with actual seam peel or tensile strengths. Sets of these correlations would be needed for each liner material to be inspected.
- We were limited to only one UIP transducer for this research, although several are available. It would be helpful to try the others to optimize the instrument/transducer combinations for each material. In particular, a transducer of a different frequency and diameter may eliminate the sometimes ambiguous results from the wider spaced scrims.
- Hardware could be developed that makes a hardcopy (such as a strip chart) of each seam inspection as it occurs.
- The UIP instrument could be adapted for monitoring the quality of factory seams in the manufacturing plant. The instrument performed well on factory seams at the field demonstration, although an automated inspection would be desirable for process control.

ACKNOWLEDGEMENTS

I would like to acknowledge assistance on this project from Rod Newton and Paul Arms of Northwest Linings and Geotextile Products for helping with fabrication of laboratory seam samples and also from Ron K. Frobel of the USBR Engineering and Research Center, Denver, Colorado for providing samples and a site for the field demonstrations. I would also like to thank J.C. Harris of PNL for his technical assistance and for designing the transducer holder.

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Extrusion Welding of High Density Polyethylene Geomembranes

For the application of geomembranes to groundwater protection, the technology used to join each sheet with another is of critical importance. The seams in a lining system are subjected to the same physical and chemical demands as each sheet of geomembrane. They must, therefore, withstand the full range of physical, chemical and biological loadings that will be encountered. Much attention has been focused in recent years on loading requirements for base materials, but the fact that these demands must also be made of geomembrane seams is too often over-looked. This paper discusses the requirements for geomembrane joining generally and explains how one particular joining technology - extrusion welding for high density polyethylene - addresses these requirements in practice.

GENERAL REQUIREMENTS FOR SEAMS: PHYSICAL, CHEMICAL AND BIOLOGICAL RESISTANCE

No matter the joining method, no matter the geomembrane material, a seam must withstand, both short term and long term, all lining system requirements for mechanical, chemical and biological loadings. Virtually all authorities agree, not surprisingly, that a seam should withstand 100% of the stress that the parent sheet endures. The seam area and any additional material used in the seam must demonstrate the same chemical resistance as the geomembrane itself. In this regard, glued connections and others that use materials different from the base material of the geomembrane may increase the risk of seam failure. The seam must furthermore be so constructed and fabricated that damage to the geomembrane, through the use of additional materials or through extreme energy transfer (which would result in adverse molecular structural changes to the sheet), is avoided.

In the geometrical construction of the seam, the objective is to achieve uniform stress transfer and distribution from one piece of geomembrane to another, through the seam. Thus, notch effects and stress concentrations caused by acute cross sectional changes must be avoided.

GENERAL REQUIREMENTS: QUALITY ASSURANCE AND TESTING

Any seaming process should, to the greatest extent possible, be independent of prevailing weather conditions. Wind, humidity and prevailing temperature must be observed, and equipment either automatically or manually adjusted for each of these conditions.

Both non-destructive and destructive testing, for seam impermeability and seam strength, should be carried out. The mechanical strength of a seam must be tested destructively through the removal of seam samples at periodic intervals in order to determine their strength. The joining process must, however, enable the repair of those areas from which samples were removed, as well as the renovation of problem areas.

There are several commonly-used methods for welding HDPE geomembranes; the most commonly used are extrusion welding and hot air welding. The following sections discuss the characteristics of the extrusion welding process developed, patented and used today by Schlegel Lining Technology.

EXTRUSION OVERLAP WELDING

Typical Extrusion Overlap Welding Detail
Figure 1

During extrusion overlap welding, a bead of extrudate is introduced by a mobile extruder between the preheated surfaces of overlapped lining sheets. These edges are then pressed together using a pressure roller system.

Pre-heating is done with hot air, and is controlled in such a way that the surface temperature of the sheets reaches the correct temperature for welding, independent of weather conditions. The bead of extrudate is introduced into the welding area at temperatures at or slightly above that of the pre-heated sheets. The pressure system that immediately follows the extrusion is sufficient in force and duration to effect a bond. Welding speed depends on environmental conditions and the capacity of the equipment.

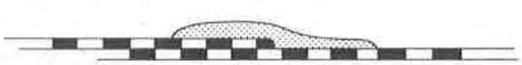
Theoretical and practical findings show that it is especially important that minimum thermal energy is introduced into the welding areas of the sheet surfaces in order to avoid total heating of the sheets, which could cause excessive softening, structural disarrangement and resulting weakening of the material. The bead of polyethylene melt is, therefore, as thin as possible, but consistent with achievement of the desired bond value which for Schlegel Lining Technology is FTB (film tear bond, i.e., the weld sample will not peel but the parent sheet will elongate and tear in a tensile peel test).

The characteristics of the extrusion overlap method may be summarized as follows:

- An assured connection for force transmission, even with variations in extrudate thickness;
- Uniform transfer and distribution of stress from one sheet to the other;
- No special fixing or alignment of the sheets is necessary before welding;
- Contact pressure, through an integrated pressure roller system, can be precisely and safely achieved;
- Extrudate material (HDPE) is minimally affected by weather and aggressive media;
- Semi-automated procedure, with control over each significant variable;
- Discontinuity testing possible with ultrasonic method; and
- High strength factors and a high welding factor with low variation

The extrusion welding process is, however, one that must be carefully controlled to achieve desired results. Even pre-heating of the surfaces of the upper and lower sheets must be carefully adjusted and controlled. Due to the level of operator skill and training required, Schlegel Lining Technology performs extrusion welding only with its own trained personnel. Surface treatment by mechanical abrasion is required, and eye control over the welding bead is necessary.

EXTRUSION SURFACE WELDING



Typical Extrusion Surface Welding Detail
Figure 2

Extrusion surface welding, in contrast to extrusion overlap welding, places extrudate over the edge of the sheet, rather than between overlapping edges. This is normally used around pipe penetrations and hard-to-reach areas. Just as with overlap welding, however, pre-heating of the zone at the sheet edge is required to achieve internal melting and bonding to the welding material. With overlap welding, mechanical strength is transmitted by the shearing force resistance of the upper sheet and the extrudate, and again with the extrudate and the other sheet. In contrast, surface welding transmits strength through the overlayed welding material, which therefore must carry the entire force transmitted by the two sheets. The overlayed extrudate must, therefore, be at least as thick in the crossover zone at the edge of the upper sheet, as are the two sheets themselves. When the increased stresses and weld factors are taken into account though, the thickness of the weld bead is actually greater than the two sheets' thickness. Special demands are therefore made regarding the alignment of the extrudate bead, as well as other adjustments in maintaining a minimum bead thickness.

Surface treatment is also easier to perform than with overlap welding, where the sheets face each other and where both are abraded.

The characteristics of the surface welding procedure are:

- Pre-treatment of surfaces by mechanical abrasion (brushing systems, for example) is easily performed and controlled;
- Adjustment and control of the pre-heating temperature is easier than it is for overlap welding;
- Even distribution of stress is achieved provided there is proper geometric placement of the weld bead, as for example, through gradual cross sectional crossover; and
- No further treatment is necessary on corners or crossovers.
- Welding speed is usually slower than that of overlap welding;
- Before welding, fixing of sheet edges is required;
- Correct alignment of the weld on the edge of the top sheet is important;
- Maintaining a minimum thickness of the weld bead is important;
- Ultrasonic testing is more complicated than with overlap welding;
- The extrudate is exposed to weather;
- Patching is the only way to repair defective areas; and
- The top sheet must be lifted and beveled at the edge to avoid a notch effect in the seam area

PRE-TREATMENT PROCEDURES

Practical experience, as well as research in testing institutions and by Schlegel's raw material supplier, have demonstrated that mechanical abrasion of the seam area is necessary. This pre-welding preparation is independent of the welding process and is used to achieve a constant high level of bond strength with a minimum weld width. Mechanical preparation is usually done with a rotating brush, and it is particularly important for sheet sections that have been exposed to weather.

PRE-HEATING: HOT AIR

Hot air tools are the easiest and safest way to preheat the surfaces of the sheets to a required temperature under construction site conditions. Hot air offers the further advantage that surfaces are again cleared of loose particles, and any remaining moisture evaporates from the seam area.

The hot air temperature is adjusted so that an exact pre-heating of surfaces is achieved. Since surface temperature is only partially influenced by the distance of the hot air nozzle from the sheet, exact positioning is not required. Hot air nozzles are also closely connected with the welding nozzle so that immediately after pre-heating, extrudate laying and compression of the seam area takes place. For these reasons the application of hot air tools has, so far, proved to be superior to other pre-heating systems.

PRESSURE APPLICATION SYSTEMS

To ensure a good weld it is necessary to compress the weld area with precise pressure. Theoretical considerations and practical experience have determined that independently operating rollers are optimal.

This pressure roller system is primarily suited for extrusion overlap welding. For extrusion surface welding there is the risk that the extrudate may be pushed aside by the pressure of the rollers, and thus the welding seam thickness would be less than the required minimum level. For extrusion surface welding, therefore, the required pressure is manually controlled through proper die design and machine handling.

CONCLUSION

The extrusion overlap welding has been developed to a very high degree of automation for welding both pond bottoms and slopes. The procedure is applicable under most weather conditions. Results are consistent and of good quality when mechanical pre-treatment of the seam areas is performed.

Extrusion surface welding is used primarily as a manual welding procedure. Short and long-term testing for the mechanical characteristics of extrusion surface welds does not show any fundamental differences from the overlap welds.

Each method requires a degree of skill in certain aspects of its operation, in order to assure welded seam quality, and none of the three methods are used casually. The need for experienced welding technicians, trained in the requirements of each system, cannot be overstated.

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A Detection and Monitoring Technique for Location of Geomembrane Leaks

There are three important considerations regarding potential leaks in geomembranes; they are detection, location and magnitude. This paper presents a method aimed at a solution to these aspects of geomembrane leak problems. The method is based upon acoustic emissions which are sounds generated by the leak, picked up by wire wave guides beneath the geomembrane, transmitted to the sides of the site, and then monitored. Laboratory tests show that increasing head, larger hole size and closeness to the wire wave guides result in higher emission rates. Large scale laboratory tests confirm these findings and also illustrate the leak location procedure.

INTRODUCTION

Due to their assumed function of liquid containment and to their relative thinness with respect to other types of liners, leak detection is critical in all geomembrane systems. The basic problem, however, is threefold for not only is the presence of a leak important, but also its location and magnitude. Indeed, if a geomembrane is found (or suspected) to be leaking, a major task still remains in locating the leak, and determining to what extent is the severity of the problem. The reason for this multifacted aspect of leak detection is that the existing reservoir, impoundment, pond or landfill generally cannot be lifted out of place or temporarily stored elsewhere while the leak is located, examined and repaired.

The preceeding comments are particularly relevant when one considers the known methods of leak detection. These are downstream monitoring wells, double liners with leak detection pipes, electrical resistivity and time domain reflectometry.

Regarding downstream monitoring wells, many problems exist. Placement of the wells as to location and depth is difficult to define, as is also the type of well and its sampling procedure. The frequency of sampling is also critical where a balance between large numbers of samples and cost of sampling must be reached. Lastly, the assessment of the samples is far from trivial in most circumstances. Here, comparison with data from upstream wells is usually the best approach.... but, the costs can become intolerable.

The use of a double liner with a gravel encased, perforated drain pipe system between the primary and secondary liners has also been used.(1) This leak detection pipe system must be gravity drained to a low point in the containment site and periodically inspected for leakage of liquids through the primary liner. Other than the obvious doubling of cost of the containment system, the method has the serious drawback of not being able to identify the location of the leak or if multiple leaks are present once flow is seen in the leak detection pipe outlet.

The method of electrical resistivity has been used to locate and trace seepage plumes of conductive liquids as they travel beyond the containment site.(2) These seepage plumes are mapped out using resistivity contours as they disperse from the site. By back estimating from the general direction of the flow contours, the leak can be located on a very approximate basis. Within the containment site, however, the technique is relatively useless since the entire area itself is often highly conductive. Thus the technique can usually be characterized as being one that identifies the leak after a considerable quantity of seepage has occurred and the plume has entered the downstream water regime.

Lastly, time domain reflectometry has been attempted on an experimental basis.(3) Here, wire wave guides are placed in sets beneath the geomembrane and amplitude versus frequency response curves are generated. By suitable interpretation, the leaking area is reasonably identified. However, a considerable effort seems to remain to bring the technique to a practical method.

With these thoughts in mind, it is felt that a leak detection, identification and monitoring system is definitely needed and that all possible schemes should be explored. One such possibility can be based on the sounds generated as the escaping liquid passes through and beneath the geomembrane. If proper wire wave guides have been placed, the sounds can be detected (within obvious limits), located and monitored in an advantageous manner. The technique, properly called acoustic emission monitoring, is the focal point of this study.

ACOUSTIC EMISSION OVERVIEW

Acoustic emissions (AE) are sounds generated within a material which has been stressed or disturbed from its equilibrium state by some external means. Sometimes these sounds are audible (wood cracking, ice expanding, soil and rock particles abrading against one another, etc.), but more often they are not, due to their low amplitude or high frequency, or both. A piezoelectric sensor (transducer) is generally used as a pickup to detect the emissions. This transducer produces an electrical signal proportional to the amplitude of sound or vibration being detected. The signal is then amplified, filtered, and counted or recorded in some quantifiable manner. The counts, or recordings, of the emissions are then correlated with the basic material behavior to empirically determine the relative stability of the material in question. Usually, if no emissions are present, the material is in equilibrium and thus stable. If, however, emissions are observed, a non-equilibrium situation exists which may eventually lead to an undesirable situation or even failure of the material.

It should be mentioned at the outset that the technique is not new. Only its application to geotechnical problems is new. The original soils reference being in the late 1960's, and our work at Drexel University commencing in 1971. Of particular interest to those considering use of the technique are the proceedings of the three Conferences held at the Pennsylvania State University[4] and a recent ASTM Symposium[5]. For the purpose of this paper, some past work on seepage monitoring (both laboratory and field) has lead to the current study.

Regarding leaks in pipelines and subsurface metal storage tanks, the technique has been used to good advantage[6]. The leaking fluid sets up vibrations in the metal which are transmitted to sensors placed at distant points and source location is usually possible. The generally high pressures and good propagation characteristics of the metal lend themselves well to AE monitoring.

On soils work in the laboratory, both clear water and turbid water were passed through a column of gravel at various flow velocities[7]. The AE sensor which was used was a waterproof hydrophone embedded in the center of the soil column. For each test performed, the AE response was initially high but then subsided to a near constant value. For the case of clear water seepage, flow rates of approximately 45 ml/s were required for AE detection using equipment and sensitivities given in Ref. 7. For the case of turbid water seepage (38 g of clayey silt per liter of water caused the turbidity), the minimum detectable flow rate was approximately 10 ml/s. The behavior of AE rates for flow velocities above these minimum values behaved as an exponentially increasing function. It was seen that high flow rates produced proportionally many more emissions than did the lower flow rates.

On soils work in the field, a problem of seepage beneath a small earth dam of 12 ft. in height and approximately 1200 ft. in length created a major burden to a site developer[8]. While grouting was an obvious solution to the problem, the cost involved in grouting the entire length of the dam was prohibitive. Thus, a series of borings was made along the axis of the dam and flow rate tests were conducted. The results indicated that a specific 200 ft. section seemed most likely to be the major contributor to the loss of water. Since the borings were available, AE monitoring was also attempted. The plastic casing could not conduct emissions and therefore was not suitable as a wave guide. Instead a heavy steel wire was inserted down the borehole to the bottom where the seepage was presumably occurring. AE count rates were recorded. The general agreement between seepage rates and AE count rates in the same 200' zone was noted. The actual mechanism causing the emissions was not known (perhaps it was the erratic flow of the seepage against and around the casing), but the use of the AE technique in monitoring for seepage seemed to hold great promise.

In closing this section, it should be noted that the AE monitoring equipment being used plays a vital role in working with this technique. The variations of equipment configurations can obviously be great, which leads to many possible systems. Currently a number of companies are making single channel AE systems for geotechnical use (Acoustic Emission Technology Corp., Dunegan Corp., Slope Indicator Co., Weston Geophysical Corp.) and some are making multi-channel systems (Acoustic Emission Technology Corp. and Physical Acoustic Corp.). The option exists of making a personalized hybrid system consisting of commercially available components from any one of a number of equipment manufacturers.

TUBE TEST AND RESULTS

In order to determine baseline data for larger scale tests (and eventual field tests), a 6" diameter plastic tube with a 20 mil PVC geomembrane covering its bottom was placed on a crushed stone foundation. The plastic tube allowed for water flow at controlled values of head as the water passed through the geomembrane which had holes of various sizes prefabricated in it, see Figure 1. A number of test series were performed with this general configuration. Each will be briefly described and a table of AE count rates used to compare the different situations.

Series 1 - This test configuration was close to the seepage study described earlier, except now the flow was downward and the hydrophone pickup was placed in the stone base beneath the geomembrane. The results are given in Table 1 where it can be seen that head, hole size and distance from the edge of the hole to the edge of the hydrophone were the variables evaluated. The trends in AE count rate are quite well behaved with larger hole size, larger head causing flow and closeness of sensor to hole all producing higher count rates. While this indeed is a viable method for AE leak detection, the cost of embedding sensors beneath the entire liner at close spacings would be cost prohibitive in most situations.

Series 2 - This test setup used an AE sensor on top of a 1/2" diameter steel rod (as a wave guide) which was placed in the tube vertically, touching the geomembrane at various distances from the leak. The AE vibration had to travel from the leak through the geomembrane or stone base, then up the rod to the sensor. Test results are given in Table 2 where head, hole size and distance from the hole to the bottom of the rod were the variables. Note that a significantly more sensitive AE monitoring system was used for these tests in comparison to the previous system. This current system will be used for the remaining work presented in this paper. It can be seen that logical trends result, in that higher heads, larger hole sizes and nearness to the hole result in higher AE count rates. However, it also can be seen that the attenuation of the signal in traveling along the geomembrane or stone subbase is very large. While this type of probing is acceptable for existing ponds or landfills, it seems logical in liner design to place the wave guides beneath the geomembrane during construction.

Series 3 - For situations where new geomembranes liners are to be constructed it is certainly conceivable to place wire wave guides in the material (stone base, soil subgrade, or geotextile) beneath the liner. This situation was modeled with the tube setup shown in Figure 1 by placing a horizontal rod in the stone base beneath the geomembrane. The sensor was attached to the exposed end of the rod. Four different rods of varying diameters were used in this test series; 1/2", 1/4", 1/8" and 1/16". The data for varying heads, hole sizes and distances from the hole to the edge of the wave guide is given in Table 3. While there are some inconsistencies in the data, certain trends are clearly observable.

(a) The larger the diameter of the wave guide, the higher the AE count rate,
(b) the higher the head forcing flow, the higher the AE count rate,
(c) the larger the diameter of the geomembrane hole, the higher the AE count rate,
(d) the closer the wave guide is to the geomembrane hole, the higher is the AE count rate, and
(e) only in a few cases did the wave guide at 3" away from the hole give AE counts.

TANK TESTS AND RESULTS

In order to scale-up from the previously described tube tests, a 10' x 10' tank was constructed using 18" high sides. Within the box was placed a 3" thick base course of crushed stone aggregate (1/2" maximum size) on top of which was placed a nylon mesh composite geotextile with a network of 1/16" copper wires in various configurations. These wires were brought up over the side boards to the outside of the tank where they could be used for AE monitoring, see Figure 2. A 20 mil PVC geomembrane was placed over the wires and formed the containment liner for water. The tank was constructed such that heads of 14", 10" and 5.5" could be maintained. A series of tests were conducted using this facility, with the grid pattern shown in Figure 3(a) being the one which produced the data shown in Table 4. Note that head, hole size and distance from the leak to the wave guide were the variables which were evaluated. The trends generally seen in the tube tests can again be observed. For example, the higher heads, larger hole sizes and closer distances to the hole produced greater AE count rates than did lower heads, smaller holes and further distances from the hole. More importantly, however, is that the x-y coordinate grid system estimated the location of the hole quite nicely. In all cases the x_0 and y_0 values were the highest in AE count rate, followed by the x_1 and y_1 values, etc. (The exception to this trend is the x_5 and y_5 pickup points which give spuriously high count rates at times). Furthermore, these coordinate pairs were approximately equal allowing for contours of equal AE count rates to be drawn, see Figure 3(b). The maximum contours being the obvious leak location. This feature becomes the real power of the technique for source location of the leak can be determined in this manner. As a third aspect of the technique, the magnitude of the leak can be inferred by the value of the AE count rate if a careful calibration is done beforehand. Easily seen is that higher count rates result from both larger hole sizes and/or higher heads which lead to this comment. Additional testing is required to further refine this aspect of the method.

SUMMARY AND CONCLUSIONS

The sensitivity of modern piezoelectric transducers and their associated amplifiers, filters and AE monitoring equipment, is quite high. For example, a disturbance pulse as small as 5×10^{-7} pounds on the face of a transducer of area 1 sq. in. will typically cause a 1 volt amplified signal to be generated.[9] Thus, when the AE method is used to monitor material instability or nonequilibrium, a very powerful and useful nondestructive testing technique is being used.

Prior work in the laboratory and field by the authors in the area of seepage monitoring using AE monitoring led to the finding that flow rates as small as 10 ml/s could be sensed and monitored.[7] Since these flow rates are generally within anticipated leak rates for small holes in geomembranes, an attempt has been made in this paper to transfer AE technology to the problem of leak detection, location and monitoring.

Using commercially available AE systems, several tube tests and various configurations of wave guides in tank tests were conducted in this study. The following generalized comments summarize the work.

- The higher the head forcing flow from the geomembrane leak, the higher the AE count rate. Heads of 24" to 6" were used during these tests.
- The larger the hole size in the geomembrane, the higher the AE count rate. Circular holes of 1/4", 1/8" and 1/16" were used.
- The shorter the distance from the edge of the hole to the pickup transducer or wave guide, the higher the AE count rate. Distances up to six inches were detected.
- Regarding pickup sensor deployment, three possibilities were investigated. The sensor directly beneath the geomembrane was seen to be effective but due to the large number of required sensors, this alternate would probably be too expensive. The sensor on a vertical rod touching the liner was seen to be effective, but impractical in many situations, e.g., landfills. Lastly, the sensors on wire wave guides in the geotextile or base course beneath the liner appears to be the most promising direction to proceed.
- The wave guides should be coupled as loosely as possible to the surrounding material since high compressive stresses cause "bleed-off" of the AE signals. Composite geotextiles appear to be a good choice in this regard.
- On wire wave guide deployment scenarios, the most obvious location is beneath field seams. Depending on their number, beneath factory seam locations would be the next obvious choice. Beyond that, a uniform grid of about 12" spacing is recommended. This spacing should pick up most leaks and heads of the type evaluated in this study.
- When the wave guides are arranged in a grid pattern beneath the geomembrane, the location of the leak can be estimated. This feature is not readily available in any other approach toward leak detection as far as the authors are aware. It is felt to be a powerful impetus in using the AE technique.
- Since the hole size and head are seen to vary the AE count rates, the method seems capable of predicting the size (magnitude) of the leak. More work, perhaps a calibration curve for a particular situation, is needed in this regard.
- Field deployment of such a AE monitored geomembrane leak detection system is currently being considered by a number of organizations.

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Table 1 - Test Series 1 Results Using a Hydrophone in the Stone Base Beneath the Leaking Geomembrane*

Head of Water (in.)	1/4" Hole Dist. to Sensor			1/8" Hole Dist. to Sensor			1/16" Hole Dist. to Sensor		
	0"	1"	3"	0"	1"	3"	0"	1"	3"
6	90	.06	.02	1.1	.5	0	.02	0	0
12	196	5	1	12	5	.02	9	.03	0
18	260	27	8	36	24	.14	21	.24	0
21	-	-	-	150	46	-	100	-	-
24	300	48	13	340	140	.50	160	.67	0

*table units are in 1000 AE counts per minute; AE monitoring by Weston AEM System (trigger level = 0.1; gain = 1 K; high pass filter = 1 kHz; low pass filter = 10 kHz)

Table 2 - Test Series 2 Results Using an AE Sensor on a 1/2" Diameter Steel Rod Wave Guide Placed Vertically in the Tube and Touching the Geomembrane

Head of Water (in.)	1/4" Hole Distance to Wave Guide			1/8" Hole Distance to Wave Guide			1/16" Hole Distance to Wave Guide		
	0"	1"	3"	0"	1"	3"	0"	1"	3"
6	630	0	0	0	0	0	0	0	0
9	-	478	7	700	-	-	-	-	-
12	86,000	1,400	190	1,100	0	0	8	0	0
18	136,000	10,200	1,100	2,000	0	0	840	0	0
24	140,000+	15,000	2,700	11,200	150	0	2,800	0	0

*table units are in 1000 AE counts per minute; AE monitoring is by AET 204 GR system (AC 30-L sensor; pot setting = 9.18; trigger level = 0.3; high pass filter = 15 kHz; low pass filter = 45 kHz)

Table 3 - Test Series 3 Results Using an AE Sensor on Rods of Varying Diameter Placed Horizontally in the Stone Base Beneath the Tube*

Head of Water (in.)	1/4" Hole Distance to Wave Guide			1/8" Hole Distance to Wave Guide			1/16" Hole Distance to Wave Guide		
	0"	1"	3"	0"	1"	3"	0"	1"	3"
(a) For 1/2" Steel Wave Guide									
6	26,000	1,300	0	7,500	0	0	1,800	0	0
12	117,000	5,200	0	80,000	65	0	26,000	170	0
18	166,000	25,000	0	140,000	800	0	100,000	870	0
24	222,000	65,000	0	135,000	3,000	0	124,000	9,400	0
(b) For 1/4" Steel Wave Guide									
6	13	1	.02	12	.4	0	32	0	0
12	140	20	.18	85	1	0	500	0	0
18	1,050	143	.30	180	15	0	800	1	0
24	1,500	690	.52	450	74	0	1,400	12	0

Table 3 - Continued

Head of Water (in.)	1/4" Hole Distance to Wave Guide 0"	1"	3"	1/8" Hole Distance to Wave Guide 0"	1"	3"	1/16" Hole Distance to Wave Guide 0"	1"	3"
(c) For 1/8" Stainless Steel Wave Guide									
6	120	3	2	18	1	0	75	0	0
12	1,000	200	140	250	6	0	220	0	0
18	1,670	650	470	513	29	0	600	30	0
24	1,670	1,100	500	1,300	80	0	1,600	5	0
(d) For 1/16" Copper Wave Guide									
6	5	1	0	1	0	0	3	0	0
12	100	16	0	120	0	0	53	0	0
18	190	97	0	280	.2	0	140	3	0
24	300	110	0.1	470	2	0	330	74	0

*table units are in 1000 AE counts per minute; AE monitoring is the same as Table 2 information.

Table 4 - Tank Test Results Using an AE Sensor on Exposed End of 1/16" Diameter Copper Wire Placed Horizontally Beneath Geomembrane for Various Heads and Hole Sizes*

Hole Size	Location (see Figure 3) x_0	x_1	x_2	x_3	x_4	x_5	y_0	y_1	y_2	y_3	y_4	y_5
(a) For Head = 14"												
3/4"	820	152	20	.7	.3	120	1,170	78	94	36	74	1.5
1/2"	28	.4	.3	0	0	2	290	3	4	.02	.04	0
1/4"	8	.5	.1	0	0	0	65	.1	.03	0	0	0
1/8"	17	.2	0	0	0	.1	55	.1	.1	.01	.04	0
(b) For Head = 10"												
3/4"	500	40	.6	.05	0	1	400	97	81	4	31	.1
1/2"	225	16	.2	.01	0	2	370	3	8	.6	15	0
1/4"	29	3	.1	0	0	9	193	2	.5	.4	1	0
1/8"	.5	0	0	0	0	0	6	0	0	0	0	0
(c) For Head = 5.5"												
3/4"	98	35	.2	.1	0	.9	32	1	1.5	.4	2	0
1/2"	93	14	.04	0	0	.1	140	7	1	.4	.4	0
1/4"	.1	0	0	0	0	0	.8	0	0	0	0	0
1/8"	.05	0	0	0	0	0	.04	0	0	0	0	0

*table units are in 1000 AE counts per minute; AE monitoring system is the same as Table 2 information.

Fig. 1 - Photographs of Tube Test Setup and AE Monitoring During Test Series 3

Fig. 2 - Photographs of Tank Test Setup with Wire Wave Guides and AE Monitoring Equipment

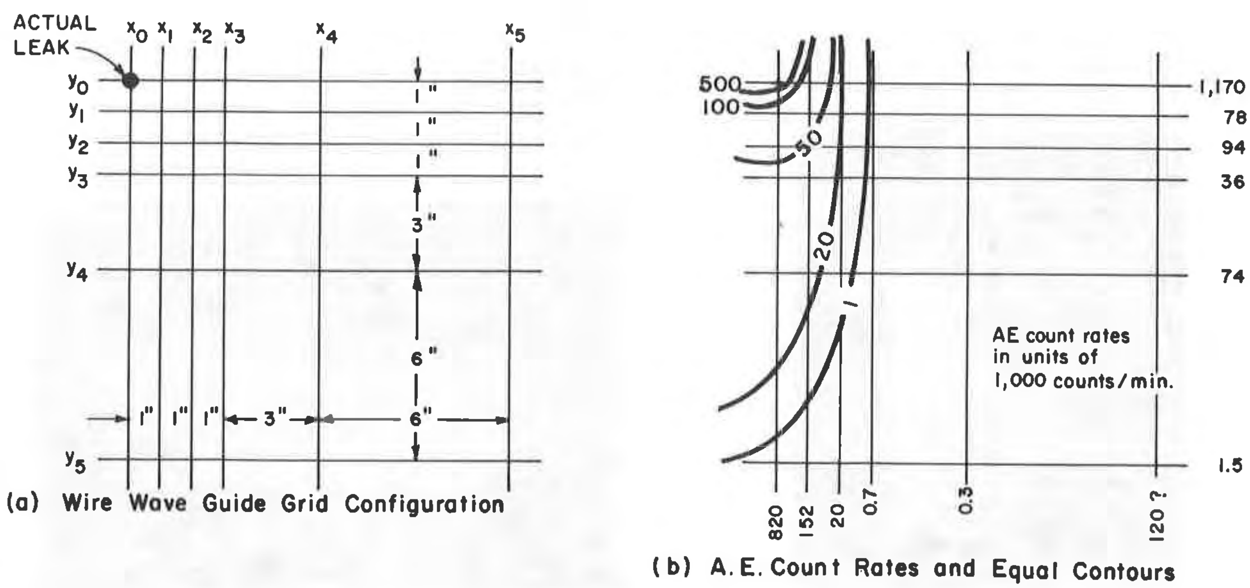

Fig. 3 - Tank Test Setup and AE Results for 20 mil PVC Liner with 3/4" Leak Under 14" Water Head

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Asphalt-Rubber Geomembrane at Palo Verde Nuclear Generating Station

An asphalt-rubber spray applied geomembrane was placed at the Palo Verde Nuclear Generating Station (PVNGS) near Phoenix, Arizona, during 1981, incorporating a high percentage of ground tire rubber. This report is an evaluation of the current charactertistics of the geomembrane after being left exposed to the elements for two years, as well as a review of quality control testing during and after construction. Recent field samples were tested and results reported before and after maintenance treatment.

The application of the Arm-R-Shield asphalt-rubber membrane at the Palo Verde Nuclear Generating Station near Phoenix, Arizona, represents the most extensive use of this product.

The Palo Verde Nuclear Generating Station (PVNGS), presently under construction, will be one of the largest nuclear generating facilities in the world when construction of the planned three units is completed. The final power output will be 3800 megawatts, which will be supplied to Arizona, Southern California, New Mexico and West Texas.

The cooling facilities at PVNGS consist of a single cell 0.3 Km^2 (80 acres) water storage reservoir, which is used to store treated sewage effluent, and a single cell 1.0 Km^2 (250 acres) evaporation pond which is used to retain cooling tower blowdown. The liquid wastes collecting in the evaporation pond will contain dissolved solids up to 300,000 parts per million. The plant is designed so that there will be no liquid discharge from the site when the water reaches the end of the cooling cycle. All water used for cooling will be retained at the site. Treated effluent is used for condenser cooling water make-up, rather than using groundwater resources or any other large body of water in an arid region.

To determine the liner type or types to be used for containing cooling water in the storage reservoir and blowdown wastes in the evaporation pond, detailed studies were done. These studies evaluated chemical resistance, ultraviolet exposure, availability, constructability and economics; and included asphalts, rubberized asphalts, soil cement, on-site clayey soils, bentonite and factory prepared membranes such as the polyethylenes.

Based on the various test results and observations, including flow characteristics of hot spray-on linings along the slopes, economy, warranty and environmental concerns, it was decided to use a minimum 200 mil thick asphalt-rubber lining on the bottom surface of the reservoir and ponds. Polyethylene membrane was used along the side slopes.

Grade preparation of the pond surfaces was completed in early August, 1981, and the manufacturing and application of the asphalt-rubber membrane began. The asphalt storage tanks, mixing equipment, distributor trucks and substantial quantities of rubber were moved to a manufacturing site established near the generating station so that the material could be prepared on site and delivered immediately to the job for application. The special blend of asphalt required for the product was prepared at the Phoenix terminal of the Arizona Refining Company and trucked to the manufacturing site for storage and preheating. This asphalt was heated to 204^{o}-$218^{o}C$ (400^{o}-$425^{o}F$) and the blend of devulcanized and vulcanized rubber was added in a specially designed mixing unit at a rate of 20 percent of the total compound. When the asphalt-rubber had been blended into distributor trucks, it was reheated to 204^{o}-$218^{o}C$ (400^{o}-$425^{o}F$).

Prior to application, the surface was graded so that no protrusions or cavities larger than 10 mm (3/8 inch) existed. The surface was then further smoothed by brooming off any surplus stone, rolling with a steel wheeled roller and, finally, hand raked to assure a smooth surface for proper application of the membrane.

As soon as the surface was prepared it was lightly sprayed with water to facilitate bonding of the asphalt-rubber membrane to the dry surface. Approximately 3.4 L/m^2 (3/4 gals./sq. yd.) of material was applied at a slight angle to the surface, with a large asphalt distributor truck using a side spray bar to prevent damage to the prepared surface by truck tires. After completion of the first pass, the procedure was repeated in the opposite direction, to assure that any pinholes or holidays were covered, as well as to assure covering of any uncoated material. Upon cooling, the result of the two applications was a seamless, tough, elastic membrane with a thickness in excess of 5 mm (200 mils).

The highest production rate achieved on any single day was 8090 m^2 (522,720 sq. ft.), completing the lining installation on both ponds by early December, 1981.

Discharge of waste water from cooling tower test operation commenced immediately. Filling of the water storage lake began shortly thereafter.

Field and Laboratory Tests

Climatic conditions were a very important consideration in deciding on a durable liner blend. Southern Arizona desert areas are subjected to very high summer temperatures reaching 48°C (118°F) and winter time temperature as low as -6°C (22°F). Surface temperatures of asphalt concrete pavements have reached as high as 82°C (180°F) on hot summer days. The geomembrane liner will ultimately be covered with water and therefore be protected against the harmful ultraviolet rays but portions of the liner will remain exposed for many months do to the uneven bottom on the evaporation pond.

In choosing the exact formula to produce the desirable liner product characteristics, the laboratory spent four months and tested 164 possible formulas to produce these desired characteristics. No less than eight different sources of asphalt were tested independently and in combination as well as the various percentage blends of high flash aromatic extender oils and various combinations and types of ground scrap rubber. The chosen formula included two types of asphalt, a high viscosity paving grade and an air blown asphalt which allowed the use of a higher percentage of aromatic extender oil giving us the correct asphaltene/resin balance to react a three way ground rubber blend. The purpose of this investigation was also to identify the greatest possible property range achievable by compounding the combination of asphalt, extender oil and ground rubber.

The test procedures for testing and evaluating asphalt-rubber blends are not uniform with straight paving asphalts in as much as you are dealing with an elastomeric aggregate as part of the blend and when attempting to run ductilities on these blends you can break the film at one of the rubber particles. Therefore, it was determined that the high and low temperature properties and the identification of the application viscosities were the most important features we were looking for. The test on viscosity was ASTM D-2994 Brookfield Viscometer measurements in centipoise. This test verifies the ability to achieve a viscosity range that will satisfactorily allow you to get a uniform spray pattern and therefore a uniform membrane thickness. When mixing the asphalt and rubber the A/R blend goes through a gel phase prior to the viscosity starting to decline at which point you have achieved a reaction between the three components. Experience has taught us that the viscosity must drop to below 1800 centipoise at 204°C (400°F) before a spray application viscosity is achieved. Laboratory experience indicates that we averaged approximately 1075 centipoise on the most desirable blend when it had achieved its application reaction.

High temperature characteristics of the asphalt-rubber blend were determined through the ring and ball softening point test procedure. The ideal blend resulted in a softening point of 68°C (155°F).

The low temperature characteristics were determined through cold temperature flexibility testing. This test is performed with a sample 15.24 cm (6 inches) by 5.0 cm (2 inches) wide and 3 mm (1/8 inch) thick. The temperature of the sample is reduced in an air bath to the point at which it no longer will bend 90° over a one inch mandrel without cracking. The selected A/R blend would pass the mandrel bend test a -14°C (7°F).

To guarantee a membrane thickness of at least 200 mils it was calculated it would require 7.2 L/m^2 (1.6 gal. per yd^2) of hot 204°C (400°F) A/R in place membrane produced an average thickness of 246 mils at the application rate indicated above. Jobsite mixing of the asphalt, extender oil and rubber blends did not quite correlate with the laboratory blending of the ingredients and testing after reacting in litre size test batches. Although we did in fact achieve the predicted high and low temperatures properties. Daily field samples averaged between 65°C (150°F) to 71°C (160°F) softening point and cold temperature flex characteristics ranging between -15°C (5°F) and -13°C (9°F). The 13,000 L (3435 gals.) distributor truck batches of the asphalt-rubber produced higher viscosities than that which was anticipated in the laboratory and required a longer reaction time to react the rubber in the blend. It is assumed that the 300 L/min. (80 gals./min.) blending rate resulted in a product that took more time to react in the large tank truck than the small laboratory vessel. This did not in any way reduce our production efficiency as indicated previously by the ability to place up to 8000 m^2 (522,000 sq. ft.) membrane per day.

Construction problems in the nuclear generating units has resulted in substantial delays in the start up and power generation at this nuclear plant. It is now anticipated that low power generation from unit 1 will begin sometime in late 1984. At the time the liner was placed it was anticipated that the low power start would occur in summer 1982 and therefore it has left the asphalt-rubber membrane exposed for over two years to the ultraviolet rays instead of being protected by water as originally anticipated. Recently large squares of the A/R membrane were removed from the evaporation pond and brought to the laboratory for testing of current properties. We used the same testing procedures as those mentioned previously for evaluating the proper blend that was chosen for construction.

The Brookfield Viscosity of the removed membrane indicated a viscosity of approximately 8,000 centipoise at 400°F compared to 1200 to 1400 at time of placement. This would indicate that the exposure to the concentrated ultraviolet had removed some of the oils that had been incorporated into the blend during construction. The ring and ball softening point had increased to approximately 105°C (186°F), which was indicative that the material was becoming stronger through aging. The membrane had lost some of its cold temperature flexibility and when measured it indicated that it had increased to -9°C (16°F) which was determined not to be detrimental because this was well below the anticipated cold temperatures that the membrane would ever be subjected to.

The uneven bottom on the evaporation pond has resulted in elevation differences of 3.7 m (12 ft.) across the dimensions of the 1.0 Km^2 (250 acres) pond. A section of the exposed membrane was tested with the application of a high flash aromatic lube stock applied at the rate of 0.15 L/m^2 (0.05 gals./sq. yd.). Visual improvement of the surface of the membrane was obvious within a few hours. Oxidized stretch marks began to mend themselves into a smooth uniformed surface within this short time frame. Samples were then removed from the membrane and returned to the laboratory for further testing against the tested material prior to treatment. Improved properties became apparent immediately upon reviewing the laboratory tests results, the Brookfield Viscosity had dropped from approximately 8,000 cps to 4,300 centipoise. The softening point was reduced slightly to 77°C (177°F) and the flexibility was reduced to -11°C (13°F). Resiliency and rebound testing indicates that it has also been improved with the oils now again available in the surface of the membrane to utilize the full potential of the ground rubber that was incorporated into the original blend. The estimated cost of this maintenance procedure, complete in place, is 7/10 of one cent per square foot. It is anticipated that this procedure will be utilized on the remaining exposed surface in the very near future.

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Construction of Geomembranes in Place by Spraying an Elastomer Over a Geotextile

Geomembranes can be constructed in place by spraying an elastomeric material over a suitable geotextile, creating an essentially seamless liner. Typical applications include pond liners for holding fresh or salt water, treated sewage, boiler wash, municipal and industrial wastes, and mine tailings. Physical properties of the coated fabric are largely influenced by properties of the fabric. Factors involved in choosing a fabric include weight per square yard, type of construction, wettability, horizontal wicking action, and resistance to chemical and biological degradation. Tensile and tear strength, puncture resistance, and impermeability after saturation with the elastomer are all very important. Preparation of the site before installation of this type of membrane is similar to that required for preformed sheets.

I. INTRODUCTION

The last ten years have seen an enormous increase in the number of pond liners installed for the control of water and waste. Driving forces for this activity have been the economic advantages of conserving valuable and scarce water supplies and various regulatory requirements to prevent the transfer of waste materials into the underground or surface water supplies. Some of the most frequent uses for pond liners are containment of fresh or salt water or in construction of evaporation ponds for treated sewage, chemical wastes, boiler wash, and mine tailings. Liners and covers for municipal landfills are additional uses.

In recent years, the predominant position of clay as a liner has been taken over, to a large extent, by preformed sheet materials. These are usually polymers such as EPDM, high density polyethylene, butyl rubber, or chlorosulfonated polyethylene. These may be reinforced or nonreinforced.

Quite recently, sprayed-in-place liners have begun to gain prominence. To form these sprayed-in-place liners, a geotextile is usually laid down as a base and sprayed with a liquid material which reacts chemically to form an elastomeric membrane. The construction of a liner in place has several advantages. Perhaps the most important is the fact that it gives an essentially seamless construction. The fit around intrusions and irregularities is simple and can be managed very easily with a combination of fabric and spray. The principal disadvantage is the possibility of nonuniform thickness. Areas which are too thick would waste money by excessive use of material. Areas which are too thin could be porous and would be more subject to degradation by ultraviolet (UV) light. Ways to avoid nonuniform thickness will be addressed later.

This paper will explore some of the factors involved in the construction of impermeable membranes in situ by spraying an elastomer precursor over a geotextile.

II. SELECTION OF THE GEOTEXTILE

Many factors enter into the choice of a geotextile for in situ constructions of a geomembrane. Among these are: resistance to microorganisms, chemical resistance, fabric construction, physical properties, behavior during installation, and wetting out properties.

Resistance to Microorganisms

A prime consideration in selection of a fabric for use in a sprayed-in-place liner is its resistance to microorganisms. Cellulosic fabrics such as cotton, linen, ramie, or jute, which are subject to attack by microorganisms when buried in the ground, are definitely not suitable for this type of application. Most of the fabrics considered satisfactory for these applications are synthetic polymers such as polypropylene, polyester, nylon, and acrylics.

Chemical Resistance

Chemical resistance is another important factor in selection of the fabric. Consideration of the purpose of the liner and the nature of the chemicals involved aid in the selection. If the service involves contact with harsh or corrosive chemicals, the fabric should be able to resist these types of chemicals. For example, nylon fabrics are attacked by acid. Polyester fabrics are attacked by strong alkalies. Polypropylene shows the best resistance to the widest variety of chemicals.

Even though the coating placed on the fabric may be resistant to chemicals and microorganisms, small amounts of these materials may diffuse through the elastomeric membrane and damage the underlying fabric.

Physical Properties

Physical properties of the fabric are extremely important in sprayed-in-place liners. Table I lists properties of several fabrics suitable for this use. Test data is included for uncoated fabrics and fabrics coated with Chevron Industrial Membrane, an asphalt-extended urethane. It should be noted that although the physical properties of the uncoated fabric are important, they may be much different from the properties of the finished product. Therefore, care must be taken in evaluation of any fabric; and tests must be made on the finished product, as well as on the initial fabric.

Fabric Construction

Fabric construction is another very important factor. This includes the type of construction of the fabric and its weight and thickness. For economic reasons, a lightweight fabric is advantageous. However, if a fabric is too lightweight, below about 75 grams per square meter (3 ounces per square yard), it often is too weak and too thin to provide a suitable substrate for a sprayed-in-place liner. Most of the fabrics used in this type of construction are nonwoven fabrics, produced by a number of different processes. Probably the most common is a fabric formed from a loosely held web of staple fibers fastened together by needle punching. Alternatively, the random web of staple fibers can be held together by spraying or dipping in a water-based latex. The latex can constitute up to 30% of the weight of the finished fabric. A third freqently used type of fabric is spun bonded, in which molten polymer is extruded through spinerettes onto a moving belt, giving a random network of continuous fibers. These are fastened together by pressing between hot rolls, giving a large number of cross-over contact points where the fibers are joined together by fusion. A fourth type of construction involves the use of staple fibers which are spread across the machine direction and then stitched together by continuous filament yarns. This type of construction is called stitch bonding.

Behavior During Installation

Behavior during installation varies considerably from one fabric to another. For example, polypropylene swells on absorption of hydrocarbons. This results in wrinkling of the fabric during installation if the elastomer or other coating contains liquid hydrocarbons. Nylon fabrics do not swell on absorption of hydrocarbons, but they do wrinkle on absorption of moisture from the atmosphere or substrate. This instability can cause problems, particularly the formation of "fishmouths" at fabric overlaps during construction. These are possible sources of leaks. Polyester fabrics, on the other hand, usually do not wrinkle either on absorption of hydrocarbons or absorption of moisture from the atmosphere. They remain more dimensionally stable than either polypropylene or nylon fabrics and are therefore often selected for roofing applications. Glass fabrics are probably the most dimensionally stable of all of the fabrics, but in most cases are unsuitable because of their nonabsorptivity, their extremely low extension to break, and their poor impact resistance.

Stiffness of fabrics during handling varies with the composition and the manner of fabric construction. Some projects require a soft, flexible fabric which conforms to irregular surfaces. This characteristic, called "drapability," is found in stitch-bonded and loosely woven fabrics. Other projects call for a stiff, "boardy" fabric. Spun-bonded and latex-treated fabrics tend to be stiff.

Wetting Out

Fabric construction has a great deal to do with how well a fabric wets out when sprayed with an elastomeric material. Coarse fabrics with large open spaces may absorb the sprayed-on material along the fibers but allow it to drip through the open spaces, resulting in a netlike structure which is of no use as a geomembrane. Other fabrics can be so thick and bulky that the sprayed-on elastomer cannot penetrate through to the back of the fabric and form a coating on the back. An ideal fabric wets out easily when sprayed with an elastomer, wicking horizontally and vertically to give complete saturation of the fabric, and retaining the elastomer so that it neither drips through nor runs off.

The number of fibers per centimeter and the diameter of the fibers determine to a large extent how the fabric will behave when sprayed. The diameter of the fibers (measured by the denier of each individual fiber) is one of the most important factors. Most of the commercial nonwoven fabrics are made of individual fibers about 3.0 denier (denier is the weight in grams of 9000 meters of fiber). In our experience, fabrics having a weight of 75-150 grams per square meter (3-6 ounces per square yard) are the most satisfactory. Fabrics below this weight usually give membranes of low tensile and tear strength. Fabrics above this weight usually are so thick that they are difficult to wet through and tend to give a nonuniform product.

III. SELECTION OF THE ELASTOMER

There are a number of types of elastomeric materials which could possibly be used for formation of a geomembrane in situ by spraying over a geotextile. Hot-applied elastomers, including rubberized asphalt, can be used. These have the disadvantage of requiring cumbersome heating, pumping, and spraying equipment in the field. Also, it is difficult to obtain a uniform application of hot-applied materials in cold weather or on a cold surface.

Cold-applied materials can be water-based emulsions or solvent-based coatings. These can be sprayed easily, but they are subject to runoff when applied heavily. They also have the disadvantage that they contain a large proportion of water or solvent, which may affect the fabric. In addition, they require a fairly long cure time before the liner goes into service.

The most practical type of cold-applied coating is a moisture-curable or two-component polymeric system. A chemical reaction takes place either by absorption of moisture from the air or by the two components reacting with each other. These materials can be catalyzed so that their setup time can be carefully controlled. They contain either no solvent or very little solvent, so that they comply with air pollution laws, and are economical because of their high solids content. Of the cold-applied, chemically reacting systems, silicones, polysulfides, and polyurethanes are among those which give elastomeric products. Epoxies and unsaturated polyesters also set up at ambient temperature, but they are usually too rigid to be used in this sort of application.

Physical properties desired in an elastomer for this purpose are a good combination of tensile strength, extension to break, and tear strength. The liquid-applied elastomer should have the ability to wet out a fabric completely and spread out in a lateral direction in order to provide a uniform, completely impermeable membrane but be viscous enough not to drain excessively from the fabric.

IV. PROPERTIES OF THE FABRIC-REINFORCED ELASTOMER

Geomembranes of this sort are usually considered to be fabric-reinforced elastomers, as opposed to coated fabrics. Coated fabrics often have exposed fabric on one side and a thin coating of elastomer, plastic, paint, or other material on the other side.

The physical properties of the finished fabric-reinforced elastomer are determined primarily by the fabric selected as the substrate and secondly by the elastomer as shown in Table I. It is not always possible to predict from the properties of the two components what the finished product will be like. For example, many fabrics have excellent tear strength uncoated. When one attempts to tear the fabric, the fibers can rearrange themselves so as to oppose the tearing force. In a coated fabric or fabric-reinforced elastomer, the individual fibers are held in place and their movement to reinforce each other is restricted. In such a case, much of the reinforcement is lost; the fibers are torn individually, and the tear strength is reduced.

Tensile strength of the composite structure is usually greater than that of the elastomer or the fabric itself. In this case, the two components tend to reinforce each other. The same is true in a Mullen burst test, which is, in effect, a two-dimensional tensile test.

Extension to break is usually determined by the properties of the fabric but in many instances can be greater than the extension of the fabric alone. In most cases, the extension of the fabric limits the total extension of the composite structure so that it does not even approach the extension of the elastomer. Elastomers often have extensions in the range of 500% or greater, whereas fabric-reinforced elastomers are usually limited to somewhere in the range of 50-150%.

V. IN SITU CONSTRUCTION OF GEOMEMBRANE BY SPRAYING

In choosing a geomembrane for a particular application, the overriding consideration is the function that the geomembrane is to serve. It cannot be emphasized enough that geomembranes are not load-bearing structures, so that the structure upon which the geomembrane is placed or installed must bear the load which the project is designed to take. If the substrate should give way, the geomembrane, no matter how strong, cannot be expected to support the load.

Site layout, excavation, and site preparation should be the same for a sprayed-in-place membrane as it is for the membrane constructed of preformed sheets. The equipment required for installing a sprayed-in-place membrane includes machines or personnel to handle rolls of fabric, scissors to cut the fabric (many fabrics cut more easily with scissors than knives), and a means to hold the fabric in place while it is being sprayed. On many jobs, the fabric has been held in place by staples up to 8.75 centimeters (3-1/2 inches) long. These are driven directly into the ground by an air-operated stapling machine. One disadvantage of this method of fastening, however, is that the staples may act as sites for stress concentration in case there is any shifting of the underlying soil or substrate or in case the load on top of the membrane shifts. This could result in tearing of the membrane. Another method of holding the fabric in place is merely to weight it down with sandbags, rocks, or pails.

During spraying of the geotextile, two techniques are used to assure uniformity of application. One is to mark off specific areas and then to use a predetermined volume of coating on each area. Another is to spray the fabric until a smooth, glossy sheen appears on the surface. With each selected combination of fabric and elastomer, this signals the fact that the fabric has been completely saturated and that a continuous layer of the membrane has accumulated on the surface. When this state is reached, the operator can be assured that the fabric is completely saturated and that he has a truly impermeable membrane.

Spray equipment is available from several manufacturers for any size job. For small jobs, a single-component sprayer which fits on top of a 5-gallon pail is convenient. The two components are mixed in a 5-gallon pail, the spray unit is placed on top of it, and spraying commences. Several buckets of material can be sprayed before the spray unit requires flushing with solvent. Hose length is usually limited to about 15 meters (50 feet) because of the possibility of mixed material setting up in the hose.

For large jobs, a two-component spray machine with up to 60 meters (200 feet) of hose is more practical. The proportioning and pumping unit can be mounted on a truck or small trailer and fed from drums or tanks on a truck. The two components pass through separate hoses to the spray gun, which has a built-in static mixer. With this type of equipment, cleaning is seldom needed.

As in pond construction with preformed sheets, the membrane should be trenched around the tops of the berms and held in place by backfilling and tamping the trenches. In the case of the sprayed-in-place membrane, the trenches can be backfilled immediately after the membrane has been sprayed. Once the fabric is saturated with the elastomer, it will retain it; and the elastomer will set up inside the fabric so that dumping fresh soil, rocks, or aggregate on the membrane will not harm it.

One of the principal advantages of sprayed-in-placed geomembranes is their seamless construction. However, careless joint construction can result in failures, just as careless seaming can result in failures in liners made from preformed sheets. Field joints are prepared by overlapping fabric strips about 15 centimeters (6 inches) and spraying elastomer first between the fabric layers and then on top of the overlap. Properly prepared joints are stronger than single layers. On large jobs which cannot be completed in one day, a 15-centimeter (6 inch) width of fabric should be left uncoated to serve as a connection when starting the following day's work. Where work will be delayed for more than a few days, any uncoated fabric must be protected from UV degradation by turning it under or otherwise covering it.

Rain can be a problem during construction of sprayed-in-place membranes. Urethanes foam if they are contaminated with water or if they are sprayed onto damp surfaces. Fabric must not be placed on a wet surface and must be sprayed with elastomer before it is contacted by rain or dew. Freshly sprayed elastomer is still subject to damage by rain up to about one-half to one hour, depending on the temperature and pot life of the elastomer. After this time, no damage results from the rain.

In order to provide protection from UV light, a cover of paint, sand, soil, or aggregate is often placed over the membrane. In the case of a sprayed-in-place membrane, the fabric absorbs and holds the elastomer so that the aggregate can be applied immediately. The aggregate will sink into the wet elastomer only as far as the top surface of the fabric, not harming the waterproof integrity of the structure and ensuring that the aggregate will stick firmly to the membrane.

VI. COMPLETED JOBS

A number of jobs using Chevron Industrial Membrane have been completed in the last several years. Applications have included ponds for irrigation water and for livestock. Others include decorative ponds for golf courses and garden apartments. On some of these projects, volcanic rocks have been placed around the edge of the ponds to camouflage the liner. For liners in contact with aqueous chemical wastes, the nature of the chemicals, their concentration, and the operating temperatures were considered in selection of the fabric. A series of major projects involved evaporation ponds for treated sewage. Some of these involved considerable areas, as much as 32 hectares (80 acres) in a single job. Figure 1 shows one of these jobs in progress. Sprayed-in-place liners have been placed not only over soil, but also over old concrete reservoirs which were badly cracked and leaking.

VII. CONCLUSIONS

We have described a method for construction of geomembranes by spraying an elastomer over a geotextile. An essentially seamless construction results in which overlaps are stronger than a single layer. Careful selection of the fabric and careful techniques in installation can eliminate large variations in thickness. Lateral wicking of the applied elastomer throughout the fabric results in complete saturation and water impermeability. Careful selection of the elastomer and the reinforcing fabric is also necessary to obtain the desired chemical resistance and physical properties. Careful attention during construction is required to avoid thin spots, runoff, and fishmouths. On large projects, two-component equipment is more efficient than single-component equipment. A cover of paint, sand, soil, or aggregate helps minimize UV degradation where required. Application of soil, sand, or aggregate can be made immediately following spraying without waiting for the elastomer to cure. Construction of geomembranes in situ has been accomplished on large projects involving millions of square feet. It can be a practical and economical means for construction of waterproof membranes.

FIGURE 1

Installation of Sprayed-in-Place Geomembrane

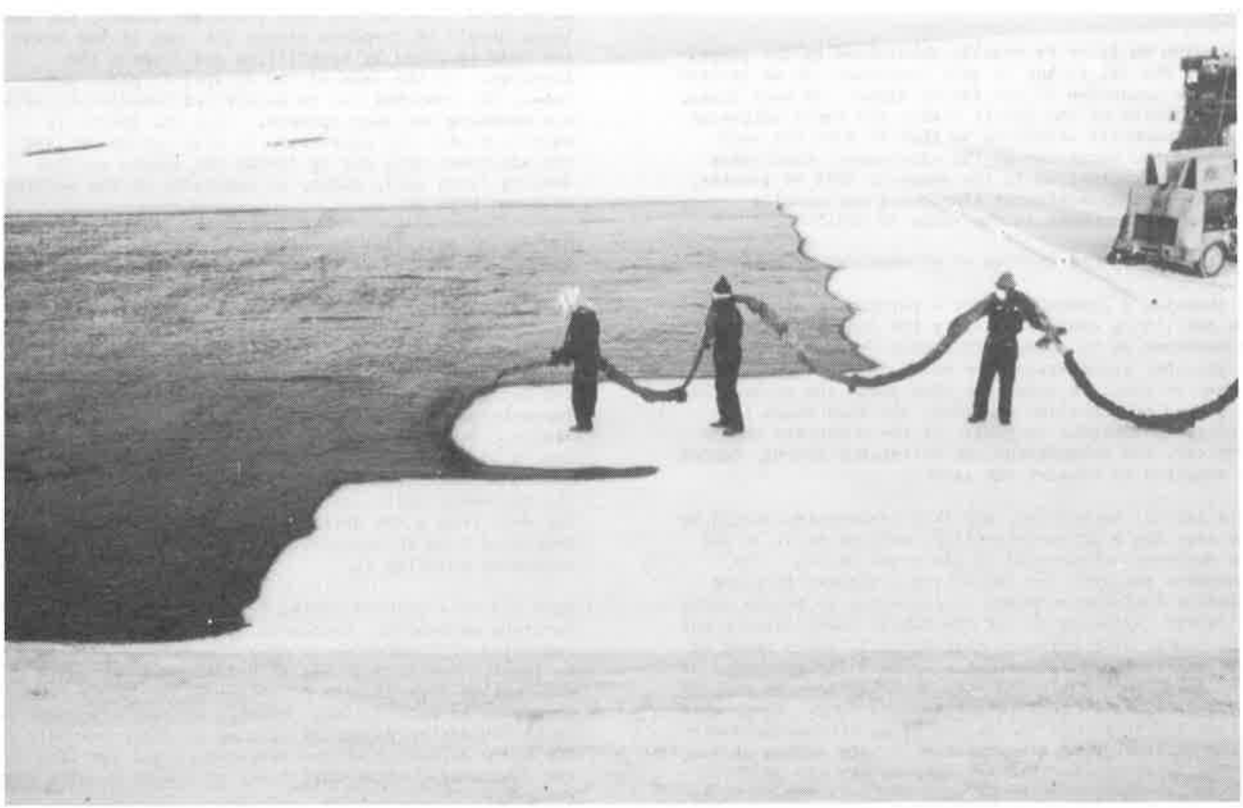

TABLE I

PHYSICAL PROPERTIES OF FABRIC-REINFORCED CHEVRON INDUSTRIAL MEMBRANE (CIM)

Fabric Name	Rufon E3N		Supac 5P		Reemay 2033		Tietex 375-18		Bamilex XP-113	
Manufacturer	Phillips		Phillips		du Pont		Tietex Fabrics		Bay Mills	
Material	Polyester Needle Punched, Heat Bonded		Polypropylene Needle Punched, Heat Bonded		Polyester Continuous Filament, Spun Bonded		Polyester Stitch-Bonded		Polyester Scrim 4 x 4 x 000 Den. Polyester Web	
Fabric Weight g/m^2 (Oz/Yd^2)	102 (3.0)		180 (5.3)		100 (2.95)		127 (3.75)		68 (2.0)	
Physical Properties ASTM D 751	Fabric Only	With 1.44 mm CIM	Fabric Only	With 2.0 mm CIM	Fabric Only	With 1.27 mm CIM	Fabric Only	With 1.52 mm CIM	Fabric Only	With 1.27 mm CIM
Breaking Load, kg (Lb) 2.54 cm (1 In.) Strips										
MD[1]	10.5	21.8	19.3	70.5	13.1	24.5	7.0	36.5	38.2	49.1
	(23)	(48)	(43)	(155)	(29)	(54)	(16)	(80)	(84)	(108)
TD[2]	8.2	-	26.9	60.0	7.0	21.5	20.7	36.6	37.3	45.5
	(18)	-	(59)	(132)	(16)	(47)	(46)	(81)	(82)	(100)
Extension to Break, %										
MD	36	74	52	112	33	69	58	49	37	47
TD	48		60	120	37	101	23	31	38	43
Split Tear Strength, kg (Lb) Force, 50 cm/Min.										
MD	3.5	2.5	-	3.2	1.9	2.0	4.2	3.3	-	5.2
	(7.7)	(5.5)	-	(7.0)	(4.2)	(4.5)	(9.3)	(7.3)	-	(11.5)
TD	4.2	-	-	2.5	1.9	2.5	4.5	3.7	-	3.4
	(9.3)	-	-	(5.5)	(4.2)	(5.5)	(9.9)	(8.1)	-	(7.5)
Mullen Burst kg/cm^2 (psi)	9.9	16.2	24.3	28.2+	-	10.7	8.9	20.2	-	16.8
	(140)	(230)	(345)	(400+)	-	(152)	(127)	(287)	-	(239)

[1]MD = Machine Direction

[2]TD = Transverse Direction

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Geotextiles Used With Geomembranes—Case Histories

Case histories are presented which illustrate three of the potential uses of geotextiles in conjunction with membrane linings: 1) slope protection, 2) drainage, 3) puncture resistance. First case involves a large water storage pond where a geotextile was placed over the subgrade on the slopes prior to installing the lining. A woven fabric was used for its high tensile strength and resistance to creep under loading. The geotextile isolates the lining from the subgrade reducing abrasion and potential migration of soil particles downslope. Second case utilizes a geotextile as the media for transporting liquid between two linings for the purpose of monitoring any leakage through the primary lining. In this case a nonwoven fabric was used because of its ability to transmit liquids within the plane of the fabric. Third case involves leach pads for a mining operation where a geotextile was placed over a secondary membrane lining to prevent puncture from a drainage course of rock placed on top. A thick nonwoven fabric was used because maximum puncture resistance was required. The installations are in service and are performing as designed.

This paper presents the case histories of three lining installations where geotextile materials were utilized to enhance the performance and durability of synthetic membrane linings (geomembranes). It is anticipated that the installations will have a longer system life and a reduction in maintenance through the use of geotextiles. These are important factors, especially where zero discharge is required and premature failure could result in high clean-up costs or irreversible damage. These case histories represent only three of the potential uses of geotextiles in lining system design and construction: Slope Protection, Drainage, and Puncture Resistance.

SLOPE PROTECTION

Exposed linings on pond slopes can sustain damage, and even fail in certain cases, due to buffeting from wind and wave action on the lining. A further complication of this condition is the migration of subgrade soil particles downslope as the lining is buffeted. When this action is allowed to continue at one elevation for a long period, a void in the subgrade somewhat above the water line develops with a corresponding bulge in the subgrade immediately at and above the water level. As the water level rises above this bulge, excessive stress is put on the lining and damage or failure may result. In extreme cases, this condition has been observed at multiple levels on pond slopes.

There are a number of ways to mitigate the effects of wind and wave action through proper design. The shape, size, and orientation of the pond are very important. Other measures that can decrease the effects of wind and waves include the use of a protective cover, weights placed at intervals, venting, and stabilization of the subgrade (which could include the addition of a binder such as asphalt). To this list add the use of a geotextile to isolate the lining from the subgrade. This use has a number of advantages including reducing abrasion between the lining and subgrade, preventing migration of soil particles downslope, improving capacity for venting beneath the lining, moderating subgrade preparation requirements and providing a clean surface over which field seams are constructed.

Large installations may require the use of a combination of preventive measures to mitigate and control the effects of wind and wave action. An example of such an installation is described below.

The project involved lining approximately 105,000 m^2 (1,130,000 ft^2) of the interior surface of water storage pond that receives water from a power plant. The pond is constructed of silty and sandy soil materials that were excavated from the pond area and compacted to form embankments along the perimeter of the pond. The internal embankment slopes are 3H:1V, and the depth is nominally 9 m (30 ft). Due to the pervious nature of the soils used to construct the pond, a lining system was added to eliminate leakage. The lining system was designed to be functional and yet cost effective.

The lining system developed for the pond consists of a 0.75 mm (30 mil) polyvinyl chloride (PVC) plastic lining on the bottom covered by 300 mm (12 in.) of protective soil. The slope is 0.90 mm (36 mil) reinforced chlorosulfonated polyethylene (Hypalon) rubber lining exposed to weather. The two different linings are joined by a factory fabricated seam located at the toe of slope. The PVC lining used on the pond bottom requires a soil cover to protect it from weather; whereas Hypalon used on the slopes eliminates the need for a soil cover, since it is weather resistant. The exposed membrane, however, is susceptible to damage from wind and wave action which must be taken into account in the design of such a system. The water level in the pond varies from full to empty during the year. Therefore, it was necessary for designers to consider the entire slope length in developing a solution to protect the lining on the slopes.

Because of the large size of the pond, wind and wave action were considered to be a major design consideration. A system of vents and weights composed of sand-filled Hypalon tubes form the basic protection system. The sand tubes are intended to help minimize buffeting of the lining. They are located at 14 m (45 ft) intervals and extend from the top of slope anchor trench to the toe of slope, approximately 30 m (100 ft) long. They are secured in the anchor trench and rest directly on the Hypalon lined slopes. An air/gas vent is located near the top of slope between adjacent sand tubes. The purpose of the vent is to release trapped air and gas from beneath the lining, and to help offset the uplift effect on the lining during windy conditions.

Because of the potential for high winds and resulting waves at the site, it was determined that protection in addition to weights and vents was required. To mitigate the migration of subgrade particles downslope, and to reduce abrasion of the lining, a geotextile was placed over the subgrade prior to the lining installation. Figure 1 shows the lining system using a geotextile for slope protection. A woven fabric was selected for this purpose. Woven fabrics characteristically have a higher tensile strength and are thinner than nonwoven fabrics. The high tensile strength of a woven fabric allows it to be used in applications where potential for creep under loading may occur. Steep slopes and vertical walls would be examples where the potential for creep exists. Creep is the gradual elongation of a material under loading. It is accomplished by a thinning of the material as elongation occurs, and can render a fabric ineffective under extreme conditions. In addition to high tensile strength, woven fabrics offer high tear strength, limited elongation and a range of equivalent sieve sizes.

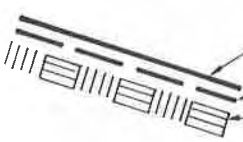

Figure 1. LINING SYSTEM USING GEOTEXTILE FOR SLOPE PROTECTION

The geotextile is anchored at the top of slope and extends downslope and onto the pond bottom for 2 meters (6 ft). Adjacent panels of geotextile are lapped 150 mm (6 in.). Sewing and seaming the geotextile panels together was not required.

Construction of the lining was completed on schedule with little or no delay caused by installation of the geotextile. Since placed into operation, the pond has shown no evidence of distress of the lining on the slopes.

DRAINAGE

Drainage beneath and between double linings is an important capability for a lining system, especially where leakage is critical and/or monitoring of leakage is required. The materials which have had the widest use as drainage courses in conjunction with membrane linings are sand and rock. They are hard, durable and provide adequate transmission of liquid for most purposes. There are certain limitations on the usage of sands and rock when placed in direct contact with a lining. The primary consideration is that sharp particles do not penetrate or damage the membrane. This requires select materials and careful control during placement.

The slopes of ponds and reservoirs also present stability problems for sand and certain gradations of rock. The steeper the slope, the greater the potential for stability problems to arise. It is also difficult to place and compact thin layers of sand and rock on slopes, and to obtain the desired in-place density.

Geotextiles of the nonwoven variety offer an alternate to the use of sand and rock. They have the capability of transmitting liquids and gases within the plane of the fabric. Placed between two membrane linings, a geotextile is an effective leak detection and monitoring media. An example of such an installation is the recent rehabilitation of a wastewater pond for a city.

The pond is one of three existing cells that total approximately 74,000 m^2 (800,000 ft^2) in area. They are used to hold treated effluent from a wastewater treatment plant. The ponds were originally clay lined with rock riprap protecting the slopes from erosion caused by wave action. The interior slopes are 3H:1V and the average depth of the ponds is about 2 m (6 ft). In the rehabilitation plan of the cell, monitoring of leakage from the pond was considered to be a major criteria for design. It was therefore necessary to develop a system that would include monitoring capabilities at the lowest cost. The system selected for the bottom and slopes includes a clay-soil subgrade sealed with an asphalt prime coat on its top surface as a secondary lining. A nonwoven geotextile was placed over the asphalt sealed surface. The primary lining consists of a 0.50 mm (20 mil) PVC membrane. On top of the PVC membrane a 300 mm (12 in.) protective soil cover was used on the pond bottom and slopes. A geotextile was also placed over the soil cover on the slopes followed by a layer of rock riprap. Figure 2 shows the lining system using a geotextile for drainage. The pond bottom gradually slopes to a sump which collects the effluent transmitted by the geotextile. Evacuation and monitoring of leachate are accomplished through an outlet pipe at the sump.

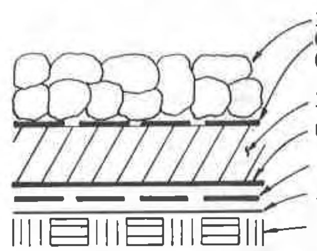

Figure 2. LINING SYSTEM USING GEOTEXTILE AS DRAINAGE LAYER

The advantages of using a geotextile as the drainage and monitoring media include the following:

1. A geotextile is relative easy to install compared to a sand or rock layer.
2. The problem of instability of some granular materials on slope is eliminated.
3. A geotextile provides a clean surface for seaming of the membrane lining.
4. A geotextile provides added puncture resistance to the membrane lining from beneath.
5. The problem of potential damage to the secondary lining from heavy equipment needed to place a drainage course of sand and rock is minimized.
6. Depending on the location, the cost of using a geotextile can be less expensive than sand or rock.

The rehabilitated pond is presently in service and is performing as designed.

PUNCTURE RESISTANCE

The use of leach pads in the mining industry has become a popular method to extract valuable metals from their native ores. In order to capture these metals as a leachate, the pads must be constructed to be free of leaks and to drain all leachate to a common collection point. If the leach pad leaks and does not control the leachate, the mining operation will not be cost effective

Generally, the ore is stacked high on the pad by heavy ore moving equipment. A solution is then applied which reacts with the ore and extracts the metal in various forms. Thus, the leach pad must also resist the traffic of heavy equipment as well as have a lining system which does not leak.

The primary lining used for five gold leach pads totaling approximately 28,500 m^2 (305,000 ft^2) consists of an exposed layer of dense asphalt concrete. The asphalt concrete is comprised of a mixture of aggregate and asphalt of proper structural strength to support the heavy ore moving equipment plus the weight of the ore. The asphalt concrete also has sufficient impermeability to convey drainage of the leachate to a common collection point.

Because of the value of the leachate, it is imperative that all be collected. Thus, a secondary lining of PVC membrane was used beneath the primary lining to trap and convey to a collection point any leakage through the primary lining. The secondary lining also serves as a safeguard to prevent contamination of the underlying soils. The primary and secondary linings are separated by a drainage course of rock. In order to provide protection to preserve the integrity of the membrane lining, a geotextile was placed between the drainage course material and membrane lining. Figure 3 shows the lining system using a thick nonwoven geotextile for puncture resistance. Construction of the leach pads were completed a few years ago and are serving their design purpose of capturing the leachate.

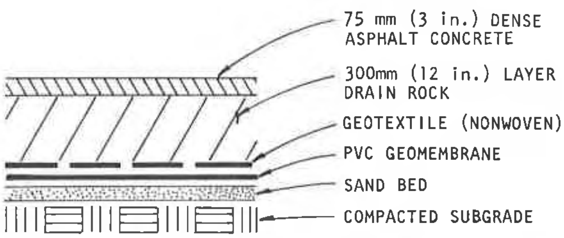

Figure 3. LINING SYSTEM USING GEOTEXTILE FOR PUNCTURE RESISTANCE

As requirments for containment of water, disposal of wastes, and control of leachates become increasingly stricter, the use of geotextiles will expand, and will certainly play an important role in enhancing the reliability and life expectancy of membrane lining systems as demonstrated by these three projects.

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Properties of Plastic Nets for Liquid and Gas Drainage Associated with Geomembranes

Synthetic drainage layers, particularly plastic nets, are increasingly used for liquid and gas drainage associated with geomembranes. This paper is intended to provide practical information on properties of plastic nets. The first part of the paper presents plastic nets, and reviews their hydraulic and mechanical properties and their durability. The second part of the paper presents an experimental study of the hydraulic transmissivity which governs the drainage capability of a plastic net. Test results show that the hydraulic transmissivity of plastic nets is significantly influenced by such parameters as: the type and number of nets; the hydraulic gradient; the applied normal stress; and the boundary conditions imposed by adjacent materials such as geomembranes or geotextiles. Practical recommendations are given to help designers benefit from the findings of this test program.

INTRODUCTION

Drainage of liquids and gases is often needed in geomembrane-lined facilities, as discussed in (1, 2). Traditionally, granular materials (sand, gravel) and pipes have been used, but these have drawbacks as discussed in (2). Synthetic drainage layers such as plastic nets (Fig. 1) have been successfully used in place of traditional drainage systems in a variety of applications (Figs. 2 and 3).

The purpose of the paper is to provide practical information regarding properties of plastic nets to be used for drainage associated with geomembranes. The first part of the paper presents an overview of plastic nets including a summary of their hydraulic and mechanical properties. The second part of the paper presents and discusses test results related to three plastic nets manufactured by Netlon Ltd. and designated herein as DN1, DN2 and DN3.

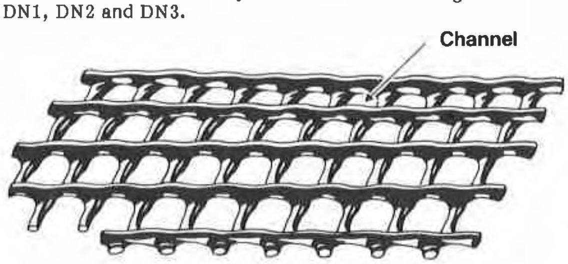


Fig. 1 Example of a plastic net comprised of two sets of parallel strands which form channels able to convey liquid and gas.

Fig. 2 Use of a plastic net drainage system with a soldier pile lagging/geomembrane system in the excavation for an underground garage.

Fig. 3 Use of a plastic net over a geomembrane for leachate collection in a hazardous solid waste landfill.

1 PRESENTATION OF PLASTIC NETS

1.1 Description

Nets (Fig. 1) consist of two sets of parallel extruded polymer strands intersecting at a constant angle (generally between 60° and 90°). Strands of one set lie on top of strands of the other set, and the two sets are bonded at the intersection. The two sets of strands create two sets of channels which can convey liquids or gases. This ability to transmit flow depends on the net geometry and properties as discussed in section 1.3, and as shown by the test results presented in section 2.3.

A large variety of nets are available. They differ by: size and shape of cross section of strands; depth of channels; opening size; and nature of the polymer. Nets discussed in this paper are made of medium density polyethylene (MDPE) and their geometry makes them suitable for drainage. Their strands are typically 1 to 3 mm (1/24 to 1/8 in.) in height and width, and their overall thickness is almost twice the strand height (channel depth is approximately equal to strand height). Opening size is typically from 5 to 10 mm (1/4 to 1/2 in.).

Not all nets are suitable for drainage. Nets with both sets of strands in the same plane do not have channels deep enough to allow significant flow of liquid or gas. Nets with large openings (typically larger than 10 mm, 1/2 in.) and deep channels could be used as a drainage medium between two rigid materials, such as concrete; but flexible materials, such as geotextiles and geomembranes, can penetrate the channels when subjected to soil or liquid pressure and block flow of liquid in the net.

Nets are available in rolls like geotextiles and geomembranes. Nets discussed in this paper are available in rolls 30 m (98 ft.) long and 1.63 m (64 in.) wide (nets DN1 and DN3) or 1.93 m (76 in.) wide (net DN2). The mass per unit area of these nets is approximately 0.8 kg/m^2 (24 oz/sq.yd) for net DN1, and 0.7 kg/m^2 (21 oz/sq.yd) for nets DN2 and DN3.

1.2 Association with Geotextiles and Geomembranes

Except in a few locations where they are in contact with coarse granular materials such as gravel, nets used for drainage are never put in direct contact with soil. Fine soil particles would rapidly penetrate net openings and hamper flow of liquids or gases. In most cases nets used for drainage are in contact with structural elements (concrete, sheet piles, lagging), geotextiles, or geomembranes. Nets, geotextiles and geomembranes can be supplied independently and installed successively. Composites comprising nets, geotextiles and/or geomembranes are also available or can be custom fabricated.

Geotextiles are used as filters between nets and materials such as soils or solid waste from which liquid or gas is drained into the net. The function of the filter is to prevent migration or soil or waste particles into the net, while allowing fluids to go from the soil or the waste into the net. Criteria for geotextile filter selection are referenced in (1). Selection should also consider the potential for the geotextile to penetrate the channels of the net as discussed in sections 2.2, 2.3 and 2.4.

1.3 Properties of Nets

Drainage Properties. Two related properties of nets govern their performance when they are used for drainage: transmissivity and conductivity.

The transmissivity of a net related to a given fluid may be calculated as follows:

$$\theta = (Q/B)/i \qquad (1)$$

where: θ = transmissivity of the net related to the considered fluid (m^2/s); Q = rate of discharge of the considered fluid (m^3/s); B = considered width of the net (m); Q/B = rate of discharge per unit width; and i = gradient (dimensionless).

If the flow is laminar and Darcy's equation applicable, the transmissivity, θ, depends only on the net, the fluid and the boundary conditions. In fact, as shown in section 2, the transmissivity of nets also depends upon the gradient.

The conductivity of a net related to a given fluid may be calculated as follows:

$$k = v/i = (Q/BT)/i = \theta/T \qquad (2)$$

where: k = conductivity (also called coefficient of permeability) of the net in the direction of the flow, for the considered fluid (m/s); v = fluid velocity (m/s); Q = rate of fluid discharge (m^3/s); B = considered width of the net (m); T = thickness of the net (m); and i = gradient (dimensionless).

To design a drainage system using a net, it is necessary to first evaluate the required rate of discharge which can be, for example, the expected rate of leakage through a geomembrane if the net is used to collect leakage. Then, Eq. 1 should be used to verify that the required discharge is compatible with the value of transmissivity given in section 2 for the considered net at the considered gradient. The important property of the net is, therefore, its transmissivity. In addition, if the net is used for leakage detection, the detection time depends on the velocity of the fluid in the net. As shown by Eq. 2, the important property of the net is then its conductivity.

Values of hydraulic (water) transmissivity at 20°C (68°F) of the nets DN1, DN2, and DN3 are given in Fig. 5 and thicknesses are given in Fig. 6, making it possible to determine their hydraulic conductivity using Eq. 2.

If the flow is laminar, values of transmissivity and conductivity of a given net, related to a fluid other than water, at any témperature, can be estimated from hydraulic transmissivity (Fig. 5) and conductivity at 20°C as follows:

$$\theta_f/\theta_{w20} = k_f/k_{w20} = \eta_{w20}\,\rho_f/(\eta_f\,\rho_{w20}) \qquad (3)$$

where: θ_f and k_f = transmissivity and conductivity of the considered net related to the considered fluid at a given temperature; θ_{w20} and k_{w20} = hydraulic transmissivity and hydraulic conductivity at 20°C; ρ_f and η_f = density (kg/m^3) and dynamic viscosity (kg/ms) of the considered fluid at the given temperature; and ρ_{w20} and η_{w20} = density (1000 kg/m^3) and dynamic viscosity (10^{-3} kg/ms) of water at 20°C.

Eq. 3 can also be used if the considered fluid is water to deduce hydraulic transmissivity and conductivity at any temperature from the values at 20°C. Eq. 3 should be used with judgement for non-laminar flow conditions.

Mechanical Properties. The most important mechanical property to consider when a net is used for drainage is its compressiblity. Other important mechanical properties are tensile strength and flexibility.

When a net is subjected to a normal stress, its thickness decreases in relation to the compressibility of the net. The conductivity of the net decreases as a result of the decrease in channel cross-sectional area, and the transmissivity of the net decreases as a result of the decrease in both conductivity and thickness, according to Eq. 2. Therefore, it is important that nets exhibit little compressibility and that the conductivity and transmissivity of nets be evaluated in tests with normal stresses representative of actual situations.

Constant rate of strain tensile tests have been conducted on the nets used in the tests discussed in section 2. The tensile strengths obtained in the machine direction (MD) and the cross-machine direction (XD) are: 4.8 kN/m (27.5 lb/in.) (MD) and 4.4 (25 lb/in.) (XD) for net DN1; 3.1 kN/m (18 lb/in.) (MD) and 2.0 (11.5 lb/in.) (XD) for net DN2; and 4.3 kN/m (24.5 lb/in.) (MD) and 3.4 kN/m (19.5 lb/in.) (XD) for net DN3. Since the mass per unit area of these nets if less than 1 kg/m^2, these nets can be hung unsupported over a height of more than 100m. In most projects, the factor of safety against tensile failure for a vertically hung net is, therefore, very large.

Plastic nets can withstand large elongations (40 to 100% at break in the tensile tests mentioned above). As a result they can deform without breaking when the supporting soil deforms due to settlement, instability, wave action, etc. Even after large deformations, a net remains continuous and keeps providing drainage, which might not be the case with traditional drainage materials. Also, when the supporting soil deforms, nets do not damage the geomembranes in contact, because these two materials have similar flexibilities.

Durability. Durability depends upon the type of polymer and the thickness of material. The durability of nets can be evaluated by comparison with the durability of other polymeric materials such as geotextiles and geomembranes for which there is 15 to 20 years of experience. Exposure to soil, weather and chemicals is discussed below.

Experience indicates that geotextiles (usually made of polypropylene or polyester) have a durability of several dozens of years when placed in most naturally occurring soil environments. Nets discussed in this paper are made of medium density polyethylene (MDPE) which is generally considered as equivalent to polypropylene or polyester from the point of view of chemical and biological resistance. Since, in addition, net strands are typically 30 times coarser than geotextile filaments, the durability of nets in the ground is expected to be equal to or greater than several dozen years.

Regarding outdoor exposure and chemicals (the latter being considered mostly for drainage at waste disposal facilities), the durability of MDPE nets can be considered equivalent to the durability of HDPE geomembranes because the thickness of net strands is of the same order as the thickness of geomembranes, and because MDPE has outdoor and chemical resistances similar to HDPE. The useful design life of HDPE geomembranes is known to be at least a dozen years in severe weather conditions, and HDPE geomembranes resist attack by a wide range of chemicals encountered in waste disposal facilities.

2 HYDRAULIC TRANSMISSIVITY TESTS

2.1 Test Description

Principle. Evaluation of hydraulic transmissivity of a drain system requires that a hydraulic gradient be established across a specimen and flow be measured as a function of time and hydraulic gradient. Normal stresses should be applied to the system to model the effects of overburden pressure. For representative results, net specimens should be large compared to net opening size.

Test Equipment. The transmissivity device shown in Fig. 4 accommodates specimens 30.5 cm by 30.5 cm (12 in. x 12 in.). The systems tested may vary from a single layer geotextile or net to 15 cm (6 in.) thick systems composed of drainage materials (such as nets), geotextiles, geomembranes and soil layers. Normal stresses from 5 to 500 kPa (100 to 10,000 psf) are applied by a stiff load plate.

Types of Tests. The transmissivity device is designed to perform either constant head or falling head tests. In constant head tests, the hydraulic gradient is kept constant by maintaining a constant water level in the inflow chamber while the water level in the outflow chamber is maintained constant by a rectangular weir. The transmissivity device is designed for constant head tests at hydraulic gradients of 0.5, 1.0, 1.5, 2.0 and 2.5. The falling head test is performed by first filling the inflow chamber to a prescribed level. The water level in the inflow chamber then decreases as water flows through the specimen. The change in water level in the inflow chamber is measured as a function of time. The quantity of flow for each time increment is calculated using the change in water level. The hydraulic gradient for each time increment is the average difference between the inflow and outflow water levels divided by the length of the specimen. During a typical falling head test, the hydraulic gradient varies from about 2.5 down to 0.1.

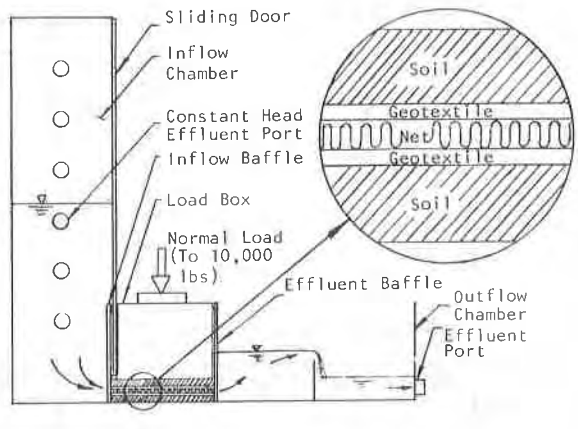


Fig. 4 Transmissivity device with detail showing net specimen and boundary conditions for tests 8 and 9.

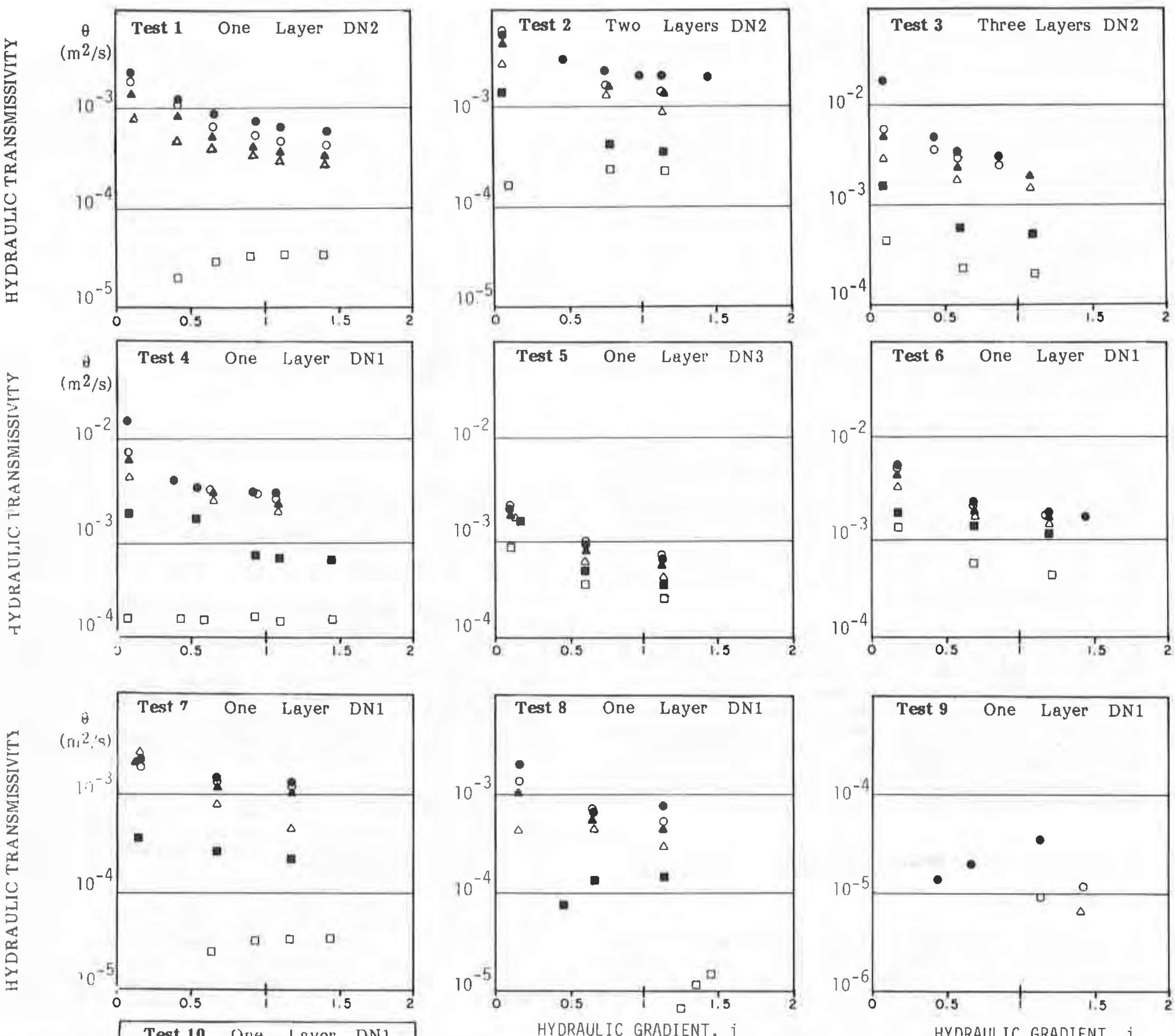
Preparation of Net Specimens and Boundary Layers. The net specimens and the geotextile or geomembrane boundary layers are trimmed. Soil layers, if any, are placed in the test chamber and compacted to the specified density. The system edges parallel to the direction of flow are sealed to the test chamber using a small amount of silicon gel which inhibits flow along the edges of the system. When geotextiles are used, they are soaked for 30 minutes in de-aired water prior to placement in the test chamber; however, tap water is used for the tests.

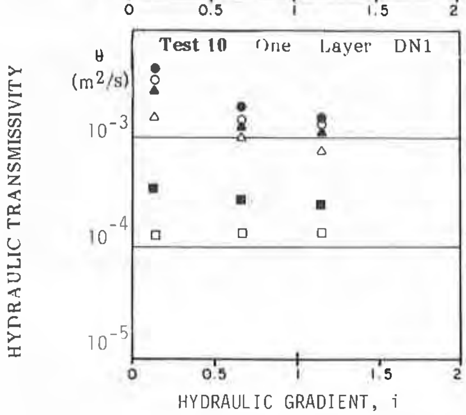
Test Methodology. Once the system to be tested is in place, the load plate is lowered onto the system. The edges of the load plate are sealed with a small amount of silicon gel and a first load of 10 kPa is applied. This seating load is necessary to prevent flow above and below the specimen. Before the test begins, water is allowed to flow through the test specimen and over the weir to saturate the sample and establish a constant water level in the outflow chamber. After a test at a given normal stress, the normal stress is increased and the process is repeated. Tests are typically performed at normal stresses of 10, 50, 100, 200, 350 and 500 kPa (200 to 10,000 psf). After completion of the test at 500 kPa, the net specimen is discarded.

Data Measurement and Interpretation. Zero readings are obtained for temperature, system height and system deflection after the 10 kPa load is applied. Constant or falling head tests may then be performed, but only falling head test results are presented herein. The change in level of water in the inflow chamber is measured as a function of time. Two tests are performed at each normal stress (except 500 kPa) to assess the reproducibility of data. Water temperature and plate deflection are measured at the beginning and end of each test at each normal stress. The hydraulic transmissivity is calculated using Eq. 1. An equivalent 20°C hydraulic transmissivity is then calculated using Eq. 3, although the flow is not laminar.

2.2 Discussion of Tests

Tests were performed on the systems summarized in Fig. 5. Tests carried out on the steel plate/plastic net/steel plate systems provide reference values of hydraulic transmissivity. These tests can be used to evaluate the influence of various parameters, including: number of layers of net; hydraulic gradient; normal stress; and creep. Tests carried out on systems incorporating geotextiles, geomembranes, and/or soil layers are used to evaluate the influence of boundary layers.





Test No.	Lower Boundary	Upper Boundary
1-5	Steel Plate	Steel Plate
6	Steel Plate	Heatbonded Geotextile/Clay
7	Steel Plate	Needlepunched Geotextile/Clay
8	Clay/Heatbonded Geotextile	Heatbonded Geotextile/Clay
9	Clay/Needlepunched Geotextile	Needlepunched Geotextile/Clay
10	Clay/Unreinforced Geomembrane	Unreinforced Geomembrane/Clay

Geotextiles used as boundary layers:
(i) Heatbonded nonwoven polypropylene, Mirafi 140 N, manufactured by Mirafi, with a mass per unit area of 140 g/m^2 (4 oz/sq.yd.);
(ii) Needlepunched nonwoven polypropylene, Fibretex 400, manufactured by Crown Zellerbach with a mass per unit area of 400 g/m^2 (12 oz/sq.yd.).
Geomembrane used as a boundary layer:
Unreinforced 0.5 mm (20 mil) thick PVC.

Normal Stress	kPa	(psf)
●	10	200
○	50	1,000
▲	100	2,000
△	200	4,000
■	350	7,000
□	500	10,000

Fig. 5 Results of transmissivity tests at 20°C on nets DN1, DN2 and DN3 performed under a range of normal stresses with various boundary conditions.

Tests 1 through 5 were carried out with steel plate/plastic net/steel plate systems. The purpose of tests 1 through 3 was to establish the relationship between hydraulic transmissivity and number of layers of net. The purpose of tests 1, 4 and 5 was to compare three nets of different geometry and compressibility.

In the field, when geotextiles are used to prevent intrusion of soil or waste into the net openings, the soil or waste pressure may cause the geotextile to penetrate the net openings, thus reducing the flow capacity of the system. When plastic nets are associated with geomembranes and are subjected to normal stresses caused by waste or liquid impounded in the geomembrane-lined facility, net openings may be partially penetrated by the geomembrane, thus reducing the flow capacity of the system.

Tests 6 through 10 were carried out to evaluate the reduction in hydraulic transmissivity associated with geotextile or geomembrane penetration. Two types of geotextiles and one type of geomembrane were used in the tests and their properties are given in Fig. 5. The two geotextiles have been selected to represent two extreme situations common in the field: the heatbonded nonwoven is thin and stiff, and is not expected to significantly penetrate net channels; the needlepunched nonwoven is thick and compressible, and is expected to penetrate net channels. A thin unreinforced (therefore highly deformable) geomembrane has been selected to represent the worst case of channel penetration by a geomembrane. A 25 mm (1 in.) thick layer of saturated clay (sand-bentonite mixture) has been placed between the steel load plate and the geotextile or geomembrane boundary layer to better simulate the condition where the geotextile or the geomembrane can be pushed into net channels by soil or liquid pressure.

2.3 Discussion of Test Results

The equivalent hydraulic transmissivity at 20°C is plotted as a function of hydraulic gradient for tests 1 through 10 in Fig. 5. The results indicate that hydraulic transmissivity depends upon the hydraulic gradient, normal stress, type of net, number of net layers, and boundary conditions. The influences of these factors are described below.

Influence of Hydraulic Gradient. The tests have been conducted with hydraulic gradients between 0.1 and 1.5. Most tests presented in Fig. 5 show that the hydraulic transmissivity decreases with increasing hydraulic gradient, which indicates transient or turbulent flow. However, in a few tests the hydraulic transmissivity does not vary with hydraulic gradient, which indicates laminar flow. This happens when the hydraulic transmissivity is less than about 2×10^{-4} m²/s, which is the case in the tests where flow is restricted by compression of the net or by penetration of the boundary layers into the net channels, these being: (i) tests 1, 2 and 4, conducted with the more compressible nets DN2 and DN1, under a normal stress of 500 kPa (10,000 psf); tests with partial penetration of the boundary layer (tests 7, 8 and 10 for a normal stress of 350 kPa or more); and tests with complete penetration of the boundary layer (test 9).

These experimental findings can be explained using the Reynolds number, Re, related to hydraulic flow in a plastic net, given by:

$$Re = v\rho d/\eta = \theta\rho i d/(\eta T) \quad (4)$$

where: v = velocity of water (m/s); θ = hydraulic transmissivity (m²/s); ρ = density of water (kg/m³); i = hydraulic gradient (dimensionless); d = diameter of net channel (m); η = dynamic viscosity of water (kg/ms); and, T = thickness of the net (m). Reynolds numbers, calculated for the nets tested and for hydraulic gradients of the order of 1, are smaller than the limiting values for laminar flow (Re = 2000) when the hydraulic transmissivity is less than approximately 10^{-3} m²/s. This is fairly consistent with the experimental findings mentioned above.

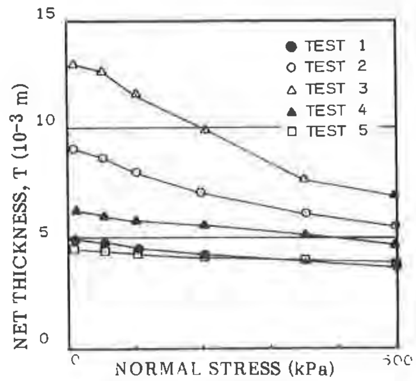


Fig. 6. Thickness of nets versus normal stress, from measurements taken during tests 1 through 5 (see Fig. 5.)

Influence of Normal Stress. In all tests, hydraulic transmissivity decreases with increasing normal stress. The hydraulic transmissivity of DN1 and DN2 is about 2 to 4 times greater at 10 kPa than at 200 kPa (4,000 psf) and about 10 to 100 times greater at 10 kPa than at 500 kPa (10,000 psf). The hydraulic transmissivity of DN3 decreases by a factor of about 2 as the normal stress is increased from 10 kPa to 500 kPa.

The relationship between thickness and normal stress for each of the nets is shown in Fig. 6. A comparison of tests 1, 2 and 3 shows that the relative thicknesses of one, two and three layers of net DN2 are, respectively: 1, 1.9 and 2.6 under a 10 kPa (200 psf) normal stress; and 1, 1.5 and 1.9 under a 500 kPa (10,000 psf) normal stress.

Influence of Type of Net. The results of hydraulic transmissivity tests on single layers of DN1, DN2 and DN3 between two steel plates are shown in Fig. 5 (tests 1, 4 and 5). At normal stresses less than 350 kPa (7,000 psf), hydraulic transmissivities of DN2 and DN3 are about equal, and approximately 5 to 10 times less than that of DN1. However, at 500 kPa (10,000 psf), due to its lesser compressibility, DN3 has a hydraulic transmissivity about twice that of DN1 and 10 times that of DN2.

Influence of Number of Net Layers. Tests 1, 2 and 3 were conducted with 1, 2 and 3 layers of net DN2, respectively. In each case, steel plates were placed directly above and below the net. The results show that the relative hydraulic transmissivity at a given gradient and a given normal stress of one, two and three layers of net DN2 are, respectively, 1, about 2 to 2.5, and about 3 to 4. Thus, even though the thickness of N nets is less than N times the thickness of one net, as mentioned above, the transmissivity of N nets is more than N times the tranmissivity of one net. This is due to the non-proportionality between flow rate and channel size.

Influence of Boundary Conditions. Comparison of the results of tests 4 and 6 in Fig. 5, for normal stresses of 350 kPa or less, indicates that hydraulic transmissivity of net DN1 is decreased by a factor of about 2 to 3 when a heatbonded nonwoven geotextile and clay are placed at one boundary. The higher transmissivity obtained at 500 kPa with a heatbonded geotextile/clay boundary on one side of the net (test 6), compared to the test with a net between two steel plates, can be explained either by a lesser compressibility of the particular net specimen used in test 6, or by geotextile/net friction and interlocking modifying the tilting, bending, and/or compression characteristics of the net strands. The results of tests 4 and 8 indicate that hydraulic transmissivity is decreased by a factor of 5 to 10 when the heatbonded nonwoven geotextile and clay are placed at both boundaries.

Comparison of tests 4, 7 and 9 (Fig. 5) indicates that the hydraulic transmissivity decreases by a factor of about 5 for a needlepunched geotextile and clay at one boundary, and by about 100 to 1000 for a needlepunched geotextile and clay at both boundaries. For the clay /needlepunched /DN1 /needlepunched /clay system, the system hydraulic transmissivity is of the order of 10^{-5} m^2/s, independent of hydraulic gradient. The 10^{-5} m^2/s value is of the same order as the hydraulic transmissivity of two layers of needlepunched geotextile subjected to relatively low normal stress (less than about 50 kPa, 1000 psf). It appears that the thick needlepunched material penetrates the channels of the net, causing the flow to be conveyed by the geotextile rather than by open channels. The strands of the net being stiffer than the geotextile accept most of the applied normal stress, resulting in a series of relatively unstressed geotextile strip drains in the channels of the net.

Test 10 (Fig. 5) was conducted with a 0.5mm (20 mil) thick unreinforced PVC geomembrane and clay, above and below a single layer of DN1 net. At pressures from 10 to 350 kPa (200 to 7,000 psf), the hydraulic transmissivity of the system is approximately two to three times less than that of the steel plate/DN1/steel plate system. At 500 kPa (10,000 psf), the hydraulic transmissivities of the two systems are about equal.

In a previous study using a different hydraulic transmissivity test device, tests were carried out on two DN1 systems: (i) lucite plate/DN1/lucite plate; and (ii) lucite plate/layer of coarse sand/0.9mm (36 mil) thick reinforced Hypalon geomembrane/lucite plate. Falling head tests were performed at a normal stress of 15 kPa (300 psf). The two systems gave similar measured values of hydraulic transmissivity, indicating minimal penetration of the reinforced Hypalon geomembrane into the channels of the net.

The geomembrane, geotextile and net specimens were examined for evidence of damage at the completion of each test. No holes, ruptures, tears or permanent indentations were left in the geotextiles or geomembranes, even after the tests performed at 500 kPa (10,000 psf) for durations of up to 8 hours. The compression of each net was measured at each load, as the loads were increased, and at the end of each series of tests when the loads were removed. These measurements indicate that the nets return to 90 to 95% of their original thickness after a single loading/unloading cycle to 500 kPa (10,000 psf). Examination of the nets after the tests appears to indicate that the individual strands bend over and flatten slightly with increasing load. As the load is removed, the strands return to their original position.

2.4 Conclusions

Hydraulic Transmissivity of Plastic Nets

- o The hydraulic transmissivity of synthetic drain materials may be measured using the test device shown in Fig. 4. This device may be used in a constant head or falling head mode. It permits modelling of field conditions including the effects of normal stress and boundary conditions.
- o The hydraulic transmissivities of the plastic nets used in the testing program described in this paper range from 10^{-5} m^2/s to 10^{-2} m^2/s.
- o The hydraulic transmissivity of plastic nets depends on temperature, hydraulic gradient, normal stress, net type, number of nets and boundary conditions.
- o The hydraulic transmissivities of the plastic nets tested were found to decrease with increasing hydraulic gradient and increasing normal stress.
- o The effect of normal stress varies for different types of nets. At normal stresses less than about 350 kPa (7,000 psf), the hydraulic transmissivity of DN1 is greater than that of DN2 or DN3. Above 350 kPa, the hydraulic transmissivity of DN3, due to its lesser compressibility, is greater than that of DN1 and DN2.
- o Under rigid boundary test conditions, the hydraulic transmissivities of multiple layers of plastic nets are approximately additive.
- o The presence of geotextiles in contact with plastic nets decreases the hydraulic transmissivity of the nets due to penetration of the geotextile into the channels of the net. This effect is greater for needlepunched geotextiles than for heatbonded geotextiles.
- o The presence of a thin unreinforced (typically 0.5 mm (20 mil) thick PVC) geomembrane on both sides of a plastic net may decrease the hydraulic transmissivity of the net by about 50%.
- o The presence of a reinforced geomembrane such as Hypalon in contact with a net has a negligible influence on the hydraulic transmissivity of the net. By extrapolation it may be concluded that for thick, stiff, geomembranes (such as 1 mm (40 mil) or thicker HDPE), the reduction in net hydraulic transmissivity due to the presence of the geomembrane is negligible.
- o Plastic nets have been tested with geotextiles and geomembranes at normal stresses up to 500 kPa (10,000 psf) for up to 8 hours without visible physical damage to any of the materials.

Design of Drainage Systems with Plastic Nets

- o The hydraulic transmissivities of DN1, DN2, and DN3 are sufficiently large that these materials may be considered for a variety of geomembrane associated drainage applications. Other types of plastic nets should be tested prior to use as drain materials.
- o When designing plastic net drainage systems, several important physical parameters must be considered including temperature, hydraulic gradient, overburden stress, and boundary conditions. Eqs. 2 and 3 and Figs. 5 and 6 can be used to design the drainage systems.
- o The hydraulic transmissivity of a plastic net in contact with a smooth stiff surface such as wood, concrete, steel, reinforced or stiff unreinforced geomembrane, may be evaluated from the test results with steel plate boundary conditions.
- o Where soil conditions permit, geotextiles used in contact with plastic nets should be heatbonded nonwoven or monofilament woven.
- o When a needlepunched geotextile is required in contact with a net and when the overburden stresses are large, the net drain can be designed making the conservative assumption that the net channel (one-half net thickness) in contact with the needlepunched geotextile has zero flow capacity. Therefore, if one layer of net is used between a needlepunched geotextile and a smooth stiff surface, the transmissivity measured between two rigid plates should be divided by 5 (see section 2.3). If a needlepunched geotextile is required on both sides of a net drain, and if the flow quantity is large, at least two layers of nets should be used.
- o One layer of plastic net may be used with needlepunched geotextiles, even under high overburden pressures, if the flow quantity is small. The hydraulic transmissivity of this system may be considered to be that of the geotextile subjected to low overburden stress, with the plastic net preventing compression of those portions of the geotextile within the net channels.

ACKNOWLEDGEMENTS

The authors are indebted to A. J. Hoyt for testing, and S. Hartmaier and N. Stuart for assistance in the preparation of the paper.

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Geomembrane-Geotextile-Geogrid Composites in a Railroad Application

The authors present the design considerations for a railroad project which combines the use of geomembranes, geotextiles and geogrids. The project is located in Boston and is being constructed by the National Railroad Passenger Corporation (AMTRAK) under the auspices of the Federal Railroad Administration's Northeast Corridor Improvement Project (NECIP).

The site considerations include soft soils, high water tables, limited right of way, and a severe track modulus interface, i.e., a rapid transition from a soft to a hard subgrade. The authors describe the design rationale which was developed in order to deal with all of these problem conditions. The final solution combines geomembranes, geotextiles and geogrids in complementary support of each other. The design is believed to be unique in the railroad industry and probably has broad civil engineering applications.

1.0 INTRODUCTION

The railroad industry has traditionally been forced to contend with some of the most difficult conditions encountered by geotechnical engineers. The necessity for low grades (train speeds are drastically affected by grades greater that 2%) has always resulted in track being constructed over low-lying areas. Accordingly, the subgrade soils tend to be soft and wet; and, when combined with the dynamic loading which occurs during train passage, these subgrades are a continual geotechnical problem. Furthermore, train loads in North America have increased dramatically in the last quarter century, and these loads have caused new problems in many miles of formerly stable track. This latter problem affects AMTRAK, a passenger line, as well as freight lines because very little track in North America is dedicated to passenger use, i.e., almost all passenger track is also used by the much heavier freight haulers.

Because the frequency and weight of trains is likely to increase even further, railroad geotechnical engineers have welcomed the introduction of geosynthetic materials which have the ability to modify soil properties. Geomembranes, geotextiles, geogrids, etc., can reinforce, drain, separate, filter, and even isolate soils (isolation is defined here as hydraulic, as well as mechanical, separation of soils). These materials thereby give the geotechnical engineer the ability to design and repair tracks constructed on otherwise unsuitable subgrades.

This paper deals with one particularly difficult example of geosynthetic applications in railroads. The project is located in Boston, Massachusetts, USA, and is being constructed by the National Railroad Passenger Corporation (AMTRAK) as part of the Federal Railroad Administration's Northeast Corridor Improvement Project (NECIP). The entire project is problematical and one location includes a transition from underground to surface track where the subgrade is a very weak saturated organic silty clay mixed and layered with other poor materials. Additionally, the water table is higher than the proposed track elevation, and the water table cannot be lowered outside of the right of way.

The subject of this paper is the design rationale for using geomembranes, geotextiles and geogrids to deal with this problem site.

2.0 SITE CONSIDERATIONS

Trains in the project area (Mile Post Ma.228.01 to Ma.228.60) have been limited to slow speeds for years due to the lack of trackbed stability. In spite of a continual maintenance effort by AMTRAK's Boston Division, the tracks would not hold line, grade or cross level, the ballast became contaminated with fines, and pumping was evident at every rail joint.

The reasons for the long history of poor track in this location are both drainage and subgrade related. The tracks are located in a "boat" section, i.e., below the phreatic surface, for almost the entire distance and surface drainage resulting from rainfall and melting snow periodically inundate the trackbed. The very high water table then makes percolation negligible. The water table is also subject to tidal action and high water rises to within 0.3 m of the bottom of tie. The subgrade material is poor as evidenced by maximal track deflections of 0.03 m. The first layer of substrate is 0 - 1.8 m of unconsolidated fill material comprised of gravel and ballast mixed with cinders. The second layer consists of 0.6 - 4.8 m of organic peat, silt, gray-brown fine sand and silty clays. The third layer consists of up to 15 m of gray-green-yellow silty clay. In localized areas, the bearing capacity is as low as 1.4 - 3.4 kPa (CBR = .08 - .15).

The importance of the above track phreatic surface cannot be over emphasized. As discussed earlier, the surrounding water table is higher than the water table in the trackbed area and the resulting head causes a serious threat of piping in the soils beneath the track. This differential head problem cannot be mitigated because a Boston, Massachusetts building and design regulation prohibits the lowering of a water table by an adjoining construction project. This regulation is, of course, intended to protect the stability of neighboring structures, i.e., to prevent alteration of the assumptions and parameters used for foundation design.

To summarize, the designers were required to deal with high surface water runoff with high ground water and poor percolation, poor subsoil conditions, and the inability to lower the water table in a conventional manner.

3.0 DESIGN CONSIDERATIONS

The initial design proposed to drive sheeting on both sides of the track inside of the existing side walls (see Fig. 1). This sheeting would have cut off ground water migration from outside the containment walls and ensured adjoining structures of the original design water table. The second step was to lower the water table within the sheeted area with well points or other accepted construction practices and install a 1 m thick concrete slab between the underground sheeting walls. This concrete slab, along with end cut off walls, would serve the dual functions of supporting the newly planned conventional track structure and separating the ground water below from the surface drainage above. The surface drainage was then to be handled with a conventional drainage system of underdrains, catch basins and ballast drains. This system, although completely workable and fulfilling all design requirements, was very expensive and not accepted.

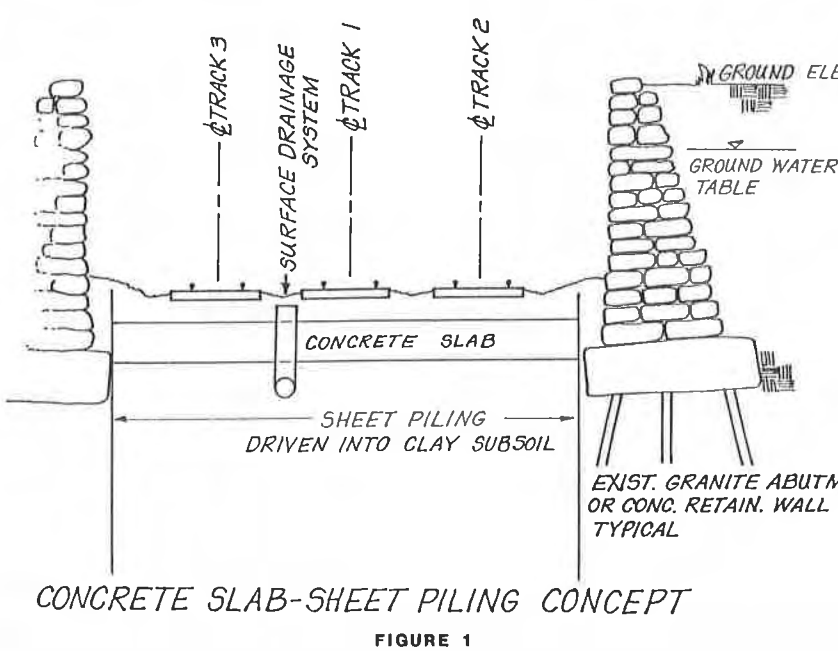
In March of 1982, the Federal Railroad Administration asked AMTRAK's Engineering Office in Philadelphia to re-examine the existing plans in an effort to extensively reduce costs. The FRA was willing to forego some minor design parameters if a considerable cost reduction (in the area of 5 to 6 million dollars) could be achieved. This re-design was carried out by the NECIP Engineering - Track and Structures staff. It was their assigned task to propose a more cost effective design which would still solve the major engineering problems and fulfill the design parameters.

To solve the high water table and water migration problems, AMTRAK proposed to line the entire trackbed area with a very low permeability geomembrane (see Fig. 2). This geomembrane was to be turned up on the sides to a level above the track water table thereby isolating the track structure. The particular geomembrane selected for the conceptual design was a 0.9 mm thick, 10 x 10 scrim-reinforced CSPE product. This geomembrane has long been used as a pond liner but has never before been used in this type of railroad application.

This railroad application is, in several aspects, very different from pond liners. Specifically, in the railroad application, seams and other potential leaks are not considered critical since the ballast above is a natural drain. On the other hand, the structural stability of the geomembrane is very critical due to the potentially high dynamic loads. To protect the geomembrane from excessive damage from ballast above or other sharp objects below, the design proposed two layers of a geotextile, one above and one below the geomembrane. It should be noted that the original concept by AMTRAK proposed that the geomembrane be installed on a 0.15 m sand bed layer but, after consideration of the extreme difficulty of spreading a sand layer by heavy equipment on a very poor, continually wet soil, the sand layer was replaced by the second layer of geotextile.

The geotextile selected is a 540 g/m^2 ultraviolet light stabilized, non-woven, needlepunched continuous filament or staple fiber material. This geotextile serves three purposes: first, as it does in other trackbed installations, as a filter, drain, and separator in the track bed; secondly, it protects the geomembrane from damage both from above and below; and, lastly, the geotextile below the geomembrane serves as a transmissivity medium for the escape of methane and other gases produced naturally by the deterioration of organic soils. The fabric above the geomembrane also serves as a transmissivity medium to carry surface water to a conventional drainage system of perforated underdrains.

The last engineering design hurdle involved the geogrid. At one end of the site, the trackbed continues onto a concrete slab supported trackbed. This creates a severe modulus interface, i.e., a rapid transition from a stiff to a soft subgrade. Such "hard spots" cause



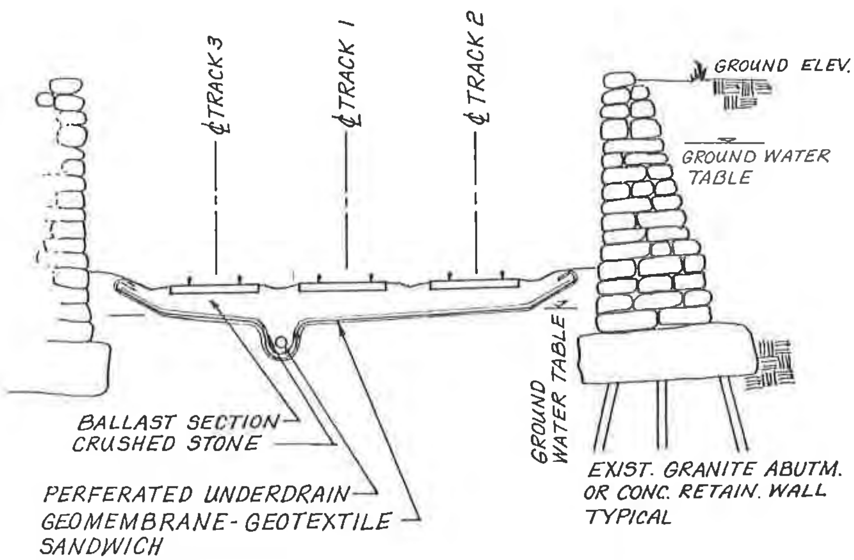
CONCRETE SLAB-SHEET PILING CONCEPT

FIGURE 1

continual maintenance problems and affect the smoothness of the train ride. This problem, as well as other design details, was addressed in the original design through the inclusion of a concrete transition slab. This treatment is similar to any approach slab to a bridge or other slab track area. However, AMTRAK wanted to soften the impact even more and thereby increase the ride quality for its passengers. The resulting solution is a gradual modulus change through the use of a geogrid system (see Fig. 3). Immediately under the transition slab, and extending at least 15 m are two layers of high modulus geogrid. The next 15 m is underlain by a single layer of this same geogrid, and the last 15 m uses a lighter weight lower modulus geogrid. It is important that each transition layer is at least 15 m long, because experience has shown that modulus transition systems must exceed one car length to be effective. The grids selected were (1) a preferred orientation bi-directional polypropylene grid with a rapid response 2% secant tensile stiffness of 500 kN/m, and (2) a preferred orientation bi-directional polypropylene grid with a rapid response 2% secant tensile stiffness of 300 kN/m.

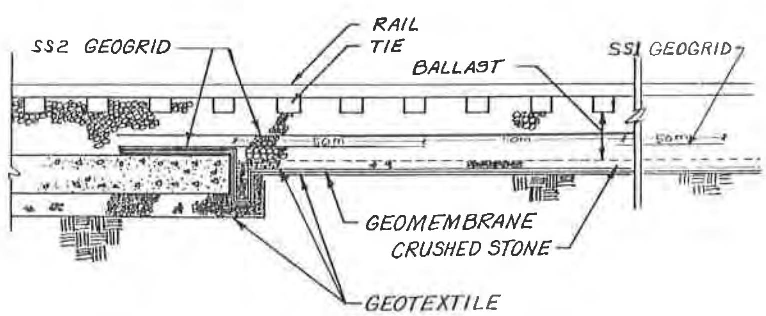
4.0 SUMMARY

There can be little doubt that this new discipline of geosynthetics is having a far reaching impact on geotechnical engineering in general and railroad track



GEOMEMBRANE - GEOTEXTILE CONCEPT

FIGURE 2



BALLAST-CONCRETE SLAB TRANSITION

FIGURE 3

design in particular. These two new uses, one a relatively new product (the geogrid), and the other a new concept for an established product (the geomembrane) will be monitored closely by AMTRAK Engineering. Construction of the project should be underway by summer of 1984 and progress will be reported at the Denver conference. Also, it is the authors' intent to report any quantifiable long term results in a future paper.

5.0 ACKNOWLEDGEMENTS

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Use of Asphaltic Geomembrane for Protecting Railway Subgrade

The National French Railway Company (S.N.C.F.) has been using for 12 years a reinforced Asphaltic Geomembrane for protecting railway subgrades ; sheltering a formation from rainwater increases its bearing capacity and it becomes then possible to decrease the thickness of foundation layers. Used on formations containing clayey materials, the geomembrane prevents the clay from rising into the ballast, thus saving the cost of further maintenance works ; tested on the "Vibrogir" Equipment this geomembrane resists the punching effect of ballast stones. It can therefore be laid directly on the formation and covered with ballast without protective layers; this technique therefore saves the cost of a deep cut below formation, followed by placing layers of new selected material. Moreover, this geomembrane can be placed on the occasion of ballast renewal works, thus saving the cost of preliminary protection works. Additionnally, no extra traffic interruption is required for placing protection layers.

I - EXPLANATION OF THE PROBLEM

1.1.- Description of the underlays Terminology

On open tracks, the foundation of the ballast track is as described schematically on figure 1.

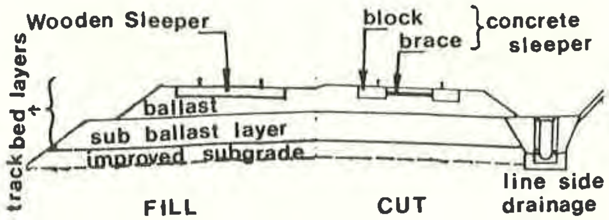


Figure 1 : typical cross-section

The track bed layers by their nature and thickness guarantee a good behavior of the railway track. They incorporate a layer of ballast and a single-layer or multi-layer sub ballast layer.

The thickness of the track bed layers depends on :
. the characteristics of the ground and of the subgrade
. the climatic conditions of the site,
. the characteristics of the traffic,
. the characteristics of the track panel.

1.2.- Conventional structures

The S.N.C.F. specifications concerning the track bed layers are schematized on figure 2.

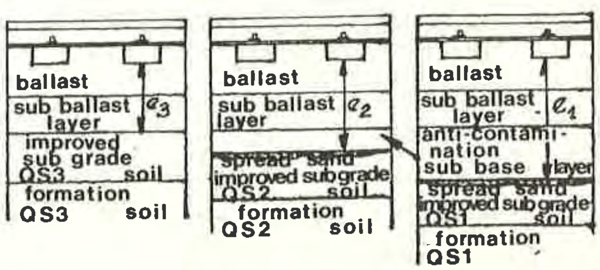


Figure 2 : conventional structures (subgrade of the same nature as the supporting ground).

They are conventional structures made of unstabilized aggregates, whose thickness depends on the grade of the supporting ground, in accordance with data sheet 719R of the International Union of Railways (Earth Works and Track bed Construction for Railway Lines). See figure in Appendix 1. There are :

. very poor soils, grade QS0 (dumped in the case of new track),
. poor soils, grade QS1,
. medium soils, " QS2,
. good soils, " QS3

This type of structures was designed to meet the following requirements :

1.2.1.- Hydraulic operation : the deterioration of the track bed layers and formation depends on the water contents and therefore on the hydraulic mechanisms involved. Providing a sufficient transverse downgrade (ranging from 3 % to 5 %)of the layer of sand and gravel mixture under the ballast, a good particle size distribution and intensive compacting,4/5 ths of the rain water flow directly from the "ballast-subbase" interface to the longitudinal draining ditch ; the rest infiltrates the sub-base which dries up in 40 minutes once it has stopped raining. Variations in the water contents of the clayey formations are very small since the upper film is constantly saturated.

1.2.2.- Anti-contamination :

Under the effect of water and of the traffic, fines may rise through the ballast ; they are either due to the attrition of the ballast and sub-base, or more frequently to the deterioration of the cohesive formations. To avoid such phenomenon, the layer in touch with the ground must have optimum filtering qualities and permeability with regard to the ground ; it is therefore essential that the filter layer contain a significant amount of fine sand. This is achieved through a composite sub-base made up of :

. a layer under the ballast at the top, composed of a well-calibarted crushed sand gravel mixture 0/31.5 mm,

. if need be, an anticontamination sub-base at the bottom, made of a sand gravel mixture containing about 20 % of elements under 0,2 mm (fully crushed sand gravel mixture or with at least 30 % crushed material).

In poor and medium soils, a 400 g/m2 long-fiber non woven geotextile is inserted in between the sub-base and the ground to improve the anticontamination properties of the structure.

1.2.3.- Frost-proofing : herein referred to for the sake of comprehensiveness - the problem of frost which is little significant under the French climate, implies however a slight increase in the thickness of the foundation structures in the French most severe areas (East of France and the mountains).

1.2.4.- Bearing capacity :

From in situ observations, tests on life-size mock-ups and mathematical models, the size of the underlays has been defined in view of the geotechnical features of the ground and of the hydrogeological conditions ; the following thicknesses shall be complied with :

$$e = E - a + b - c + d$$

where the value of E is 45 - 55 or 70 cm depending on the strength of the formation, and a, b, c, d, parameters.

"a" is a function of the IUR category of the line (consequently of the yearly tonnage carried)

"b" is a function of the length and nature of the sleeper (wooden, concrete or prestressed sleeper)

"c" is a function of the sizing mode

"d" is a function of the nominal axle load.

The values of these parameters are determined in the Table of Appendix II.

All the notions which have led to the above calculations have been carefully studied to obtain reliable structures and a first-rate track behavior. However, they present the disadvantage of being very thick and it has been suggested to reduce their thickness through the addition of a geomembrane which would protect the formation against rain water.

II - STRUCTURES WITH A REINFORCED ASPHALTIC GEOMEMBRANE

2.1.- Definition of the structure

On susceptible grounds (grade QS1), soft rocks or rocks liable to decay, the thickness of the underlays may be significantly reduced by using a geomembrane (see figure 3).

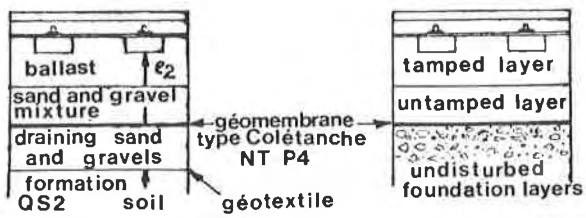


Figure 3 : Structures with a geomembrane

Then, the supporting ground is protected against rain water and its quality may be considered upgraded to the next grade.

Of course, when the ground contains a water table or an inner circulation, it is then necessary to lower the water and collect it carefully to prevent it from being trapped under the geomembrane and from soaking the supporting ground.

On new tracks, it is wise to provide for a filter layer below the geomembrane in addition to the abovementioned possible lowering measures.

On old tracks, during localized operations applying that technique, it is in general not possible to place the filter layer ; it is then necessary to carry out as thin a clearing as possible to preserve the pre-existing structure below the geomembrane. Moreover, the works must be done in such a way to avoid deteriorating the geomembrane upon lifting the track with a mechanical tamper (after laying a layer of untampered ballast or chippings).

It must be stressed that it is far easier to solve the problem of anticontamination by inserting a geomembrane than by inserting several thick layers of material of selected particle size. This structure is therefore of a special interest in the case of subgrades with a high fine or clay content.

2.2.- The asphaltic geomembrane

More than ten years ago, the reinforced asphaltic geomembrane "Coletanche NTP4" was used by the S.N.C.F. for water tighting railway subgrades ; the reinforcement consists of a non-woven fabric made of long fiber polyester ; the binder is blown asphalt (100/40 grade) which is therefore harder than a regular road asphalt and has a higher softening point ; moreover, its mechanical properties are more stable with regard to time and temperature range ; 20 % limestone filler is added into this hydrocarbon binder in order to still improve these characteristics. The mechanical properties of the reinforced asphaltic geomembrane used derive from those of its two components, asphalt and non-woven fabric : both of them react in a plastic way to slowly applied stresses and therefore this membrane adapts to any possible deformation in the supporting ground within its elongation range (about 50 %) by simply reaching a new equilibrium state. Moreover, this property of stress relaxation allows this kind of geomembrane to resist ageing better than a more elastic material.

This property, together with the fact that coating the fibers with asphalt modifies their mechanical behaviour within the fabric, probably explains why this kind of "reinforced asphaltic geomembrane" has a very good resistance to punching ; this property which was already suspected had to be ascertained by laboratory tests, before the decision was taken to use this material for practical applications on commercially operated railway links.

2.3.- Preliminary laboratory tests

S.N.C.F. therefore tested the "reinforced asphaltic geomembrane COLETANCHE" on the "Vigrogir testing Equipment".

The vibrogir is a device which allows to simulate, through pulsed loading, the action of the axles onto a sleeper and its foundation.

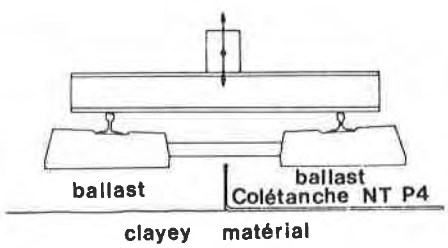


Figure 4 : Vibrogir

Under half of the sleeper, the ballast is laid directly on the clay ; under the other half, a strip of "Coletanche NTP4" geomembrane is placed as per figure 4. After operating the system over a period estimated to be equivalent to at least 20 years of traffic on the busiest S.N.C.F. tracks, it was noticed that, failing a geomembrane, the clay rose through the ballast and contaminated it entirely.

However, when Coltenache was used, the system had but slightly evolved with only a few ballast stones embedded in the asphaltic product without punching the geomembrane, and therefore no ballast contamination.

The results of the test being satisfactory, it was decided to carry out field tests.

2.4.- Historical description of placing Coletanche on railway formations

In fact, several development steps were undertaken before the condition of application of Coletanche were perfectly defined. The first application was made in 1973, using the "in situ" process, known under the name of "Coletanche N.T." ; at the new railway station of "Saint-Quentin" near Paris, blowns asphalt was sprayed onto a geotextile at a rate of 10 kg/sq m for protecting a new embankment made of clayey sand against rainwater. The 40 000 sq m geomembrane was protected from the ballast by a sand and gravel layer before the ballast was placed, taking not yet advantage of the geomembrane resistance to punching.

The same year, the same technique wes used in another location near Paris in chennevières ; however, the "Coletanche N.T." used was then prefabricated in plates 9 m long and 2 m wide which allowed its characteristiques to be improved and its asphalt content to decrease. This was the first step toward prefabrication in factory which now allows the asphalt content to be limited to 7 kg/sq m and the Coletanche N.T.P. 4 (grade used for railway plateform protection) to be manufactured in strips 55 m long and 4 m wide.

Coletanche was then used, near Limoges, to shelter a weathered granite subgrade formation against rainwater thus improving its bearing capacity and more especially to prevent the rising of clay into the ballast as this had caused frequent maintenance operations. The site was located in a deep cut, leading to a tunnel. The side ditch level could therefore not be modified and thick layers of gravely materials could not be placed. It was then the occasion of taking advantage of the good resistance of Coletanche to punching. After the tests carried out on the S.N.C.F. vibrogir equipment, proved to be positive, it was decided to lay this geomembrane directly on the formation cleared from old ballast (see photograph n° 1) and to spread the new ballast directly on it. This site was considered by S.N.C.F. as a probatory section of this technique.

Photograph n° 1

The geomembrane was checked after 5 and 10 years and the ballast stones were found to have become incrusted into the Coletanche without punching it, thus confirming the findings of the "vibrogir" tests. (see photograph n° 2).

The good results observed on the test section led S.N.C.F to resume using the reinforced asphaltic geomembrane on its railway links.

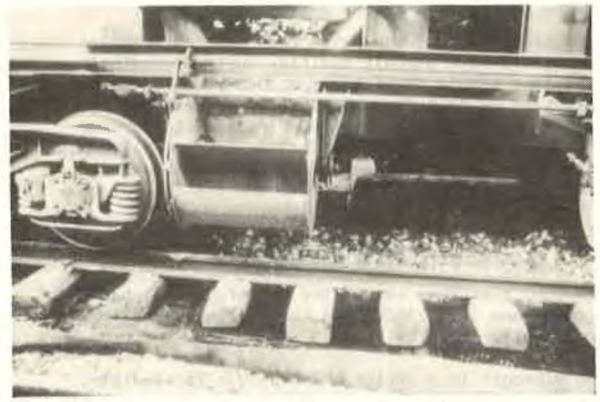
It was then used in 1982-1983 for protecting a railway limestone subgrade where short stretches containing plastic material were found. Small lengths would not justified "heavy protection wroks" while protecting them on the occasion of renewing of track and ballast became possible. Once the old ballast is cleared, track panels are lifted with jacks for inserting geomembrane panels (see photograph n° 3).

Photograph n° 2

The new ballast is placed directly on the geomembrane before the old track panels are removed, renewal works can then be carried on without slowing down. (See photograph n° 4).

Photograph n° 3

This technique worked satisfactorily and was then extented to longer stretches where heavy protection works had initially been planned.



Photograph n° 4

This technique is therefore cheaper than conventional protections of railway subgrades for two reasons :

1.- it saves the cost of deep cut below formation followed by placing layers of new selected material

2.- it can be placed on the occasion of rail and ballast renewal works, thus saving the cost of preliminary protection works. Additionnally no extra interruption is required for placing protection layers.

2.5.- Conclusions

Developments in the railway technology have led to a highly precise definition of the foundation structures to be laid, to guarantee a good track behavior given the bearing capacity of the formation ground ; it is in particular necessary to prevent the fines from rising through the foundation structures.

The use of a reinforced asphaltic geomembrane may help obtain a thinner structure (the bearing capacity of the ground protected against rain water is therefore increased) and prevent the fines from rising; it requires less maintenance operations and is consequently cost-saving. However, it is essential to avoid blocking any inside water through efficacious lowering and draining operations.

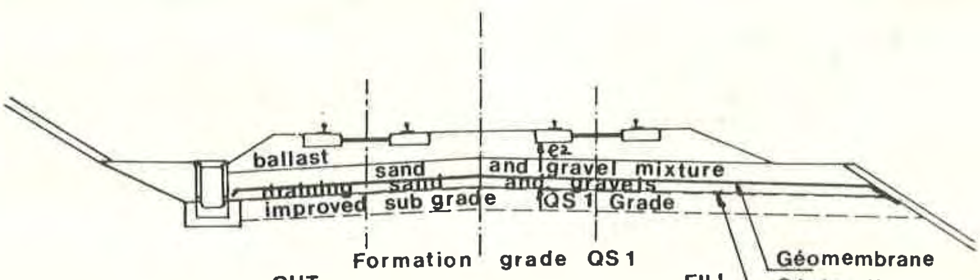


Figure 5 : typical cross-section ; new track

APPENDIX 1 : SOIL GRADES

Excerpt of the International Union of Railways data sheet 719 R "Earth Works and Track bed Construction for Railway Lines"

Soil classification (geotechnical identification)	Soil grade
0.1 Soft organic soils 0.2 Fine soils (with over 15 % fines (1) bulked, moist and therefore uncompactable) 0.3 Thixotropic soils (2) (quick-clay for example) 0.4 Soluble materials (for example soil with rock salt or gypsum) 0.5 Polluting materials (polluting industrial refuse for example) 0.6 Mixed "mineral-organic" soils (2)	QS 0
1.1 Soils with over 40 % fines (1) 1.2 Highly evolutive rocks for example Highly brittle chalk with $\rho_d < 1{,}7$ t/m3 Marl Weathered schists	QS 1
1.3 Soils with 15 to 40 % fines (1) 1.4 Evolutive rocks for example Slightly brittle chalk with $\rho_d < 1{,}7$ t/m3 Non-weathered schists 1.5 Soft rocks for example when dry Deval < 6 and Los Angeles > 33	QS 1 (3)
2.1 Soils with 5 to 15 % fines (1) 2.2 Sand with at least 5 % fines (1) but uniform 2.3 Fairly hard rocks for example when $6 \leq$ dry Deval > 9 and $33 \geq$ Los Angeles > 30	QS 1 (3)
3.1 Soils with less than 5 % fines (1) 3.2 Hard rocks for example when dry Deval ≥ 9 and Los Angeles ≤ 30	QS 3

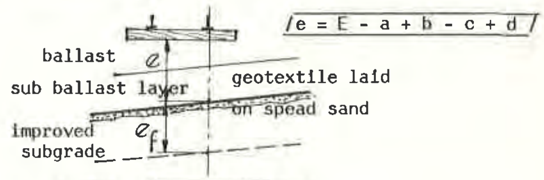
(1) Particle size analyses are carried out on materials passing a 60 mm test sieve to determine the percentages. The abovementioned percentages are approximate (the prevailing rules somewhat differ according to the Networks) ; they may be increased by up yo 5 % providing the analyses involve a sufficiently representative number of samples.

(2) Under specific circumstances, a few Networks include these soils into grade QS 1.

(3) These soils may be of grade QS 2 if it is known with certainty that the hydrogeological and hydrological conditions are good.

(4) These soils may be of grade QS 3 if it is known with certainty that the hydrogeological and hydrological conditions are good.

APPENDIX 2 : THICKNESS E (ballast - sub ballast layer)



Values of "E"

Grade of the supporting ground	Improved subgrade positioned: Grade	Improved subgrade positioned: Thickness of (meter)	Bearing capacity of the subgrade	E (meter)
QS 1	QS 1	--	P1	.70 + geotextile
	binder-stabilized fine soil	.30	P2	.55 + geotextile
	QS 2	.55	P2	.55 + geotextile
	QS 3	.40	P2	.55
	QS 3	.60	P3	.45
QS 2	QS 2	--	P2	.55 + geotextile
	QS 3	.40	P3	.45
QS 3	QS 3	--	P3	.45

Values of "a"

- 0 — for lines of IUR categories 1 and 2 or lines with V $\geq$ 200 km/h
- .05 m — for lines of IUR category 3
- .10 m — for lines of IUR categories 4, 5, 6 and 7, 8, 9 with passengers
- .15 m — for lines of IUR categories 7, 8, 9 without passengers

Values of "b"

- 0 — for wooden sleepers 2.60 m long
- $\frac{2.50-L}{2}$ — for concrete sleepers L long (b and c are in meters, b may be netagive when L > 2,50 m)

Values of "c"

- 0 — for standard sizes
- .10 m — on exceptional occasions, for difficult operations on existing lines of IUR categories other than 7, 8, 9 without passengers
- .05 m — on exceptional occasions, for difficult operations on existing lines of IUR categories 7, 8 9 without passengers

Values of "d"

- 0 — when the maximum nominal axle load of hauled vehicles is $\leq$ 200 kN
- .05 m — when the nominal axle load is 225 kN
- .12 m — when the nominal axle load is 250 kN
- .35 m — when the nominal axle load is 300 kN

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Reliable Groundwater Protection With Fabric-Reinforced Bitumen Membranes

Early in 1970 a reinforced, 5m-wide bitumen membrane was developed to serve as lining and waterproofing material of the dikes enclosing three large water reservoirs near the city of Rotterdam in the Netherlands. More than a decade on, the Hypofors liner then evolved is well proven in a variety of field conditions. This paper discusses the record of experience with regard to penetration by plants; behavior in sunlight, rain, wind, heat, and cold; and the effect of environmentally hazardous chemicals. The function of the reinforcement under two-dimensional stresses and in tensile and puncture tests is also dealt with. Special attention is devoted to the watertight, tension-releasing, sliding bitumen joints which allow individual widths of liner to be linked up simply and securely.
The data furnished should permit independent assessment of the author's claim that reinforced bitumen membranes are not only a strong and reliable means of impounding water but can also efficiently keep many solid and liquid wastes from contaminating the soil.

INTRODUCTION

Bitumen is a material which has a long history of use by man in his efforts to deny entry to water. To-day, stone or gravel enveloped in thin layer of bitumen is widely employed in embankments and dams, and in the construction of road surfaces.Installation of prefab bitumen sheets is very common practice in the waterproofing of roofs. However, the use of similar flexible bitumen sheets as waterproof liner in civil engineering did not gain wide currency until the last 15 years.
Several factors delayed the civil engineering application:
* Many hydraulic engineers have seen evidence of plants penetrating thick asphalt layers.
* Thin bitumen films adhering to stone become hard and brittle after having been exposed for a time to wind and weather.
* Where soil settlement is irregular or where asphaltic concrete is connected to rigid structures, the low deformability of asphalt can easily result in its cracking.
* Bitumen is known to dissolve in a number of common chlorinated and aromatic hydrocarbons.

This paper is concerned to demonstrate that it is possible to circumvent these pitfalls and hazards, and that reinforced bitumen membranes have their legitimate uses both in hydraulic engineering and in pollution control.

LINING OF THE BIESBOSCH WATER RESERVOIRS NEAR ROTTERDAM

In order to secure a sufficient supply of drinking water for the southwestern part of Holland, in the early seventies a huge project was begun in the Biesbosch area. It called for the construction of three reservoirs with a total water surface of about 7 million m^2 with a partially permeable silt layer at the bottom and a waterproof construction on the slopes.
One consequence of this design is that the groundwater in the dike body reaches almost the same level as the water in the reservoirs (see Figure 1).

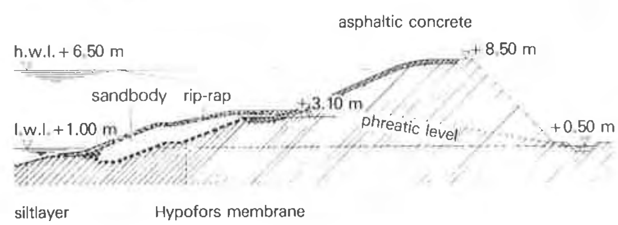

Figure 1. Construction of Biesbosch reservoir dikes

However, a sudden surge in water demand will cause the level in the reservoirs to drop more quickly than the groundwater level in the dikes, creating a hydraulic excess pressure under the lining. To compensate for this pressure, the lining at the lower part of the dike had to be ballasted with two metres of sand.
Predicted movements in the supporting subsoil called for the use of a flexible liner capable of sustaining the two-dimensional stresses under a heavy ballast. Extensive studies and full-scale experiments initiated by the engineers responsible for the project resulted in the creation of the first Hypofors liner, a 5m-wide, fabric-reinforced bitumen membrane (1).
Its reinforcing fabric, with 900 ends/picks of 940 dtex high-tenacity Enka Nylon yarn per meter, has an almost uniform breaking force per unit width in all directions, as illustrated in Figure 2.

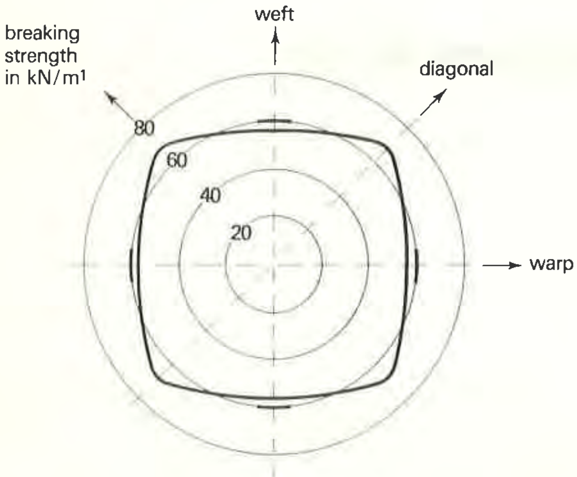

Figure 2. Force at failure (α_f) of Hypofors reinforcement fabrics as a function of the direction of testing

The elongation at failure diagonal to the yarn direction is about 1.5 times higher than in the direction of warp and weft (Figure 3).

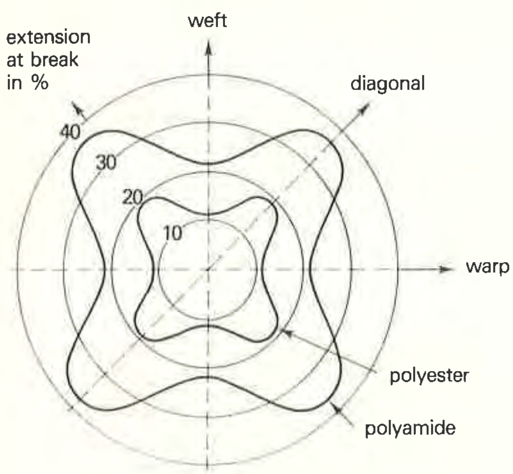

Figure 3. Elongation at failure (ε_f) of Hypofors reinforcement fabrics as a function of the direction of testing

The results shown were derived from experiments made on sleeves blown up by water pressure, while clamps at top and bottom prevented shortening. A thin rubber membrane was fitted inside the sleeve to render it waterproof (2).

FINDINGS DURING PRODUCTION AND USE IN THE FIELD

While the composition of the Hypofors membrane (Figure 4) has not been fundamentally changed over the past 15 years, production and installation of over 4 million m^2 of bituminous liner brought to light certain important aspects which have to be taken into account whenever application of reinforced bitumen membranes is considered.

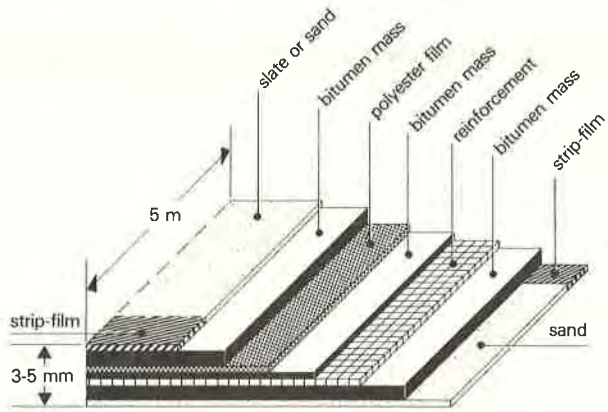

Figure 4. Composition of Hypofors membranes

(a) Resistance to penetration by vegetation

The 23-micron biaxially-drawn polyester film incorporated in the membrane possesses high resistance to deformation and thus constitutes an effective barrier to plant tops or roots which do penetrate into the bitumen layer but are then deflected sideways by the film.
The development of a reliable method of joining the single widths of film into a continuous 5-metre width has absorbed much time and effort.
The protective bitumen layer on the film limits the risk of damage in handling and placement.
Penetration of plants through the overlaps between sheets can be prevented by the use of additional bitumen, containing a suitable growth-inhibiting additive, poured into the joint.

(b) Atmospheric influences

The component which confers watertightness, namely the bituminous mass, should be sufficiently flexible to adapt to the irregularities of the subsoil, sufficiently strong not to be damaged by rough handling or by sharp objects in the soil, and sufficiently resistant to UV degradation and other weather influences during outdoor storage.
* Sunlight and rain
The degradation of a 3-5mm thick membrane upon exposure to sunlight and rain is substantially lower than that of the thin bitumen layers found in asphalt applications. The malthenes present in the bitumen migrate from the inside outward to the hardening surface and so ensure that it retains its flexibility for a long time without cracking (3).
Tests with various types of Hypofors in a Xenotester have shown that weatherability can be considerably improved by covering the surface with crushed slate (4).
* High and low temperatures
Achievement of the desired membrane behavior at high and low temperatures is a matter of careful selection of blown bitumen, filler, and elastic additives.
A good flexibility at low temperatures requires the production of bitumen membranes of uniform thickness made from a homogeneous bitumen mixture which under controlled-temperature conditions saturates and covers the inlays. Naturally this also requires machinery of an appropriate level of sophistication.
While thermoplastic or elastomeric films require

flexural testing at -20°C or -40°C to ascertain serviceability, bitumen membranes can be laboratory-tested at the temperatures likely to occur in the field.
The design of the tests should, however, make appropriate allowance for interactions of the various external influences exerted on the liners. Thus the flexibility of a bitumen membrane, normally a function of thickness and composition, may be diminished by long residence at high temperatures; conversely, it may may be increased by contact with certain chemicals.

* Wind

On account of its mass - more than 6kg/m^2 - and the close contact of its sanded surface with the soil, the reinforced bitumen membrane does not have to be weighted during installation.

(c) Behavior under two-dimensional stresses

A practical finding of the past few years, supplementary to the facts set forth in section 2, is that it takes specialty fabrics and strict production control to make a bitumen membrane with a reinforcement which is properly positioned and which affords proper adhesion to the coating bitumen.
The choice of the reinforcing material must allow for the high temperatures during the coating process and for the strength needed for the project. (Note that the thickness of the bitumen layer has little relevance to that strength.) Table 1 presents some characteristic properties of common Hypofors types.

Table 1 - Some characteristic properties of common Hypofors types.

Property	Hypofors Type		
	NF 1000	NF 3000	CF 7000
Force at failure (in kN/m)	52	32	12
Strain at failure (in %)	18	18	45
Tear propagation (in N)	850	450	210

(d) Resistance to puncturing

Reinforced bitumen membranes generally have a high resistance to puncturing, by virtue of their thickness of at least 3mm needed to properly embed the inlays.
The bitumen will particularly resist deformation in response to the impact of falling objects. To prevent slow puncturing by hard objects, such as stones, the meshes in the reinforcing fabric need to be closely spaced.
The resistance to damage can be much increased by facing the membrane with a nonwoven (5).

(e) Resistance to tearing and tear propagation

Coated fabrics, reinforced bitumen membranes included, will generally show a higher breaking force in tear tests than nonreinforced liners. In-depth study of various types of coated fabrics evaluated by different tear tests have demonstrated that the results of different tear test methods must never be used to compare competitive products (6). For fabric-reinforced membranes the trapezoidal method is preferred since it strains the yarns parallel to the direction of testing. The ASTM D1004-70 method, frequently recommended for plastic films, is not suitable for reinforced liners because of the shape of the test specimen.

(f) Resistance to chemicals

Especially where the membrane is used to line waste storage pits or lagoons, its chemical resistance is of major interest. Since the hazardous chemicals are usually carried to the waterproof liner dissolved in (rain) water, both the Enka laboratory and an independent German testing institute investigated the ability of Hypofors to withstand 10-% dilute or saturated solutions of the following mixtures of chemicals:

Aromatic hydrocarbons
40% Iso-octane
15% Benzene
15% Toluene
10% Methyl naphthalene
15% Xylene

Oils and fats
35% Diesel oil
35% Paraffin oil (C_{10}-C_{20})
30% Lubricating oil HD30

Amines
Dimethyl amine

Alcohols
30% Methanol
30% Isopropanol
40% Glycol

Aliphatic chlorinated hydrocarbons
30% Trichloroethylene
30% Tetrachloroethylene
40% Methylene chloride

Aliphatic esters and ketones
50% Ethyl acetate
50% Methylisobutyl ketone

Aliphatic aldehydes
Formaldehyde

Organic Acids
50% Acetic acid
50% Propionic acid

Inorganic Acids
50% Sulfuric acid
50% Nitric acid

Inorganic bases
Sodium hydroxide

Inorganic salts
50% Sodium Chloride
50% Sodium sulfate

The test specimens were rated after 7, 14, 28, 56, and 90 days for appearance, flexibility, and change in weight. As the chemicals which are most aggressive to bitumen have poor solubility in water, none of the mixtures tested had a demonstrable negative effect on the membrane specimens(7).

THE WATERPROOF CONNECTION OF BITUMEN MEMBRANES

The performance of a liner must be matched by the joints between the sheets. Joints in reinforced bitumen membranes differ in some basic respects from rigid seams in thermoplastic or elastomeric liners.
* The rheological properties of bitumen cause sheet overlaps to slide over each other under stress, thereby relieving the stresses exerted.
Consequently, overlaps need to be sufficiently wide to absorb all stresses - e.g. such as are caused by irregular movements in the supporting subsoil - without loss of watertightness; in the concrete this means that a minimum overlap of 75mm must remain at all times.
* Pouring hot bitumen into the joint does not call for special skills on the part of the operator and is never detrimental to the lining material next to the joint. Thus there is no risk of a decrease in strength or tightness by local overheating or by overdoses of solvent.
* The hot bitumen softens the overlap sections of the bottom and top sheets, making them extremely flexible and therefore capable of adapting to minor irregularities in the supporting soil.
Because of these factors, a visual inspection, supplemented perhaps by an occasional vacuum test, is sufficient to establish the reliability of the joints.
In making the joints, it is recommendable to use hot blown bitumen of a slightly lower viscosity than that used in the coating process. While this will result in a somewhat lower shear strength in the joint, it reduces the risk of formation of a sliding plane inside one of the sheets rather than between adjacent sheets (see also Figure 5).

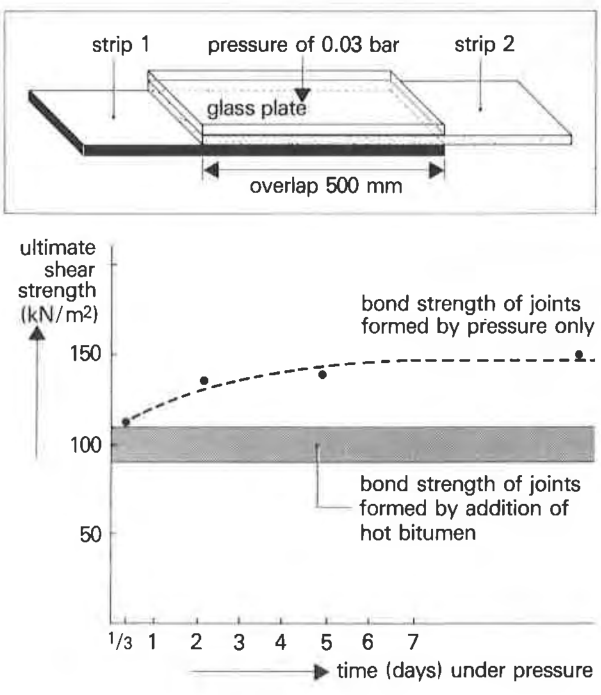

Figure 5. Pressure bonding of bitumen joints

The bitumen poured into the joint may also help prevent penetration of the overlap by plants.
The behavior of the bitumen joint has been extensively researched by Enka's Arnhem Laboratories. Figure 6 illustrates the behavior of bitumen joints permanently exposed to shear.
In a series of experiments the force exerted on the overlap by different weights suspended from the lower strip was plotted against the time elapsing until the two strips completely separated. For example, a shear force of 500N per 250m^2 overlap (20kN/m^2) was found to cause separation after 24 hours at 21°C.
It should be stressed, however, that this is not representative of the conditions met in practice, when the soil under the joint will only give way until it has consolidated far enough to provide adequate support.

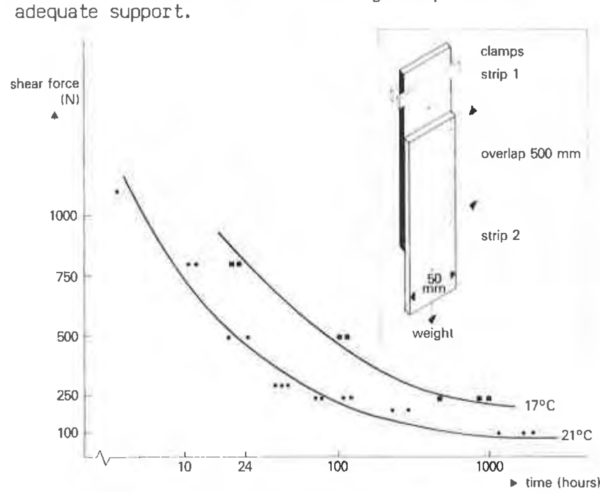

Figure 6. Behavior of bitumen joints under shear

In a second series of experiments, a number of 50mm-wide strips were fused together by pressure only.
The ultimate shear strength of the bonds so formed was determined by testing the 250cm^2 overlaps at a pull rate of 50mm/min at 21°C. Figure 5 plots the data obtained against the number of days under pressure.
Figure 5 permits the conclusion that two bitumen surfaces placed on top of each other will form a tight connection within 10 hours, even when the pressure is relatively low.
A comparison of the results from both tests clearly illustrates the influence of the rate of pull: at a rate of 50mm/min the shear strength is about 5 times higher than at a rate of 0.35mm/min (500mm/24 hours).

CONCLUSIONS

(a) It is possible to produce a prefab membrane of high functionality in environmental protection and water conservation from blown bitumen, fillers and elastomeric additives, a strong isotropic reinforcing fabric, and a biaxially-drawn polyester film.

(b) The composite character of such a membrane offers the possibility of custom-tailoring liners to the specific demands of a given project.

(c) The manufacture of a wide, reinforced bitumen membrane requires sophisticated machinery and careful control of the production process

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Performance of the Fabric-Reinforced Geomembrane at Mt. Elbert Forebay Reservoir

The Mt. Elbert Forebay Reservoir was lined with a 1.14-mm (45-mil) CPER (reinforced chlorinated polyethylene) geomembrane in 1980. Concern that seepage through a previously constructed earth lining was threatening stability of an ancient landslide in the hillside directly south of the reservoir led to the decision to install a continuous polymeric geomembrane. Since January 1981, measured groundwater levels in the hillside have shown a slow but consistent decline in groundwater levels. Inclinometers installed in the hillside have not indicated any movement.

As part of a warranty program and long-term observation of the geomembrane, a field test section was installed within the reservoir. The test section consists of large coupons of the geomembrane containing all field seam types used in the manufacture and installation of the geomembrane. Coupons have been recovered on a yearly basis for laboratory testing. After fulfilling the requirements for warranty purposes, the coupons will be recovered as needed to monitor long-term performance.

INTRODUCTION

The USBR (U.S. Bureau of Reclamation) installed approximately 117 ha (290 acres) of 1.14-mm (45-mil) CPER flexible membrane lining in the Mt. Elbert Forebay Reservoir in 1980. The reservoir is situated immediately north of Twin Lakes, a pair of glacial-age lakes located approximately 24 km (15 mi) southwest of Leadville, Colorado. Located at an elevation of 2940 m (9645 ft), the reservoir serves as the forebay for the Mt. Elbert Pumped-Storage Powerplant and is approximately 133 m (435 ft) above the powerplant. The forebay reservoir and the powerplant are used to provide peaking power for the East Slope Power System of the Bureau's Fryingpan-Arkansas Project.

The reservoir has a capacity of 14.22 x 10^6 m^3 (11,530 acre-ft) of water of which 8.8 x 10^6 m^3 (7,160 acre-ft) are available for generating electrical power. Two 103-MW (138,000-hp) hydroelectric turbine-generators which generate the power are designed also to operate as 127-MW (170,000-hp) motors to pump water from Twin Lakes back to the forebay reservoir during periods of low electricity demand.

The reservoir was built between 1975 and 1977 by constructing a 27-m (90-ft) high zoned earth embankment across the open north side of a topographic depression, and a small dike in the open southwest corner of the depression.

A portion of the hillside between the forebay reservoir and the powerplant had been mapped geologically as an ancient landslide. Concern had been expressed that seepage from the reservoir might reactivate the slide. The concern over seepage and its effects on the hillside resulted in a decision to completely line the reservoir with a 1.5-m (5-ft) thick compacted earth lining. The earth lining was placed over the entire reservoir area up to an elevation of 0.9 m (3 ft) above maximum water surface.

Between November 1977 and March 1978, water was pumped into the eastern half of the forebay reservoir. The water was introduced in stages until a depth of approximately 7.6 m (25 ft) [elevation 2926 m (9600 ft)] was reached. The water level remained at or slightly below this elevation until September 1979 when the reservoir was drained in anticipation of installing a membrane lining. During the time that the reservoir was at the elevation of 2926 m (9600 ft), several observation wells installed in the hillside between the reservoir and the powerplant showed an upward trend in the groundwater level. By August 1979, it had become apparent that the reservoir was causing a rise in the groundwater levels in the hillside between the forebay reservoir and the powerplant. Fears that the rising groundwater would lessen the stability of the hillside and reactivate the ancient landslide led to the decision in August 1979 to place a continuous polymeric geomembrane in the forebay reservoir. This paper briefly discusses the selection and installation of the 1.14-mm (45-mil) thick CPER geomembrane in 1980, the extensive ongoing monitoring programs, and the performance of the lining after 3 years of service. Also, future monitoring of the lining will be discussed. A discussion of the events leading to installation of the geomembrane and a description of the actual installation procedures may be found in reference (1).

SELECTION AND INSTALLATION OF THE MT. ELBERT GEOMEMBRANE

An intensive effort was made to investigate all critical requirements of a membrane lining for long-term service under the unique and severe climatic and operating conditions to be encountered at Mt. Elbert. The reservoir, located in a high mountainous terrain, is subjected to severe daily and seasonal temperature extremes, high winds, and thick ice formation in the winter. The maximum head in the reservoir is 21 m (70 ft) with the water level fluctuating approximately 9 m/week (30 ft/week) as a result of the pump-generating operations.

The investigation revealed few similar installations and none with extended service records on membrane materials being produced at that time. The study indicated that membrane formulations changed frequently as industry corrected weaknesses in earlier products and sought to exploit new technology to improve materials. Effective factory and field seaming methods were recognized as critical. Also, a heavy earth cover was deemed essential to long service under severe climatic conditions. Literature surveys provided only fragmentary and inconclusive records regarding the long-term properties and performance of current products.

The investigation indicated that a flexible membrane lining was best suited for the purpose of providing a tight, durable lining for the forebay reservoir. Based on the "state-of-the-art" survey, availability of materials, time frame to accomplish the work, and costs, specifications were prepared in the fall of 1979 and issued in January 1980. Alternate bidding schedules for installation of any one of three lining materials were included in the specifications. These included 1.14-mm (45-mil) CPER, 1.14-mm (45-mil) RCSPE (reinforced chlorosulfonated polyethylene), and 2-mm (80-mil) HDPE (high density polyethylene). The low bidder selected CPER.

The contract for installation of the membrane lining under USBR Specifications No. DC-7418 (2) was awarded on April 16, 1980, and installation was completed on September 20, 1980.

Construction Considerations (3) were prepared which outlined the concepts used in preparation of the specifications.

The principal features of work covered by the specifications for the Mt. Elbert Forebay Reservoir membrane lining included: removal of all existing slope protection; excavating and processing the top 0.61 m (24 in) of the impervious earth lining; preparation of the subgrade; manufacturing, fabrication, testing, and installation of the geomembrane; placement of the earth cover; and replacement of the slope protection.

It was important that the geomembrane be placed upon a subgrade that did not contain gravel or cobbles which could puncture it. Before placement, subgrade material obtained from the excavation of the existing earth lining was processed to remove all aggregate larger than 25 mm (1 in). Crushing in lieu of separation was not permitted because that would result in angular fragments which could puncture the geomembrane.

Processed earth material was placed and compacted to a minimum depth of 0.15 m (6 in) as a subgrade for the membrane lining. To obtain a smooth subgrade, hand labor was used to remove loose or protruding gravel and other materials which could puncture the membrane. A minimum of 0.45 m (18 in) of processed earth was placed over the membrane lining.

GEOMEMBRANE DESCRIPTION

The CPER lining material is of three-layer construction consisting of two equal thicknesses of CPE (chlorinated polyethylene) laminated to one layer of 10 by 10, 1,000-denier polyester scrim. The specified physical properties requirements for this lining are given in table 1.

The lining was factory fabricated into "blankets," each 1300 m^2 (14,000 ft^2) in size and weighing approximately 2268 kg (5,000 lb). Two shapes of blankets were furnished: 61 by 21 m (200 by 70 ft), containing 14 factory seams made with a Leister™ hot-air gun, and 30 by 43 m (100 by 140 ft), containing 29 factory seams made dielectrically.

To install the membrane lining, labor crews unfolded and positioned the blankets. Adjacent blankets were overlapped a minimum of 0.15 m (6 in). A three-man crew thoroughly cleaned the contact surfaces with trichloroethylene solvent, and the manufacturer's bodied-solvent CPE adhesive was applied to a minimum width of 100 mm (4 in). The field seams were then hand rolled and allowed to cure a minimum of 4 hours before air lance testing to detect any weak or unbonded areas. The air lance test consisted of air at 345 kPa (50 lb/in^2), supplied through a 5-mm (3/16-in) nozzle directed at the upper seam edge. After the field seams were tested and approved, a cap strip was applied over the field seam. The cap strip consisted of a 0.76-mm (30-mil) thick unsupported CPE, 75 mm (3 in) wide.

An extensive QA (quality assurance) program was conducted in conjunction with the Mt. Elbert flexible membrane lining installation. The QA program consisted of the following major activities:

1. Furnishing (by the Contractor) toxicity data to verify no toxic effects on cold water species of aquatic life.

2. Verification (by a third party) of the chemical formulation of the CPER.

3. Visits by USBR personnel to inspect factory manufacturing and testing facilities and operations.

Table 1. - Test methods and physical properties requirements-CPER

Property	Test method	Minimum requirement
Thickness	ASTM: D 751-79	1.04 mm (0.041 in)
Breaking strength, each direction	ASTM: D 751-79 Grab Method A	890 N (200 lbf)
Tear strength, each direction	ASTM: D 751-79 Tongue Tear Method B	334 N (75 lbf)
Bonded seam[1] strength in shear	ASTM: D 751-79 Grab Method A	Equals parent material breaking strength
Bonded seam[1] strength in peel	ASTM: D 1876-78	No specs requirement
Dimensional stability (percent change, maximum)	ASTM: D 1204-78 1 hour at 100 °C (212 °F)	2 percent
Low temperature bend	ASTM: D 2136-78 3-mm (1/8-in) mandrel; 4 hours at -40 °C (-40 °F)	Pass
Hydrostatic resistance	ASTM: D 751-79 Method A	2.07 MPa (300 lb/in^2)
Ply adhesion	ASTM: D 413-76 Machine Method Type A Specimens	1400 N/m (8 lb/in)
Infrared spectroscopy	B.F. Goodrich lab procedure	Matching IR scan

[1] These same test methods were used on all seam samples taken in the field.

4. Submittal by the Contractor of certified laboratory test reports on physical properties for each day's production of CPER.

5. Visits on a weekly basis (by USBR inspectors) to view and monitor factory seaming methods.

6. Testing, in the Bureau laboratories, samples from every tenth blanket produced.

7. Air lance testing the entire length of all factory and field seams.

8. Daily sampling and visual inspection of all field seams and blankets at the site.

9. Mechanical testing of daily field seam samples.

A comprehensive discussion of the QA program is included in reference (4).

LINING PERFORMANCE

Reservoir Behavior

After completion of the lining installation in 1980, the reservoir was refilled beginning in January, 1981. By June 1981, the reservoir had been filled to elevation 2940 m (9645 ft). The reservoir has remained above elevation 2935 m (9630 ft) since that time except for two short periods.

Piezometers and observation wells installed in the hillside south of the reservoir continue to be monitored on a regular basis. In general, all of the instruments, which had begun to rise in an apparent response to the initial reservoir filling, have shown a leveling off or decline in groundwater levels. Two piezometers which showed significant increases in groundwater level during the first reservoir filling have shown a steady rate of water level decline since January 1982.

Inclinometers installed along the south side of the reservoir have not indicated any movement of the old landslide mass.

Visual examination of the reservoir slopes in 1983 did not show any movement of the riprap slope protection which protects the earth cover above the lining.

Special Coupon Monitoring Test Section

Included in the Mt. Elbert specifications was a 5-year maintenance warranty period on the membrane lining (2). To monitor the performance of the lining during the warranty period and for long-term research purposes, a special field test section with CPER coupons was installed in the forebay reservoir. On August 14, 1980, a 6- by 30-m (20- by 100-ft) test section was installed in the southwest corner of the reservoir. This location at elevation 2936 m (9633 ft) was selected to allow easy retrieval of samples over the years. The test section is within the 9-m (30-ft) water level fluctuation of the reservoir and on a relatively flat beaching slope. The coupons were placed on a 50-mm (2-in) cushion of sand above and separate from the main lining, thus precluding the need to cut and patch the actual lining to obtain aged samples.

It was decided to compare all three types of seams (hot-air, dielectric, and bodied-solvent adhesive), as well as capped versus uncapped field seams. Also, anticipating a potential problem of water wicking within the polyester scrim, it was decided to test coated edges versus noncoated edges in an effort to detect any noticeable physical property changes. The long edges of each panel section were protected from wicking by coating them with the bodied-solvent adhesive used in fabricating the field seams.

Eleven test panels, each measuring 1.5-m by 6-m (5- by 20-ft) comprise the total test section. The panels were positioned approximately 1.5-m (5-ft) apart starting with panel No. 1 at the south end of the test section location. The area of the test panels was covered with the select cover materials as originally specified for covering the Mt. Elbert lining. Figure 1 shows the installation of CPER coupons in the test section area.

Figure 1. - Installation of Special Coupon Monitoring Test Section in Mt. Elbert Forebay

Because of the location of the test section and the method of installation, the following exposure conditions will be different from that of the main liner:

- Reservoir water will have access to both sides of the coupons.
- Effects of stresses introduced into the actual reservoir lining during installation and operation will not be reflected except for freeze-thaw cycling.
- Effects of hydrostatic pressure present in deep parts of the reservoir will not be evident.

The coupon sampling program for the test section was initiated with panel No. 1 in the spring of 1981. Panels No. 2 and 3 were taken in 1982 and 1983. Panels No. 4 through 11 will be taken on a yearly basis thereafter. If, after 5 years, there is little significant change in the physical properties, the sampling program will be changed to one sample panel every 2 or more years until all panels have been removed.

The following listed items are the physical, analytical, and mechanical property tests which are conducted on the coupons to determine changes in the CPER sheet material and the various types of seams used in the lining installations:

- Weight and thickness changes.

- GC (gas chromatography) and GCMS (gas chromatography-mass spectrometry).
- Hydrostatic resistance using the Mullen Burst (ANSI/ASTM: D 751-79, Method A).
- Hydrostatic puncture resistance using the USBR test facility (5).
- Ply adhesion (ANSI/ASTM: D 413-76, machine method, type A).
- Tear resistance using the tongue tear (ANSI/ASTM: D 751, Tongue Tear, Method B).
- Low temperature bend test using a 1/8-inch mandrel (ANSI/ASTM: D 2136-78).
- Seam strength retention in peel (ANSI/ASTM: D 1876-78).
- Seam strength retention in shear (ANSI/ASTM: D 751).

The wicking phenomenon associated with the polyester scrim will be examined by comparing sections of the test coupons that have exposed (cut) scrim edges with those that are coated with bodied-solvent adhesive. In addition, GC and GCMS will be used in an effort to detect any changes in chemical composition over time.

Additional laboratory studies on long-term water immersion were started in November 1980 on retained samples of the CPER membrane lining and seams. These samples are being subjected to Denver, Colorado running tapwater at a temperature of 10 to 16 °C (50 to 60 °F). The samples are removed and tested after the following intervals: 13, 28, 56, 78, 108, 156, 208, and 260 weeks. Laboratory tests will be conducted to monitor changes in the membrane lining using the same tests as outlined previously with the exception of puncture resistance. The results of laboratory testing is being compared to field performance data.

Site Inspection and Physical Property Testing of CPER after 36 Months of Service

An inspection of the test section was made on September 15, 1983. Test panel No. 3 was uncovered after 36 months of exposure in the field. The test panel coupon was buried under approximately 1 m (39 inches) of earth cover material. Figure 2 shows the panel being removed. Visual inspection of the main liner under the test section showed the CPER to be in excellent condition. The water level in the forebay had been lowered to allow for the test panel removal. Temperature at and below the test panel was 46 °F (8 °C). General observations of the test panel upon extraction indicated that it was in good condition with no apparent signs of deterioration and no damage due to installation and placement of the cover material.

Test section panel No. 3 was returned to the E&R Center, Bureau of Reclamation, for testing and evaluation. Table 2 lists the results of these tests. Five to 10 specimens were cut and tested for both the original and aged test results. The tear and Mullen hydrostatic tests utilized 10 specimens each. Original test specimens were taken from the same blanket panels as those used in the test section in an effort to reduce the effects due to variability between manufactured blanket panels and roll goods. Therefore, the results listed in table 2 are a direct comparison of properties within the same blanket panel before and after exposure at Mt. Elbert. In addition to the tests in table 2, USBR

a. Removal of earth cover.

b. Removal of 36-month coupon panel after excavation (note saturated soil conditions and sand layer on bottom).

Figure 2. - Removal of the special test section coupon.

hydrostatic puncture resistance testing was completed. The hydrostatic puncture resistance tests indicated that the CPER retained its distributed stress puncture resistance strength as compared to the original tests performed in 1980. This is an indicator of actual scrim strength and resistance of the laminated material to creep under loading. The Mullen burst test results also indicate good scrim strength retention.

Low temperature bend testing provided noticeable differences between the original CPER material in one of the blankets and the test section failure rates. All low temperature specimens tested failed from blanket 2097, whereas 80 percent passed from blanket 570.

Blankets 2097 and 570 were produced from different production lot numbers.

With reference to table 2, the following specific observations on test results can be made:

- The coupon gained an average of 15.6 percent moisture which was within the expected water absorption rate for the 1.14-mm (45-mil) CPER as shown by earlier testing.

- The Mullen burst resistance tests primarily reflect the scrim strength with some strength contribution due to the composite construction. This mechanical property did drop 9 to 15 percent reflecting some change within the composite or scrim alone.

- Tear resistance testing utilizing the tongue tear method was inconclusive showing a loss in strength in one blanket and a slight increase in the other. It was noted during testing that the scrim pulled out relatively easily in some specimens and gave higher results because of "roping" or accumulation of scrim. This would account for higher values than originally obtained but does not indicate an increase in this mechanical property.

- The ply adhesion test shows the strength of CPE "strike-through" and scrim/CPE bond. The ply adhesion dropped an average of 24.6 percent for the blanket panels in test section coupon No. 3 reflecting a reduction in scrim/CPE bond and/or CPE polymer strength at the "strike-through."

- Bonded seam shear testing of both the thermally bonded factory seam methods showed the greatest change in mechanical properties. The bonded seam shear for the thermal seam methods dropped 54.5 percent as compared to the original unsaturated CPER.

- The bonded seam peel strength for the thermal dielectric and thermal hot air seams also deteriorated in strength by 27.4 and 22.3 percent, respectively. The adhesive seam peel showed a lower drop (10.7 percent) in peel strength whereas previous panels indicated little or no change in adhesive peel.

- Shear strength testing on the adhesive seams showed an average loss of 29.9 percent, also reflecting a weakening in tensile properties across the seam.

The loss in shear strength does not reflect actual seam interface failure (i.e., CPE to CPE bond using adhesive or thermal welding) but rather the polyester scrim pullout at the seam overlap. It is apparent that the scrim/CPE bond within the CPER laminate has deteriorated, and this weakening of bond is being reflected in loss in mechanical properties such as the Mullen hydrostatic, ply adhesion, and most notably the overlap seam shear strength. Because the specified overlaps resulted in only 25 mm (1 inch) of "scrim-to-scrim" bond (for the thermally welded bonds), the 1-inch overlap is the only contributing area for the seam shear values. The adhesive field seam required a 100-mm (4-inch) wide bonded area and thus had more "scrim-to-scrim" contact area which resulted in a less dramatic loss in tensile shear. In all cases of seam shear testing, the scrim pulled out of the overlaps relatively easily as compared to original unsaturated testing.

The water absorption of the CPER (15.6 percent) is probably the greatest contributing factor to the changes in the CPE polymer as reflected in the reduction in the ply adhesion and bonded seam shear values of the 1.14-mm (45-mil) CPER. The adhesion of the scrim to the CPE contributed to the original ply adhesion and shear mechanical properties and may also have deteriorated with time giving lower results on the aged CPER.

Analytical techniques using GC/MS (Gas Chromatography - Mass Spectrometry) were conducted on extracts obtained from CPER geomembrane specimens taken at the time of installation and after 36 months of exposure. The analyses indicate that some subtle changes in the chemical composition of the CPER have occurred. These changes may be related to degradation of the polymer and/or effects of water absorption.

The testing of sealed versus unsealed edges of CPER showed no major differences in mechanical properties due to anticipated water wicking through the exposed scrim. Changes in the CPER are primarily due to water absorption within the CPE sheet material and not through exposed scrim. These changes are not considered detrimental to the integrity of the Mt. Elbert CPER lining at this time.

Future Monitoring

In addition to the field instrumentation monitoring, additional test coupons No. 4 and 5 of the CPER will be removed and tested in September 1984 and 1985. After fulfilling the 5-year warranty monitoring of the 1.14-mm (45-mil) CPER in accordance with the original specifications, the long-term monitoring program will be continued with coupons or partial coupons taken every 2 or more years until all samples are retrieved. If, after 5 years of testing, little change is occurring in the membrane system, physical monitoring and testing of the coupons will be extended to year 2010.

REFERENCES

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(4) Frobel, R. K., "Quality Assurance Program for the Mt. Elbert Forebay Flexible Membrane Lining Installation," Proceedings, Symposium on Water Proofing Membranes for Dams, Paris, France, February, 1983.

(5) Frobel, R. K., "Design and Development of an Automated Hydrostatic Flexible Membrane Test Facility," USBR REC-ERC-80-9, 1981.

Table 2. - Mt. Elbert test section results, test section panel No. 3, after 36-months' exposure

Property	Blanket 570 panels 12-13 (thermal-hot air seam)			Blanket 2097 panels 27-28 (thermal-dielectric seam)		
	Original data	Aged data (range)	Percent change	Original data	Aged data (range)	Percent change
Weight gain	-	-	+15.6	-	-	+15.6
Mullen burst (kPa)	2997	2702 (2549-2756)	- 9.4	2811	2390 (2239-2480)	-14.9
Tear[a] (N)	355	300 (275-318)	-15.4	283	298 (282-318)	+ 5.3
Ply adhesion (N/m)	1505	1208 (1138-1225)	-19.7	1838	1295 (1225-1365)	-29.5
Bonded seam[b] shear (N)	1342	609 (587-627)	-54.6	1225	557 (485-592)	-54.5
Bonded seam peel (N/m)	5688	4130 (3990-4270)	-27.4	7368	5723 (5460-5950)	-22.3
Low temperature[c]	P	P	-	P	F	-
Adhesive seam[b] shear (N)	1339	948 (890-1099)	-29.2	1339	948 (890-1099)	-29.2
Adhesive seam peel (N/m)	6038	5390 (5163-5915)	-10.7	6038	5390 (5163-5915)	-10.7

Notes:

[a] Tear specimens. - Scrim sometimes pulled out due to delamination causing "roping" of scrim during test and, therefore, erratic test results.

[b] Shear seams. - Scrim pulls out of overlap relatively easily on the aged seams, whereas original seams failed in the parent material.

[c] Low temperature bend test at -40 °C (-40 °F) - P/F = pass/fail.

1 kPa = .145 lbf/in^2
1 N = .225 lbf
1 N/m = .0057 lbf/in

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The Special Problems Encountered in the Installation of a Geomembrane Liner at Prudhoe Bay, Alaska

Envision a climate where the "good summer weather" lasts approximately two weeks in August. Then envision a maximum temperature of 36°F with prevailing winds of 20-30 mph as the "good weather". You have just described Prudhoe Bay, Alaska, the northern end of the Alaska Pipeline.

The North Slope of Alaska presents many special problems for survival - one of which is the preservation of liquid water during the long Arctic winters. A shallow lake is the source of fresh water - and it freezes solid during the winter.

A gravel dike was pushed out into the lake to create a 250 feet x 500 feet rectangular pond. This pond was lined with a geomembrane liner - fabricated into two large "super-panels" to minimize field seaming.

Then hot water from the power plant was circulated under the ice to keep the rest of the pond water liquid through the winter.

Prudhoe Bay, on the North Slope of Alaska, is the oil drilling site and Northern terminus of the Alaskan Pipeline. It has one of the most inhospitable climates in the world. The temperature rises above freezing for only a few weeks in the summer, and the wind never stops. During the long winters, virtually everything freezes solid.

You may well ask, as we did, what possible need could there be for a pond liner in such a location? Actually, the climate produced the need. Fresh water for the inhabitants, and for the boilers and other equipment, comes from a large lake - Big Lake, by name. Although Big Lake has a very large surface area, it is quite shallow - some five to six feet deep - and consequently freezes solid in the winter, like everything else.

The question arose as to how to maintain liquid water for the men and equipment that must operate at Prudhoe Bay throughout the winter. The answer was to push gravel dikes out into the lake to partition off a pond that could be isolated from the lake. This pond could be covered with insulation and constant circulation of heated water from one of the boilers would maintain a reservoir of liquid water throughout the winter.

The pond needed an impermeable liner to keep the water inside from diffusing through the porous gravel dikes during the summer. However, the normal procedures for lining a pond could not be used at Prudhoe Bay. It would have to be installed during the short summer, when the lake was not frozen - but at that period there was no way to drain the pond for a conventional liner installation. Draining the pond would have meant draining the entire lake as the porous gravel dikes allowed free passage of the lake water. Also, the normal conditions for field installation and field seaming of a membrane liner never occurred at Prudhoe Bay.

The only possible solution was to pre-fabricate the liner in a single panel, and to install it while the pond was full. The initial pond was 250 feet by 250 feet, requiring more than 62,500 square feet of material fabricated into a single panel. This was larger than any panel fabricated before.

Three panels, 85 feet by 250 feet were fabricated, then joined together to create the "super panel". An inflatable tube was attached to one 250 foot side to aid in floating the liner into position. This panel was accordion folded in 10 foot widths and rolled up on a 6 inch steel pipe core. A custom-made skid held the 5 foot diameter roll which weighed almost seven tons.

The skid was air-lifted to meet the barges scheduled for Sea-lift that spring. These barges are brought in when the Bering Sea thaws, and must be unloaded and brought out within approximately two months, or they risk being trapped in the ice for the ensuing winter. All buildings, drill platforms, et cetera are pre-assembled and barged into Prudhoe Bay intact. Once the liner had arrived and been off-loaded to its proper site on the pond dike, installation could begin.

The first "Super Panel" performed as anticipated, following repairs caused by a submerged angle iron that was undetected during the initial installation. The following year, a second Super Panel was manufactured, fabricated and installed in a second pond adjacent to the first.

The final phase consisted of installing two new panels after deepening the pond and removing the center dike between the two ponds. The two new panels were joined together into one "super-super panel" with a field seam and re-deployed into the enlarged pond. The procedure for this final phase went as follows:

The first of the two super panels is placed at one corner of the pond at a predesignated position. It is then unrolled along the top of the 250 foot dike. An attached inflatable tube is filled with air and a steel pipe is attached. Cables from pneumatic winches are pulled across the 500 foot length of the pond and attached to the steel pipe.

This winding operation moves along slowly until 150 feet of the first panel is pulled into the pond. Ropes attached to each side of the panel provide a small amount of pulling power, but mainly keep the edges of the hypalon from sliding down into the water. The pulling continues across the pond until the last strip from the first panel is the only remaining strip left on the dike. The second panel is then positioned in the exact same position as the first. It is unrolled and the first strip of panel number 2 is overlapped onto the last strip of panel number 1.

The two strips are then seamed together. The seaming is done with a bodied solvent adhesive. The solvent solvates both surfaces, and once the solvent evaporates it leaves a rubber-to-rubber weld impossible to break. Due to the cold weather, the solvent will not readily flash off, and the high winds keep a normal hot air gun's heated air from effectively warming the surface. Special wooden frames with clear plastic covers were built to protect a section of seaming area. These shelters were big enough for the seaming crew to work in. A space heater supplied the heat inside the shelter. As the crew seamed from one end of the shelter to the other, the shelter was moved to seam the next section.

After the seaming is completed, the pulling of the panel can continue. Eventually more ropes are attached to each side of the panel, pulling and keeping the edges out of the water. Once the panel reaches the far side of the pond, it is centered as evenly as possible.

Now, the panel had to be sunk to the bottom. Therefore water was pumped from Big Lake onto the top of the liner so the water could begin to sink it. Because the construction of the pond was well-rounded gravel and built in the middle of Big Lake, the water is able to enter and escape through the porous dikes. As the liner began to sink, the rate of pumping was increased, and as the liner sank further below the surface, the demand for more and more liner caused the outside liner edges to move into the pond. The liner was filled until the water level reached twice the depth of Big Lake. The liner had now completely settled on the bottom and could be anchored into its final position.

To protect the liner from ice damage, gravel had to be placed on the slopes of the pond. The pond was now ready for the insulation and freezing weather to begin the continuous re-circulation of heated water. Survival of men and machines at Prudhoe Bay was assured throughout the Arctic winters.

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Application of Geomembranes in Northern Canada

The feasibility of various types of geomembrane liners in a northern environment was investigated by summarizing previous investigations, and evaluating more recent installations in the north. The most significant conclusions from these evaluations are the following: Prefabricated liners which must be jointed in the field are limited in their applications to a large degree by the seaming process. Many damages to prefabricated liners occur during installation due to lack of suitable bedding material in the north. All permanent liner installations should be protected by a suitable overburden material, since exposed membranes invariably become damaged.

INTRODUCTION

The development of the Canadian North has resulted in the need for storage facilities of various sizes and for various purposes such as petroleum product tank containments, drinking water reservoirs, sewage lagoons and mine tailing containments. Permafrost, the harsh climate and the general lack of suitable fine-grained borrow material for impervious fill have created problems for adequate design, construction and maintenance of these containments.

Since the early 1970's the application of geomembranes has become more and more common for lining containments in the north. Satisfactory performance of petroleum product containments was particularly important due to their immediate impact on the environment. Thus, existing facilities were inspected since 1974 at several instances by the Environmental Protection Service (EPS) of Canada, and the findings were summarized in reports, e.g. (1), (2). In addition the applicability of alternative liners for petroleum storage areas in the north was investigated and the findings summarized in reports (3), (4), (5). Information about the performance of geomembranes for other applications, such as drinking water reservoirs, sewage lagoons or mine tailings containments is less complete. Thus, if no negative performance reports have been obtained, positive performance can be assumed. The locations of the installations discussed are shown on the site plans on Figs. No. 1 and 2.

PERFORMANCE OF GEOMEMBRANE LINER INSTALLATIONS FOR PETROLEUM PRODUCT CONTAINMENT

The first performance records of petroluem containments in the north were reported in 1974 and 1975 by E.P.S. (1), (2). These reports referred to inspection tours undertaken to various sites in the western Arctic. The locations of the liner installations inspected are shown on the site plan on Fig. 2. Details of the installations and respective comments are summarized on Figure 3.

The earliest liner installations were Oil PVC (30 mil) liners at sites 1, 2 and 3. These liners had been installed only in the dikes on a gravel base, being anchored along the reservoir floor into relatively impermeable permafrost at sites 1 and 2 or into a substrate clay at site 3. Field seams for these installations were undertaken at temperatures above +10°C, and appeared successful. It was reported, however, that during handling and unfolding of the membranes at temperatures as high as +5°C brittle fractures were inflicted to the membranes even though laboratory tests had indicated that the material remained ductile above -18°C. Damages to liners covered by protective overburden could almost exclusively be related to careless installation practices, which had resulted in brittle fractures or puncturing of the membranes from sharp edged rocks in inadequately prepared gravel bedding. At locations where the liners had been exposed, failures of the oil PVC films were common, almost exclusively caused by brittle fractures.

The liner at site 4 was installed on a gravel base and protected by a gravel layer 150 to 500 mm thick. Although pinholes and several small cuts were detected below the overburden layer, there were no major punctures. Minor holes could be easily mended using PVC cement and patches of lining material.

A 10 mil polyethylene liner installation was inspected at site 5. The membrane was placed on a carefully prepared base consisting of 50 mm of polyurethane foam over a 800 to 1000 mm thick gravel zone. The foam served to insulate the underlying permafrost and provided a smooth bedding for the polyethylene liner, which was installed during the summer. Approximately 300 to 500 mm of fine gravel overburden was emplaced over the liner. No punctures were observed below the surface cover, clearly indicating that careful preparation of the bedding and substantial cover of fine overburden had well protected a rather thin membrane.

Reinforced laminated polyethylene liner (6 mil) had been installed at sites 6 and 7. At site 6 the liner was underlain by polyurethane foamed on sacking to provide thermal protection for the underlying ice-rich silt and to form a protective bedding for the membrane. The liner was partly covered by polyurethane blankets and gravel and partly was left unprotected. No damages could be found where the liner was protected. There were numerous tears where the liner had been exposed to sunlight. High puncture resistance was evidenced for the liner. The polyurethane foam was successful in preserving the permafrost up to the ground surface, thus providing protection against thaw instability. Good low temperature ductility was reported for the polyethylene liner, as the liner was installed during the winter at temperatures around -30°C. At site 7 sawdust had been used to cushion the membrane from coarse bedding and overburden. Again very little evidence of mechanical damages was apparent.

A urethane liner installation (20 mil) was visited at site 8. The liner was sandwiched between two layers of sawdust, each approximately 100 mm thick to protect the membrane from a fine sandy gravel base and gravel overburden 150 to 500 mm in thickness. No deficiencies were observed along the floor of the reservoir in this installation. However, at one location in the dike slope where the liner had been exposed due to sloughing of the protective overburden, the membrane was damaged.

Fibre-reinforced polyurethane (30 mil) had been placed at site 9 over a gravel foundation and was left unprotected. The sheets had previously been used at other locations over several years. A number of tears were evident in the exposed sheets; nevertheless, the age of the sheets and their frequent re-use indicated high strength of this material even at low temperatures.

Unreinforced polyurethane liner (20 mil) was used at a rig site at location 10. Here the liner was placed loosely over the gravel floor and the dike slopes of the fuel storage area. In spite of a number of minor defects the liner exhibited overall good integrity. No brittle fractures were apparent during the inspection, even though the liner had been installed and unfolded at temperatures of -35°C.

Subsequent to the above discussed installations many more petroleum product containments were built. Two examples for which brittle fracture were reported are in Resolute Bay, N.W.T. (location 11) and in Repulse Bay, N.W.T. (location 12). At Resolute Bay an exposed nonreinforced Hypalon liner (30 mil) was applied to a containment for a tankfarm. In the berms of the dike areas, where the liner was left unprotected failure due to brittle fracture was reported. At Repulse Bay a PVC membrane (30 mil) was installed below and adjacent new storage tanks. Already during construction it was noticed that brittle fracture had occurred, where the membrane was left exposed for one winter.

DESCRIPTION OF PREFABRICATED GEOMEMBRANE LINERS FOR APPLICATIONS SUCH AS DRINKING WATER RESERVOIRS, BRINE PONDS, TAILINGS DAMS

A 120 Million U.S. gallon pretreatment water reservoir was constructed in 1977 at Pine Point, N.W.T. The open reservoir is located along a sand and gravel ridge and was lined with a nonreinforced CPE (30 mil) membrane to prevent exfiltration through the bottom and benched dikes. The membrane was placed over a carefully prepared sand bedding which had been raked to remove any angular gravel particles. The membrane along the reservoir bottom, which would always be water covered, was left exposed. On the side dikes the membrane was protected by a 160 mm sand layer overlain by approximately 1000 mm of gravel material. This thick protective layer was provided to prevent mechanical damage by ice and wave action and to permit tracked vehicle access down the side slopes. The liner was field seamed by solvent welding in September at temperatures between 0°C and 5°C. Portable enclosures were used to shelter the seaming operation from rain. Heat was applied to the membrane prior to seaming. No damage has been reported in seven years of service.

A similar reservoir was constructed in 1976 in Eskimo Point (6). This reservoir used a nonreinforced Hypalon (30 mil) membrane, 600 mm of sand bedding and a sand and gravel protection layer of varying thickness. This membrane has been in service for approximately 8 years without reported problems.

In the late 1970's a nonreinforced Hypalon membrane (30 mil) was used for a water storage reservoir at Pangnirtung, N.W.T. Initially it was reported that the membrane was not covered. Plans are on the way for replacement of the facility.

In a more recent installation a high-density-polyethylene liner (60 mil) was applied for a brine pond near Edmonton, Alberta (7). The liner was placed on sand and was left unprotected. Field joints were connected by a special welding process, which however, could be undertaken successfully only during the three warmest summer months, when ground temperatures were above 14°C. The pond has been in operation since 1982 and the unprotected HDPE liner so far has not exhibited leaks or damages.

At a mine site in Yellowknife a nonreinforced Hypalon liner (30 mil) was installed in 1981 as an upstream impervious blanket of a small tailings pond retention dam based on highly compressible soil (8). Thus, the liner was designed to tolerate considerable differential movements between rigid abutments and soft dam portions. The liner was emplaced on silt tailings and was cushioned by the same material against a protective cover of rock fill. Field seaming was successfully undertaken at temperatures between 0°C and +5°C using a one-compound cold-setting adhesive. A summary of the characteristic data of the installations described above is given in Figure 4.

PERFORMANCE OF IN-PLACE-FABRICATED GEOMEMBRANE LINERS

The applicability of in-place fabricated liners as an alternate to pre-fabricated membranes was investigated for petroleum product storage areas by E.P.S. and discussed in various studies (3), (4), (5). From an initial review of spray-on or grouting sealants it was indicated, that bentonite sulphur and spray-on-liners would be the most promising alternatives for northern applications (3). Thus these liner types were subsequently investigated. The main results of these investigations are summarized on Figure 5.

Spray-On Liners

The study on spray-on liners (4) was undertaken in two phases. In a laboratory investigation the most promising systems were evaluated and subsequently tested in a field installation at the site of a tank farm in Yellowknife, N.W.T. The materials installed were: Carboline, Elastuff 504, and Urethane Foam. The polymer materials were sprayed on Tufton backing which had been placed on sand or silt bedding. The urethane foam was sprayed directly onto the ground surface.

Various lap joints were tried out for the polymer-Tufton liner, as shown on Figure 6. The most promising joint type is using adhesive between the Tufton layers.

Urethan foam could be sprayed directly on rough ground surface, but appeared to be susceptible to mechanical damages due to its relatively low strength. Urethane foam is also susceptible to attack by ultraviolet light, thus requiring some coating to limit exposure of the foam surface to sunlight. When carboline covering was used, it became apparent that carboline coating on urethane foam attracts ravens, which peck holes through the carboline into the foam.

During liner installation it was noticed that spray-on elastomer and polyurethane foam liner systems require a high degree of skill in application. Furthermore installation is possible only in summer at temperatures above 15°C and at dry conditions. Special fire precautions must be effected during applications because of the volatile solvents involved. It is important to note that all equipment and material necessary for application of spray-on liners and urethane foams is separable into small components and therefore is readily air-transportable to remote sites.

Bentonite and sulphur liners were installed at the same test site in Yellowknife as the previous liners (5). For both liner types it was noticed that the installation was rather labour-intensive and required considerably larger equipment and material weights than the spray-on liner types investigated. For installation the same conditions were required: temperatures above 15°C and no rain.

The test liners were left unprotected and inspected one year later. While the spray-on and urethane foam liners showed good performance, the sulphur liner was affected by substantial cracking, which reduced considerably the retention capability of the liner. For the bentonite liner it was indicated that desiccation of the layer was not a problem, whereas the opposite situation of excessively moist bentonite appeared more of concern, because the liner became very susceptible to mechanical damages.

DISCUSSION

Geomembranes are among the most economical materials which are applicable for impermeable liners in northern Canada, however, many types are not suitable in the arctic environment.

Joining of plastic sheets and patching of tears is a particular problem. Heat sealing and to some degree also solvent welding processes require reasonable warm temperatures to be successful. This requirement may limit e.g. the application of HDPE liners for the high arctic, in spite of its good cold weather properties, high ductility and general sturdiness.

Liners should preferably exhibit good low temperature ductility. Even if installation is performed during summer, ambient temperatures may be encountered which are below the ductile-brittle transition temperature of some membrane types. In addition good low temperature ductility reduces the probability of damages if portions of the membrane become exposed.

High puncture strength and tear strength as well as high ductility are desirable properties for membranes installed in the north, because good base preparation cannot be relied on. There is a general decrease in quality of work once temperature drop to more uncomfortable levels.

Careful preparation of membrane foundation and adequate protection by overburden is very important. Preparation of the material contacting the membrane can dominate the ultimate effectiveness of the installation more than individual membrane properties. It was found that even weak films, such as 10 mil polyethylene with poor puncture strength, low resistance to weathering and only fair ductility can perform satisfactorily if properly cushioned against coarse and sharp edged aggregate. On the other hand even materials with high puncture strength and good ductility will be damaged if placed in direct contact with sharp-edged aggregate.

Adequate cushioning materials are sands or silts, as well as saw-dust and polyurethane foam. The latter material also provides thermal insulation to protect underlying permafrost from thawing and thus helps to preserve the stability of the foundation base of the membrane.

For permanent liners or liners which are to be in place over an extended time period, an adequate cover of protective overburden is essential. It was found that in all cases where a membrane was exposed damages had occurred.

For temporary installations exposed liners may be applicable. It was indicated however, that only membranes were suitable which have good low temperature ductility, particularly as the membranes will be retrieved mostly during the winter months. Polyurethane (nonreinforced as well as fibre-reinforced) and prestressed laminated polyethylene appeared to show very promising properties in this respect. It was found essential, too, that exposed liners are anchored adequately to provide protection from damage by wind.

The application of spray-on-liners and urethane foams is characterized by relatively small equipment sizes and material weights. This, combined with good cold weather resistance would favour its application in remote northern areas. Foamed urethane liner could also be applied on rough substrate. However, because spray-on-liners can be applied only during the summer in dry conditions at temperatures above +15°C its potential for application in the north is limited. These reasons may explain that so far, at least to the authors' knowledge, no major spray-on-liner installation has occured in northern Canada.

Sulphur liners do not appear suitable for application in the north as they crack seriously during cold winter conditions.

Bentonite liners would be applicable for northern regions if protected by overburden against mechanical damage. The disadvantage of high labour intensity during installation, may be eliminated if prefabricated bentonite sheets (e.g. "Volclay") would be used. Because of the small thickness of those prefabricated sheets a careful base preparation would be required.

CONCLUSIONS AND RECOMMENDATIONS

In addition to the basic requirements of a geomembrane, such as chemical stability in contact with soil or liquid, or certain mechanical strength values, additional factors must be considered for application in the north.

Prefabricated liners, which must be jointed in the field are limited in their application to a large degree by the seaming process. Successful seams require in some instances certain minimum air and ground temperatures which are often not prevalent in the north.

Many damages to prefabricated liners occur during installation due to the lack of suitable granular material and due to careless handling practice, which must be considered a factor in the north.

Ideally prefabricated membranes and spray-on-liners should be placed between two cushioning layers of bedding material. Foamed urethane, however can be placed immediately on rough substrate.

Spray-on-liners require warm and dry weather conditions as well as experienced personnel for their installation. These conditions will limit their application in the north, in spite of logistic advantages like easy air-transportability.

All permanent liner installations (pre-fabricated liners, spray-on-liners, bentonite liners) should be protected by a suitable overburden material, since exposed membranes invariably become damaged.

Permafrost below a reservoir must be protected from thawing by properly designed thermal insulation to preserve the stability of the foundation base. The insulation layer may at the same time act as a cushioning material for the membrane.

All types of geomembranes should possess good low-temperature ductility even if they will be installed during the summer and will be protected by a surface cover.

Good low-temperature ductility is essential for temporary liner installations, as they frequently will be left unprotected and in most cases will be retrieved during the winter months.

For temporary liner installations properties such as high puncture resistance, high tear strength, low temperature ductility, resistance to weathering are essential. Exposed liners must be adequately anchored to reduce damage caused by wind.

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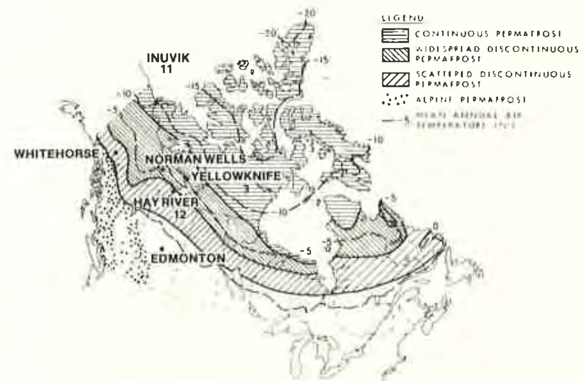


FIG. 1: MAP OF CANADA SHOWING EXTENT OF PERMAFROST AND LOCATIONS OF RESERVOIRS LINED WITH GEOMEMBRANES.

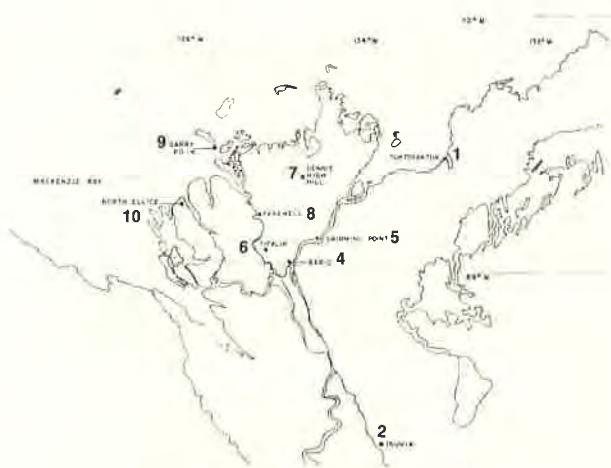


FIG. 2: SITE PLAN OF MACKENZIE DELTA WITH LOCATIONS OF PETROLEUM PRODUCT CONTAINMENTS INSPECTED BY E.P.S. IN 1974/75.

FIGURE 3: PREFABRICATED GEOMEMBRANES FOR PETROLEUM PRODUCT CONTAINMENT IN THE NORTH

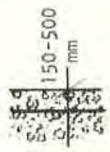
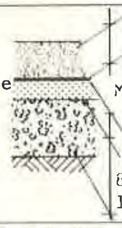
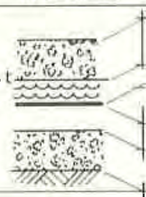
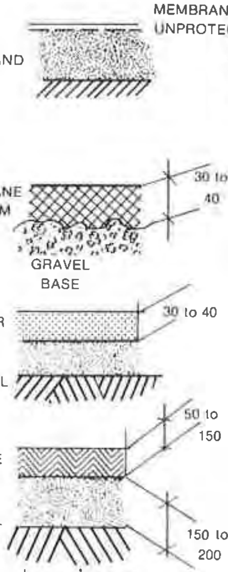
Location (see Fig.1,2)	Geomembrane Type	Arrangement of Membrane	Installation	Performance
1, 2, 3, 4	Oil PVC (30 mil)		Installation at Temp. T = +10°C; Brittle failures during handling at Temp. T = +5° (1–3) Installation at Temp. T = +20°C (4)	In protected portions: Defects only due to careless handling during installation. In exposed portions: (1–3) Brittle failures (4)
5	Polyethylene (10 mil)		Installation during summer	In protected portions: no defects
6, 7	Reinforced laminated polyethylene (6 mil)		Installation during winter at Temp. T = 30°C; no brittle failures	In protected portions: no damage; In exposed portions: numerous tears
8	Urethane (20 mil)			
9	Fibre reinforced polyurethane (30 mil)		-Temporary liner; Installation during winter at -35°C (9–10)	Exposed liner - re-used material: some tears, but generally good performance at low temperature
10	Nonreinforced polyurethane (20 mil)			Exposed liner - minor defect - general good integrity at low temperature
11	Nonreinforced Hypalon (30 mil)			Exposed portions: brittle fracture
12	PVC (30 mil)			Winter exposure during construction caused brittle fracture

FIG. 5: IN-PLACE-FABRICATED GEOMEMBRANES AT TEST-SITE IN YELLOWKNIFE (E.P.S., 1977, 1978)

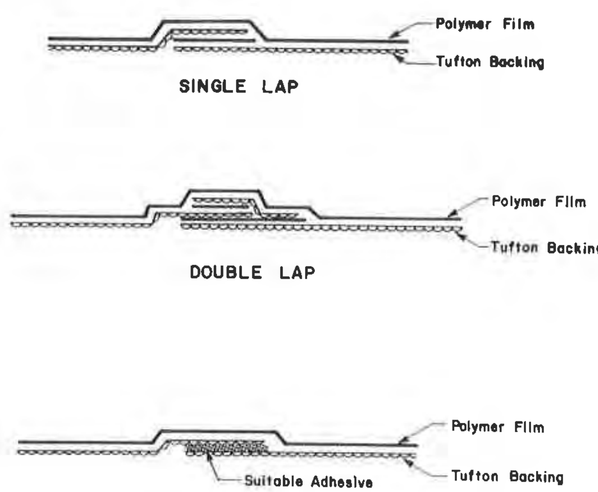
	Liner type	Arrangement of Liner	Comments on Installation	Comments on Performance
Spray-on Liner on Tufton backing	Carboline		-Installation at Temp T > 15°C and at dry cond.	-good resistance to weathering and cold temperatures after 1 year
	Elastuff 504		-Equipment and material required is easily air-transportable	-Urethane foam must be protected against sunlight and damage by birds
	Urethane foam		-Highly skilled personnel required	
	Processed Sulphur		-Installation at Temp T > 15°C and at dry cond.	-serious cracking due to cold weather after 1 year
	Processed Bentonite		-large equipment and large material volumes needed -very labour intensive	-good resistance if protected with layer of sand and gravel

NOTE: Dimensions shown in mm.

FIG. 4: PREFABRICATED GEOMEMBRANES IN THE NORTH

Location (year of inst)	Purpose	Geomembrane Type	Arrangement of Membrane	Installation	Performance
Edmonton, Alta (1982)	Brine Pond	HDPE (60 mil)		Placed in Aug/Sept Field welded in subsequent summer at ground temp T 14°C	no damage after 2 years
Yellowknife, N.W.T (1981)	Water Retention Dam	Hypalon unreinforced (30 mil)		Installed in Sept. Field seamed at Temp. T=0°C to 5°C	no damages after 3 years
Eskimo Point, N.W.T (1976)	Raw Water Reservoir	Hypalon non-reinforced (30 mil)		Installed during summer	no negative reports after 8 years
Pine Point, N.W.T. (1977)	Raw Water Reservoir	CPE non-reinforced (30 mil)		Placed in Sept. at temp. T=0 to +5°C Field seamed using portable heated shelters with solvent	no damage report after 7 years
Pangnirtung (late 1970's)	Drink water storage	Hypalon non-reinforced (30 mil)			Replacement after appr. 5 years

NOTE: Dimensions shown in mm.



DETAILS OF LAP JOINTS

FIG. 6: EPS-TEST SECTION YELLOWKNIFE DETAILS OF LAP JOINTS FOR SPRAY-ON-LINER

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Case Histories of Geotextile/Geomembrane Systems in Ponds, Reservoirs, and Landfills

Geotextile fabrics are increasingly being used to offer protection to geomembranes in ponds, reservoirs, and landfills. Thick, nonwoven, needlepunched, geotextile fabrics provide puncture protection and abrasion resistance when placed directly beneath and/or above geomembranes. Since the geotextile is a highly porous, permeable fabric, it also has the ability to drain off liquids and vent gases to protect the geomembrane from potential uplift or rupture. This paper briefly describes case histories of three recent U.S. projects which comprise geotextile/geomembrane systems in ponds, reservoirs, and landfills.

INTRODUCTION

Case Histories of Geotextile/Geomembrane Systems in Ponds, Reservoirs, and Landfills

Conventional geomembrane installations in pond and landfill construction have often utilized a sand or soil interlayer directly beneath and/or above the geomembrane to provide protection against puncture and abrasion from rocks in the subgrade or landfill solid wastes. Another purpose of the sand or soil interlayer has been to vent gases from within the subgrade to avoid potential uplift or rupture of the geomembrane.

Recent field and laboratory tests have revealed that thick, nonwoven, needlepunched geotextile fabrics in a weight range of 400 to 600 g/m^2 may be substituted for sand or soil to effectively protect geomembranes in large pond and landfill construction. These particular geotextile fabrics provide the optimum combination of strength, durability, and lateral transmissivity to perform satisfactorily as an underlining/overlining in pond or landfill construction (1).

Geotextiles offer significant protection to geomembranes as evidenced by over 10 years of European experience and recent U.S. experience on ponds, reservoirs, and landfills. This paper briefly describes case histories of three recent U.S. projects which comprise geotextile/geomembrane systems in ponds, reservoirs, and landfills.

Case History No. 1: Construction of New Uranium Tailings Ponds for Mineral Company's Effluent Storage and Evaporation System (Central New Mexico)

In 1980, Anaconda Minerals Company constructed three ponds encompassing 0.29 Km^2 (71 acres) to provide the necessary storage for leachate evaporation at their Bluewater uranium mill located in central New Mexico. The process of refining uranium ore requires using sulfuric acid as a leaching agent. Residual material stored in these ponds contained both spent sulfuric acid, plus some uranium in solution. The ponds effectively contained the liquid, thereby protecting the deep groundwater regime. Additional amounts of uranium were recovered through recycling of a concentrated solution within the ponds following significant evaporation.

Various polyester woven scrim reinforcements were considered for the geomembrane to improve its performance and durability over the minimum 20-year design service life of the project. The geomembrane chosen for this project consisted of a three-ply laminated industrial grade reinforced chlorosulphonated polyethylene, or Hypalon, as produced by E. I. duPont de Nemours & Company. Two 0.38 mm (15 mil) layers encapsulated a single layer of high-strength polyester woven fabric scrim to produce a single 0.91 mm (36 mil) laminated geomembrane. This particular geomembrane configuration was selected over other alternatives that were considered in different thicknesses for their chemical resistance, impermeability, resistance to ultraviolet light, strength, flexibility, durability, repairability, and cost.

A 400 g/m^2, 2.79 mm (110 mil) nonwoven, needlepunched, polypropylene, geotextile fabric was used as an underlining beneath the geomembrane on both the bottom and 3:1 sideslopes of all three leachate evaporation ponds. Fibretex Grade 400 geotextile fabric (2) marketed by Crown Zellerbach Corporation was selected and installed on this project. The geotextile provided the necessary cushioning protection against puncture or abrasion of the geomembrane from rocks in the subgrade. Another primary function of the geotextile was to act as an effective conduit to safely vent gases trapped within the subgrade to prevent the formation of gas bubbles or "whales" beneath the geomembrane which could cause a rupture.

Embankment construction was specified in lifts not exceeding 305 mm (12 inches) and compacted to 95% of maximum dry density per ASTM D-1557, Method D. The dikes were compacted oversize and then trimmed to proper line and grade. The bottom of each pond was graded at a 2.5% slope toward a center low line and compacted to 90% of maximum dry density per ASTM D-1557, Method D. The 2.5% slope to a center low line in each pond was intended to facilitate the upward migration of gases through the geotextile underlining for escape to the atmosphere through vents situated on 7.62 m (25-foot) centers along the top of the embankment. Test data provided by the geotextile manufacturer assured the owner that gas could be transmitted laterally through the fabric at a sufficient rate under substantial water pressure so as to avoid any potential gas uplift problem.

The decision to use a geotextile fabric underlining as a substitute for conventional construction materials had an economic impact on the project. The geotextile underlining was installed at approximately a 50% cost savings, as compared to the estimated cost of providing a 76.2 mm (3-inch) layer of pea gravel spread onto the site, after transporting it 145 Km (90 miles) from its nearest source near Albuquerque, New Mexico (3).

Case History No. 2: Rehabilitation of Existing Reservoir for City's Potable Water Supply (Northern Maryland)

In 1983, the City of Cumberland, Maryland, completed rehabilitation of its Ridgedale Reservoir. The 28.39 million liter (7.5 million gallon) capacity reservoir is a concrete-lined, earthen embankment impoundment in the hills on the west side of the city. It had been in constant use as an uncovered, finished potable water reservoir from 1917 until 1975, when surface drainage contamination and excessive leakage necessitated that it be taken out of service. However, subsequent engineering studies of the city's water system dictated that renewed operation of the Ridgedale Reservoir was essential to maintain adequate water supply and pressure during high demand conditions, such as a major fire.

The 65-year-old wedge-shaped structure was badly deteriorated by 1982 due to its 7-year period of inactivity and exposure to the elements. At this time, an engineering study was performed which recommended complete rehabilitation of the reservoir, including concrete restoration of the structure, installation of a fixed geomembrane liner on the floor and sideslopes of the impoundment, and provision of a floating geomembrane cover. These innovative improvements were believed to be an economically feasible solution for rehabilitation of the reservoir over the minimum 20-year design service life of the project.

An industrial grade, reinforced, chlorosulphonated polyethylene, or Hypalon, was selected for the fixed geomembrane liner and the floating geomembrane cover. Two layers encapsulated a polyester reinforcing fabric to produce a single 1.14 mm (45 mil) geomembrane. It was necessary that the finished product exhibit high-strength for resistance against tearing, weathering, and aging. It was decided that the fixed geomembrane liner was to be black, while the floating geomembrane cover would be green.

A 400 g/m^2, 2.79 mm (110 mil) nonwoven, needle-punched, polypropylene, geotextile fabric was placed between the fixed geomembrane liner and the concrete sideslopes of the structure. Fibretex Grade 400 geotextile fabric was selected for use. The geotextile provided three major functions: (1) abrasion protection of the geomembrane against the rough concrete sideslopes; (2) a means of safely venting gases trapped within the subgrade to the atmosphere; and (3) a means of adequately draining surface water or ground water that might seep from cracks or seams in the slabs to the underdrain system beneath the floor of the reservoir (4).

Use of sand as a cushioning device and gas/liquid transport media was questionable because of the relatively steep 2:1 sideslopes of the structure. Consequently, the decision to use a geotextile fabric in lieu of sand for this situation provided an economical alternative solution. An added benefit of using the geotextile was that it provided a clean working surface, free of contamination, for effective heat seaming of the individual liner panels.

Case History No. 3: Expansion of Existing Regional Solid Waste Landfill (Eastern Pennsylvania)

An expansion in 1983 of the existing Colebrookdale Landfill near Shanesville, Pennsylvania, incorporated the installation of a 0.76 mm (30 mil) polyvinyl chloride (PVC) geomembrane along the bottom and sideslopes of an expanded landfill section. The purpose of the geomembrane was to contain all leachate within the expanded landfill section to allow it to be conveyed through a plastic perforated pipe network to holding tanks at the perimeter of the landfill.

Three options were considered by the design engineering firm and owner of the facility to ensure maximum protection against puncture of the geomembrane from the landfill solid wastes. The first option specified the placement of river sand whose small and uniform particle size would not damage the geomembrane. However, it was decided that implementation of this option would be extremely expensive because of the need to import the offsite river sand for this project. The second option required placement of native soil which had been sifted through a small screen. Although carrying out this option appeared economically feasible, the possibility existed that sharp rocks as small as 6.35 mm (1/4-inch) diameter might come into contact with and damage the geomembrane. The third alternative was to install a thick, nonwoven, needlepunched, geotextile fabric directly on top of the geomembrane.

The owner selected the use of the geotextile as an overliner, after carefully evaluating the associated costs and benefits of the three options. Fibretex Grade 400 geotextile fabric was selected to serve as the overliner. A total of approximately 32,500 m^2 of geotextile fabric was used to effectively protect the geomembrane. The geotextile was placed directly on top of the geomembrane, with 457 mm (18 inches) of native material placed over the fabric for additional puncture protection. A layer of 76.2 mm (3 inches) of river sand was placed beneath the geomembrane, as well, to provide puncture protection on its bottom side.

Use of a geotextile overliner provided four major functions for the Colebrookdale Landfill expansion. The geotextile overliner: (1) cushioned the geomembrane, thereby protecting it from puncture or abrasion from sharp rocks in the landfill solid wastes; (2) reinforced the geomembrane by spreading static loads and spanning small cracks and voids; (3) provided a lateral conduit for release of trapped gases to the atmosphere that otherwise might build up beneath the landfill; and (4) assisted in the conveyance of leachate to the plastic perforated pipe drain network.

The geotextile fabric proved especially conducive to the particular application at Colebrookdale Landfill due to the liquid leachate problem and the topography comprised of rolling hills and steep slopes. Since the geotextile is manufactured from pure polypropylene, it has no additives to leach out or react with the geomembrane or soil. Unlike sand or gravel, the fabric will not slide down slopes or disappear down small voids and cracks. The geotextile is also inert to oils, chemical solvents, acids, and all but the strongest alkalis (5).

SUMMARY

Geotextile fabrics have been increasingly used to protect geomembranes in ponds, reservoirs, and landfills. This paper briefly describes case histories of three recent U.S. projects which comprise geotextile/geomembrane systems for these types of installations.

It has been demonstrated that thick, nonwoven, needlepunched geotextiles in a weight range of 400 to 600 g/m^2 provide the optimum combination of strength, durability, and lateral transmissivity to adequately protect geomembranes. These types of geotextiles provide adequate cushioning to protect against puncture and abrasion and effectively drain off liquids and vent gases to protect geomembranes from potential uplift or rupture.

Commercial use of geotextile/geomembrane systems in ponds, reservoirs, and landfills may be expanded successfully if care is exercised in the geotextile/ geomembrane system selection.

ACKNOWLEDGEMENTS

Appreciation is extended to Mr. George Cain of LS/CS, Inc. (Alexandria, Virginia) for providing background information on the Ridgedale Reservoir project. Mr. Cain's firm installed the geotextile/ geomembrane system for the rehabilitation of this reservoir.

Appreciation is also extended to Mr. Dick Welch of R.R.M. Corporation (Boyertown, Pennsylvania) for providing insight on the Colebrookdale Landfill project. Mr. Welch is in charge of the day-to-day operations of this landfill.

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Selected Aspects of Dimensioning Geomembranes for Groundwater Protection Applications

Increasing demands are being made today for technical solutions in the field of ground water protection, as further environmental contamination must be avoided.

There is keen competition between geomembranes of plastics and minerals, although it is difficult to compare these systems and to apply the same criteria to them. This is illustrated even by their thickness differences of 2 to 3 magnitudes (m to mm). Moreover, the historical base of experience is distinctly longer for mineral liners.

It is, however, this author's opinion that technically sound solutions can be attained with plastic goemembranes when material selection and dimensioning are done carefully and when construction of the lining system is matched to the environment. The purpose of this paper is to show that geomembranes can be dimensioned for selected loading conditions when short-term and long-term loadings (installation and operating conditions) are considered separately.

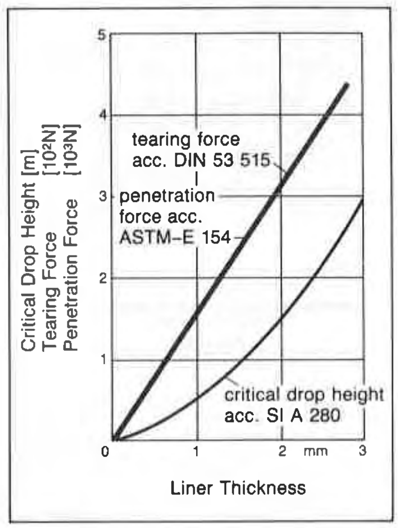
CONSIDERATION OF INSTALLATION CONDITIONS

Short-term loadings occur during installation. Geomembranes must be of such a condition that they will resist these mechanical loadings which are often uncontrollable. The determination of exact design loads is not possible. One can, however, estimate a material's behavior against these loadings on the basis of selected tests, such as testing for:

- tear resistance
- penetration resistance
- puncture resistance

Although these properties are specific for and unique to the material involved, they are largely influenced by the thickness of the geomembrane. The condition of geomembrane dependence on thickness can be linear or super-proportional, as is shown in Figure 1 for geomembranes made from HDPE. In this case it means that - by increasing the thickness - the liner resistance to these forces can be improved.

The uncertainty of determining a precise loading capacity is answered only by an adequate minimum thickness, as is already specified in several German guidelines (1) (2) in the fields of ground water protection and water construction. Experience with projects already carried out in Europe and elsewhere has shown that these minimum thicknesses should be established at more than 2.0 mm ; it is assumed that the material in question also has good mechanical properties. This measure can be seen as "constructive thickness", a term that is used in other fields of construction engineering where



Selected mechanical properties of geomembrane (HDPE) as a function of thickness

FIGURE 1

one must take into account those loading conditions which cannot be predicted.

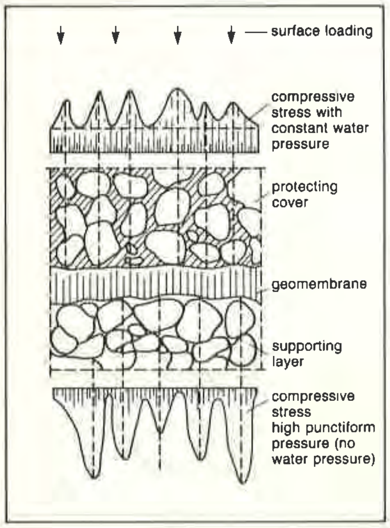
CONSIDERATION OF LONG-TERM LOADINGS DURING OPERATION

It is known today that geomembranes must not only perform within an engineered system, but must also be able to absorb long-term mechanical loadings (3). These mechanical loadings are most often the result of the weight of the basin contents and/or differential settlements of the subgrade. Settling of basin contents may involve constant or permanently increasing loadings, which lead to constant stresses and thereby to growing deformation of the material. In the case of differential settlement, constant or permanently increasing deformations are involved, but will likely result in a relaxation of the stresses with time. Geomembranes can be subjected in particular to compressive, shearing and tensile stresses.

COMPRESSIVE STRESSES

Compressive forces occur in bottom and embankment areas from loadings which may easily reach 100 Mp/sqm, for instance from dumping heights of 50 m with density of the dumped material of 2 Mp/m^3. Such a loading will result in a compressive stress of 1 N/mm^2.

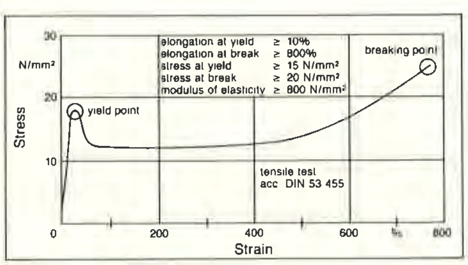
With adequate design of the entire lining system, this loading is distributed evenly and does not cause punctiform overloadings. Supporting and protective layers have to be coordinated with the grain shape, the maximum grain and the grading curve, thus minimizing local stress concentrations, as is shown in Figure 2.



Loading of geomembrane by punctiform and uniform pressure

FIGURE 2

Regarding the material loadings, a compressive stress of 1 N/mm^2 for HDPE, for example, is far below the material's admissible loadings. It is this author's opinion that this material is therefore preferably employed in ground water protection measures. One can assume that the compressive deformation behavior within the quasi-linear range is equal to a material's stress-strain behavior. The stress-strain behavior of a medium density HDPE geomembrane is illustrated in Figure 3. This diagram shows that a stress of 1 N/mm^2 corresponds with the factor 15 and is below the material's yield point.



Stress-strain diagram of polyethylene (medium density)

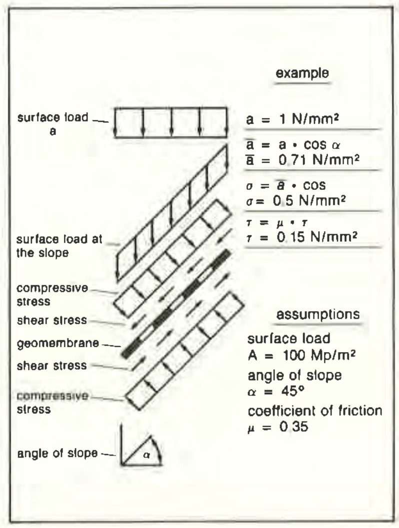
FIGURE 3

SHEARING STRESSES

Shearing forces typically occur when loadings in the embankment area, due, for example, to settlements within the waste body, move against the lining membrane. To avoid unacceptable tensile stresses in the geomembrane due to these settlements it must be ensured that friction between the geomembrane and the supporting and protective layers is greater towards the subgrade than towards the surface.

The shearing stress is calculated, assuming the same friction towards subgrade and surface, according to Figure 4, on

$$\tau = a \cdot \cos^2\alpha \cdot \mu$$



Shear stress in the plastic liner under surface loading at the slope

FIGURE 4

This shows that shearing stress decreases with an increasing slope angle.

Proceeding from the assumption mentioned above, that deformation behavior is equal to stress-strain behavior, it follows that at a slope angle of 45°, a coefficient of friction of 0.35 and a maximum compressive stress of 1 N/mm^2, that:

$$\tau = 0.18 \ N/mm^2$$

The admissible shearing stresses of plastics are between 0.5 and 0.75 of the acceptable tensile stress. For HDPE, shearing stresses of 0.18 N/mm^2 are - even in the most disadvantageous cases - almost two magnitudes below the short-term admissible tensile stress.

Failure of the material from unacceptably high shearing stresses can hence be excluded if the system has been designed appropriately.

TENSILE STRESSES

Tensile stress can be uniaxial or biaxial and, in either situation, is a critical loading condition.

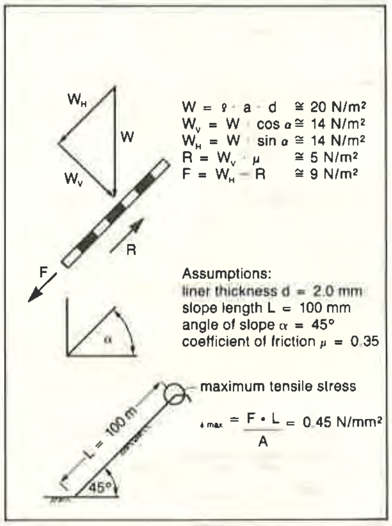
Two loading conditions are here explained:

- dead weight on embankments caused by settled loadings, and
- deformations forced upon the membrane by differential settlement of the subgrade.

DEAD WEIGHT ON THE EMBANKMENT

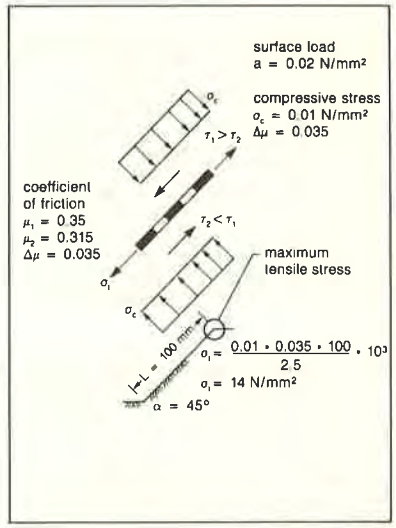
Tensile stresses to the lining membrane, as calculated according to Figure 5, are based on:

$$F = W (\sin\alpha - \cos\alpha \cdot \mu)$$



Tensile stress in the geomembrane at the slope under dead weight

FIGURE 5



Tensile stress in the geomembrane cause by surface loading at the slope

FIGURE 6

$W \cdot \sin\alpha$ increases with rising slope angle, while the frictional force $W \cdot \cos\alpha \cdot \mu$ decreases. Equilibrium is reached under the condition:

$$W \cdot \sin\alpha \quad = \quad W \cos\alpha \cdot \mu$$

$$\text{resp.} \quad \tan\alpha = \mu$$

Under these conditions no tensile loading to the geomembrane caused by dead weight will occur. This example shows a slope angle of 1:2.86.

If a lining membrane made from HDPE has a thickness of 2.0 mm and a density of 1.0 g/cm^3, at a slope angle of 45° and embankment length of 100 m, the tensile stress to the liner at the upper edge of the embankment results in:

$$\sigma_z \cong 0.45 \text{ N/mm}^2$$

This stress is decidedly below the maximum admissible stress of HDPE, even at temperatures as high as 80°C, as could occur on exposed embankments. With bituminous lining membranes, on the other hand, with their high dependence on temperature and low level of strength, such stresses may lead to unacceptable deformations.

TENSILE STRESSES CAUSED BY LOADINGS

If tensile forces are caused by low friction between liner and subgrade, however, and if loadings are passed into the lining membrane, unacceptable limits will soon be reached.

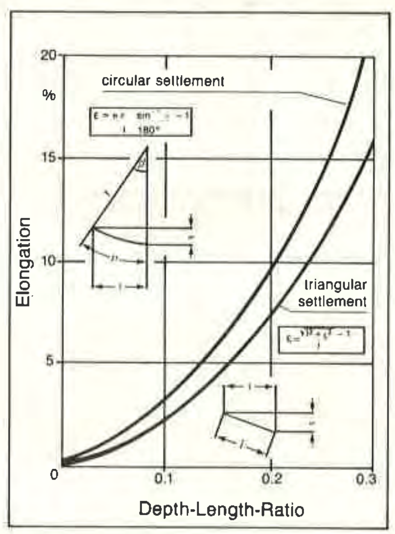
At a medium loading of 2 Mp/m^2 and a reduction in friction of 10% ($\Delta\mu$ = 0.035) tensile stresses - as shown in the example in Figure 6 - would occur before dead weight loadings and thus:

$$\sigma_z = 14 \text{ N/mm}^2$$

This degree of tensile stress approaches the range of the short-term yield stress for HDPE and must therefore be considered unacceptable.

DIFFERENTIAL SETTLEMENT

Deformation is forced upon a lining membrane when settlement is differential. Assuming that these deformations are distributed evenly to the area of settlement, certain elongations result from a particular mode of settlement, because of their geometric depth-to-length relationship, as is shown in Figure 7.



Elongation in the area of settlement as a function of the depth-length ratio

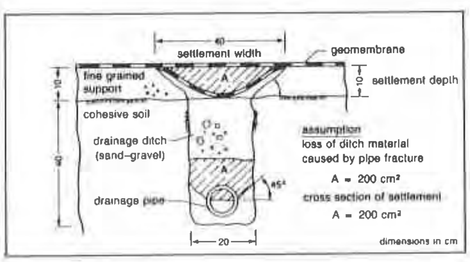
FIGURE 7

If the admissible deformation of a geomembrane made from HDPE is 5%, settlement ratios of 0.15 will follow; thus, a settlement of 15 cm can be absorbed at a length of 1m without stressing the material unacceptably.

It must be guaranteed, however, that the geomembrane does deform evenly across the area of settlement. This requirement is met if lining membranes with an adequately high modulus of elasticity and an appropriate minimum thickness are used. Earlier it was shown that geomembranes of HDPE fulfill this qualification at a minimum thickness of 2.0mm. Further it can be shown that these geomembranes are able to deform beyond the actual area of settlement, thus reducing deformation in this specific area.

Such deformation-related dimensioning is accepted for supporting plastic structures in other fields of applications (5). Dimensioning is thus simplified, because the relatively complicated relations between time, temperature and other influences do not have to be taken into consideration when it is ensured that material deformation values will not be exceeded. This simple method is easily applied to geomembranes under tensile loading where the deformation forced upon the membrane results from design loads.

The example illustrated in Figure 8 shows that by changing the thickness of the lining membrane and by constructive change of the lining system, it is guaranteed that under differential settlement the deformations are kept within given limits. The example shows the loading that could occur on a lining system if a drainage pipe below the liner were to fracture.

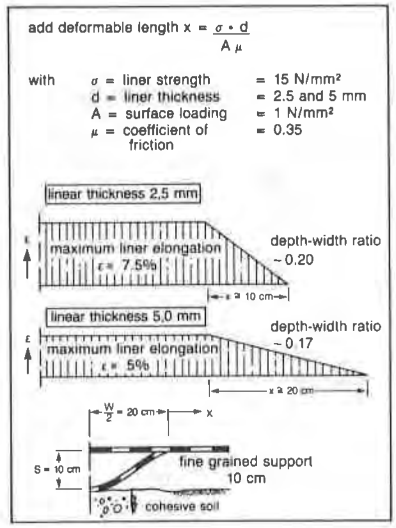


Deformation of the geomembrane under differential settlement (Example for calculation)

FIGURE 8

Figure 8 shows a cross-section of the lining system. Assuming that the fine sandy particles will be scoured by the fracture of the pipe, creating a triangular trough with a depth of 10cm, it follows from Figure 7 that for a settlement ratio of 0.25, an elongation of more than 10% will take place, without taking the additionally deformable length into account. Such elongation would not be acceptable.

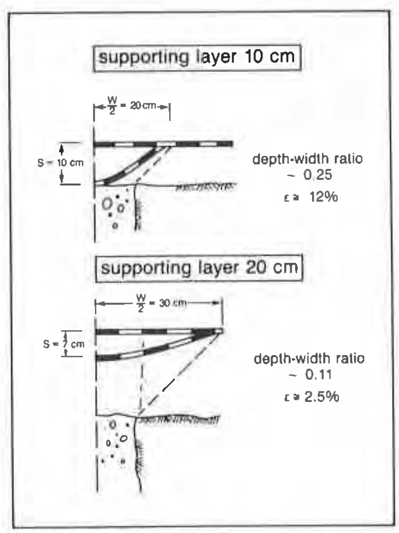
When this additionally deformable length is considered, the length of the area involved in the deformation is $\ell_2 \geqq 50$ cm, according to Figure 9. Thus the elongation decreases to approximately 7.5%, as shown in Figure 7, yet it still remains unacceptably high. In doubling the liner thickness to 5.0 mm, the additional deformable length increases to a total of 20 cm, with a ratio of settlement of 0.15 and an elongation of less than 5%.



Influence of the liner thickness on the additionally deformable length (Example for calculation)

FIGURE 9

If the height of the supporting layer is doubled, the width of settlement increases to 60 cm, as shown in Figure 10. From Figure 7 one can calculate an elongation of less than 5%, taking into account the additionally deformable length for the 2.5mm HDPE geomembrane.



Influence of the supporting layer thickness on the geomembrane deformation (Example for calculation)

FIGURE 10

These examples show that, by changing the liner thickness as well as by making constructive changes to the lining system, it is possible to dimension geomembranes in such a way that they will not be subjected to unacceptable loadings. Other loading cases can similarly be calculated.

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Performance of an Electrical Resistivity Technique for Detecting and Locating Geomembrane Failures

An electrical resistivity survey technique has been developed and tested for assessing the integrity of geomembrane liner systems installed in fluid impoundments. Development of the technique has included two-dimensional computer modeling and three-dimensional physical model testing. A one-acre geomembrane lined surface impoundment was used for full-scale testing of the technique. Tests were conducted to detect and locate single and multiple leaks of different sizes. Results indicate the technique can be used to detect and locate single and multiple leaks as small as one inch in diameter with an accuracy of five feet or less.

INTRODUCTION

The detection and location of leaks in geomembrane liner systems at hazardous waste disposal/storage facilities is necessary in the assessment of the performance of the liner system. Southwest Research Institute has developed an electrical resistivity technique to detect and locate leaks in these liner systems. The technique takes advantage of the high electrical insulating properties of a liner in comparison with the fluid contained above the liner and the soil under the liner. If the liner is physically punctured or separated, the electrical conductivity of the fluid and underlying soil form a detectable current path by which the leak may be detected and located. The technique can be applied on the surface of the fluid above the liner. A summary of the project activities and results is presented in the following section.

PROJECT SUMMARY

Geomembrane liners made from impervious plastics and rubbers exhibit high electrical resistance. When a liner is installed in a landfill or surface impoundment, it effectively acts as an electrical insulator between the materials contained in the facility and the surrounding earth. If the liner is physically punctured or separated, conductive fluid flow through the leak establishes an electrical shunt through the liner. This low resistance shunt forms an electrically detectable region which is the basis by which a leak may be detected and located.

Figure 1 shows the basic surface electrical resistivity testing technique for detecting and locating a leak in a geomembrane liner. A current source is used to inject current across the boundary of the liner. The liner has an electrical leak path, as shown. When a voltage is applied between the source and remote current return electrodes, current flows through the leak as shown in the figure. If a soil cover were present over the edges of the liner, current would also flow through this soil cover to the remote current return electrode, by-passing the liner. Potentials measured on the surface are affected by the current distributions near the leak and can be used to locate the leak.

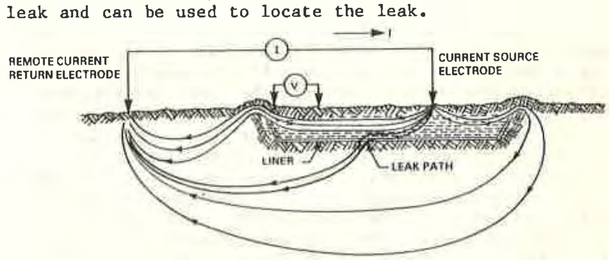

Figure 1. Conceptual electrical resistivity testing technique applied to detect and locate leaks in a geomembrane liner system.

Model studies were performed to validate the electrical resistivity liner survey technique. These studies provided an opportunity to experimentally analyze the current distribution and surface potentials resulting from various leak configurations. This information was needed to develop field survey equipment. The modeling studies and results are presented in the next section.

Computer Modeling Studies

Two-dimensional resistive network computer modeling studies were performed to predict the influence of an electrical current penetration through a geomembrane liner and associated surface potential voltages inside a landfill or fluid impoundment. The influences sought were the magnitude of the surface potentials at given locations for cases where fluid leaks or electrical current paths existed through the liner. The two-dimensional resistor network designed to simulate a liner was modeled using a general purpose circuit simulation computer program called SPICE. This software allows simulation of circuits containing resistors, capacitors, inductors, and voltage and current sources. The resistivity of the fill inside the liner (either soil or fluid) and the earth surrounding the liner was modeled using normalized resistance values of one ohm. The liner was characterized in the model by using parallel 1000-ohm resistors along the path of the liner, depicted by the solid line marked "liner" in Figure 2. To simulate a leak or conductive path through the liner, one of

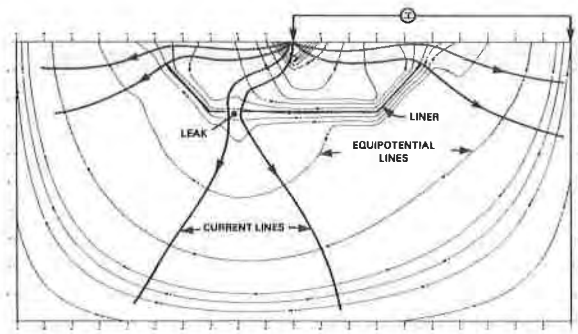

Figure 2. Two-dimensional computer model of a liner with a leak. Equipotential lines at the top are nonsymmetrical, indicating the presence of a leak.

the 1000-ohm resistors representing the liner was replaced by a 1-ohm resistor. As a result, the leak path has the same resistance as the earth. In effect, the liner is thus removed at that location.

Computer Results

Figure 2 illustrates the results of one of the computer generated two-dimensional analyses of a liner having a leak in the bottom. This figure represents a two-dimension cross-section through the liner and surrounding earth. The outline of the liner has been sketched in the figure and is represented by the three-sided trapezoidal figure in the center of the plots. The equipotential lines which appear in the figure are generated and plotted automatically by the computer program described earlier. The equipotential lines showing the voltage distribution patterns were computer generated. The current flow paths are at right angles to the equipotential lines and were sketched in by hand.

This figure shows that, as predicted, when a leak penetration is present in the liner, current flow between electrodes located inside and outside the liner will follow two paths, namely, through the leak and over the buried edges of the liner. Note the non-symmetry of the equipotential lines terminating on the surface of the facility, above the liner. The voltage gradient is clearly steeper along the surface on the left side of the current injection electrode above the leak. This voltage gradient is due to more current flow in the direction of the leak.

Based upon the results of the computer analyses, a three-dimensional physical scale model was designed and constructed. The following section describes the model and presents the experimental results.

Physical Scale Modeling Studies

After completion of the computer modeling studies, a three-dimensional physical scale model was designed and constructed. The objectives of using this model were to: (1) observe the surface voltage distribution patterns created by a fixed liner geometry, injection current location, fill material inside the liner, and leak configuration; (2) test the system instrumentation and assess its suitability for full scale testing; and (3) determine the accuracy of the technique in locating a leak.

Model Design

The model was designed to allow studies of the resolution capabilities of the technique, i.e., how accurately it could locate a leak in the liner. Outside dimensions were established at 3.05 m (10 ft) on each side, for a total lined floor area of 9.3 m^2 (100 ft^2) Maximum depth was established at one foot, allowing variation of water depth during the experiments. A black polyethylene sheet 6 mils thick was selected as the geomembrane. A sand bed was placed below the liner. The moisture level of the sand was typical of the surrounding soil in the area.

Results

Surface potential measurements were made with 0.1 m (0.33 ft) of water in the model. Figure 3 presents a polar coordinate equipotential plot of the measured data

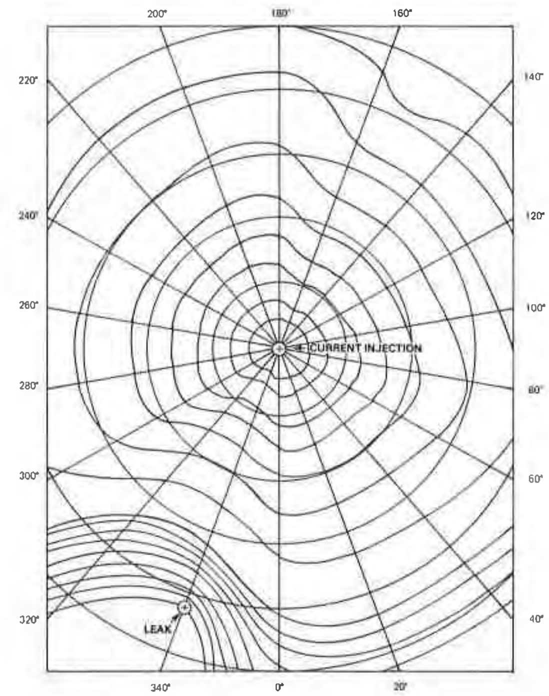

Figure 3. Equipotential plot of voltages on the surface of the water with a single leak.

from one experimental configuration. The current injection electrode is shown in the center of the plot. A leak is shown on the 340° radial. The distortion of the equipotential lines indicates increased flow of current around the leak.

Testing on the water-filled model (simulating a fluid impoundment) confirmed that leaks can be detected and located with the technique. To determine if the system would achieve the same performance on a simulated landfill, the water in the model was replaced with soil. Figure 4 presents the results of a test with 0.06 m (0.22 ft) of soil in the model and one leak. The current injection electrode is shown in the center of the plot. The equipotential contours are distorted in the area of the leak as with the water-filled condition.

These studies further demonstrated the fundamental concepts of this approach which were first defined by the computer modeling efforts. Based upon these results, the measurement equipment, data processing software, and electrodes were specified for full-scale testing at a one-acre lined facility located on Institute grounds. Testing at this scale was performed to measure

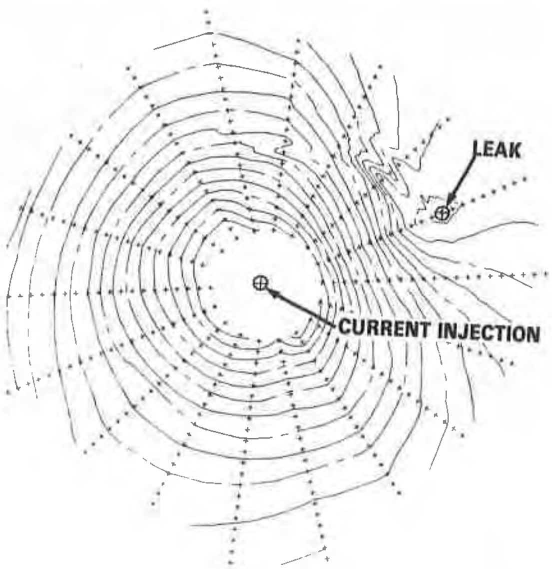

Figure 4. Equipotential plot of voltages on the surface of the soil with a single leak.

the performance of the technique at full-scale conditions which more accurately simulate actual field conditions. The design of the one-acre test facility and test results are presented in the following section.

Full-Scale Studies

The electrical resistivity leak detection technique was tested at a one-acre lined water-filled impoundment. The overall goal of the testing program was to determine how well the technique worked in a field environment under full-scale conditions. To accomplish this goal, the instrumentation required to apply the electrical resistivity technique was assembled. A 100-mil geomembrane liner made of high-density polyethylene was installed in a one-acre impoundment. Controllable leaks were installed in the liner for detection and location studies. Experiments were then performed to evaluate the technique and the instrumentation.

Impoundment Design

A one-acre impoundment was utilized to test the liner assessment technique. The facility was designed to accommodate up to 6.5 feet of water. Overall dimensions are approximately 65.8 m (216 ft) x 65.8 m (216 ft) from the top of the berms. Side slopes are approximately three to one. An access road exists around the facility to allow for vehicular traffic during testing. A 100-mil high density polyethylene liner was installed in the facility to serve as the test liner. The liner was anchored at the top of the berm in a 0.6 m (2 ft) deep trench. The trench was backfilled with soil.

It was necessary to construct leaks in the liner to facilitate testing of the leak detection technique. Five pipes, each 0.3 m (1 ft) in diameter and 0.1 m (4 in.) long, were installed through the floor of the liner. The pipes were installed with 0.05 m (2 in.) above the liner and 0.05 m (2 in.) below the liner. Each pipe was made of HDPE to allow the liner to be welded to the pipe, creating a water-tight seal. A cap was constructed to fit over this pipe. The cap was made from two PVC rings which were bolted together. To create various leak sizes, disc-shaped HDPE inserts having various sized holes were placed between the rings. The rings were bolted together and placed over the top of the pipe. These leak points allowed control of leak location during the performance tests.

Full-Scale Measurements

Figure 5 shows a conceptual drawing of the full-scale test impoundment and operation of the equipment. All measurements were taken along radial lines 5° or 10° apart, beginning at the current source electrode out to markings on the berm. The logging cable was positioned on these marks during data acquisition runs. The location of any anomaly, such as a leak, could be calculated from the radial and the odometer data. The odometer, located on the electric winch, established the exact location of the measurement electrode with respect to the current electrode. To establish a baseline condition, the entire surface of the pond was surveyed with all leaks closed.

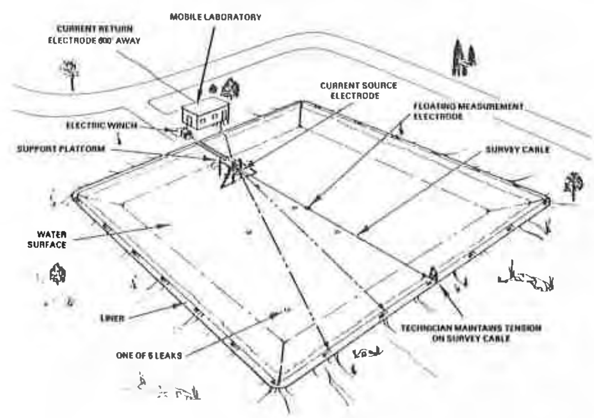

Figure 5. Conceptual drawing of the test impoundment and operation of the test equipment.

Then, various leaks were opened by removing the caps previously described. Potential measurments were then taken over the surface of the water. Soil moisture conditions below the liner were not measured. A contour plotting computer program was used to produce equipotential contour plots of the no-leak and leak data. The results of the testing are presented in the following sections.

No-Leak and Single Leak Results

A contour plotting computer program was used to produce an equipotential contour plot of the voltages measured over the surface of the water in the no-leak case. This plot is shown in Figure 6. The X and Y axes define the approximate north and east water level boundaries of the impoundment. The angles of the radials are indicated. Voltage measurements were taken at 0.3 m (1 ft.) increments along each radial. The current source electrode is identified by a dot above the X axis at the 28-m (92-ft) point. The water was approximately 1.5 m (5 ft.) in depth.

The equipotential contour lines in Figure 6 show insignificant distortions of the surface potentials across the water surface. These results are similar to the no-leak contour plots of data taken with the physical scale model. The contour lines tend to be concentric semicircles close to the current injection point. Moving away from this region, the contour lines begin to straighten out, due to the effect of small amounts of current flow across the liner. Part of this current

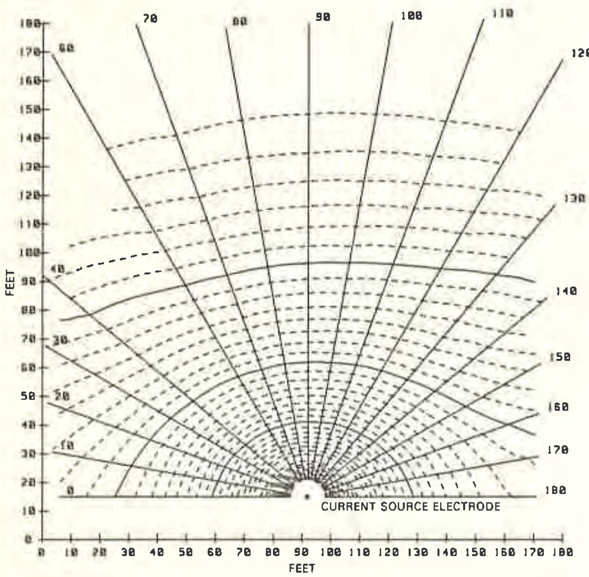

Figure 6. An equipotential contour plot of no-leak measurements.

flow is due to the capacitive effects of the liner. Overall, this plot indicates no sudden, unexpected changes or perturbations in the current flow on the surface of the water.

To determine if the survey technique could detect and locate a single leak, a 0.3 m (1 ft) diameter leak was opened. The surface potentials were measured and the results plotted as before. An examination of the equipotential contour plot in Figure 7 shows that the contour lines close around the location of the leak. This indicates that the flow of current converges in the area of the leak, shown by the "bull's-eye" pattern centered on the leak. The bull's-eye potential pattern located on the surface of the water serves to graphically reveal the location of the leak. The plots of the data for each radial along with the equipotential contour plots serve as excellent analytical tools to detect and locate a leak.

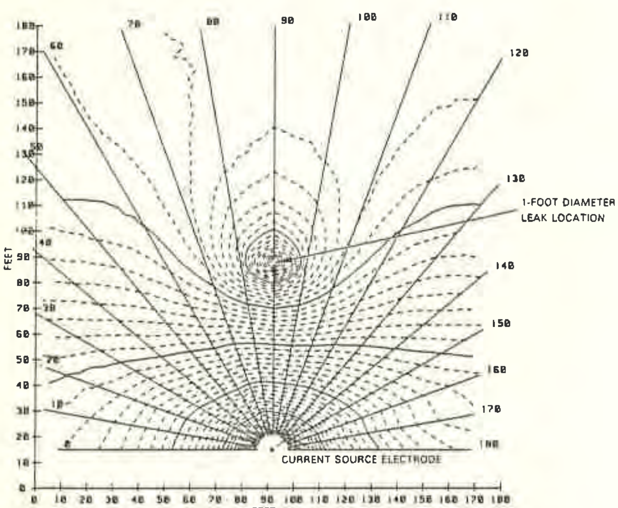

Figure 7. An equipotential contour plot showing the distortions from a 1-foot diameter leak on the 90° radial.

To determine if the technique could detect and locate a smaller leak size, a 0.02 m (1 in.) diameter leak was installed on the 90° radial 22 m (72.3 ft) from the current electrode. Surface potential measurements were taken at 0.3 m (1 ft) increments along the 60° through 120° radials. The equipotential contour plot of these data shown in Figure 8 indicates a significant distortion on the 90° radial in the proximity of the true leak location. The results of this test indicate that the electrical resistivity technique can be used to detect and locate a 0.02 m (1 in.) diameter leak in approximately 3716 m^2 (40,000 ft^2) of liner surface area to an accuracy of one foot. This accuracy is obtainable when the contour plots are used together with the raw data (not shown). The contour line shifts along the 110° radial in Figure 8 do not indicate an anomaly, since the shift is consistently the same magnitude.

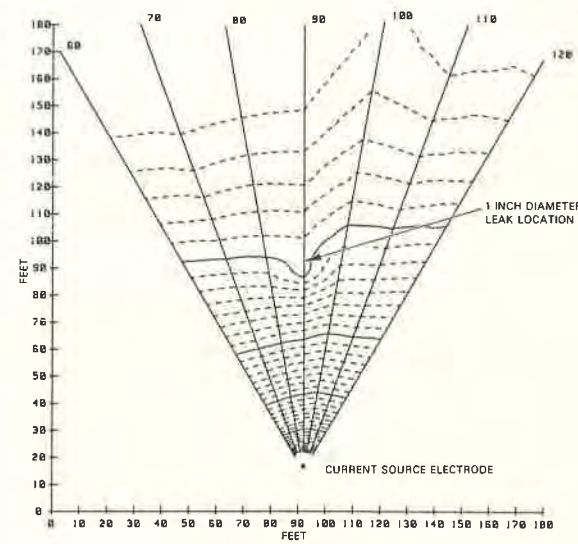

Figure 8. An equipotential contour plot showing the distortions from a 1-inch diameter leak.

Multiple Leak Results

Multiple leak configuration experiments were performed to determine the leak signatures and sensitivity of the electrical resistivity technique. An experiment was performed in which three one-foot diameter leaks were detected and located. These leaks were established at locations 01, 04, and 05. Surface potential measurements were at one-foot increments along the 70° through 125° radial lines. Voltage peaks occur directly over each of the three leaks. The equipotential contour plot shown in Figure 9 clearly shows the presence of each leak.

CONCLUSIONS

Full-scale tests have demonstrated the ability of the electrical survey technique to detect and locate single and multiple leaks in geomembrane liner systems installed in water-filled impoundments. For all tests, the technique detected the presence of a leak. The accuracy of leak location ranged from within 0.3 m

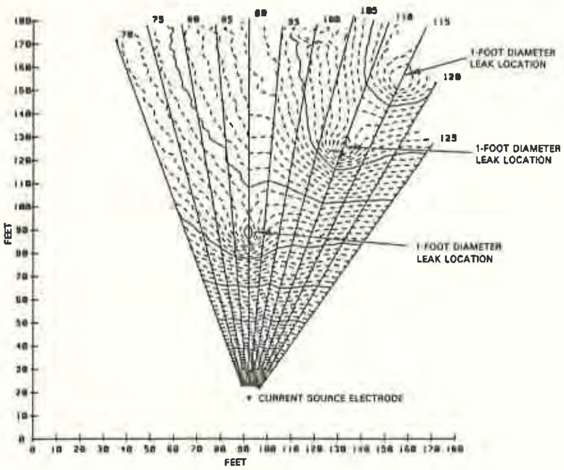

Figure 9. An equipotential contour plot showing the distortions from three 1-foot diameter leaks on different radials.

(1 ft) to 1.5 m (5 ft), depending on the location of the leak with respect to the current source electrode and the total number of leaks present. Although full-scale tests of the technique with soil fill over the liner were not conducted as part of this project, the results of model scale tests strongly indicate that the technique can be used to survey geomembranes installed at landfills.

The approach to finding leaks when their location is unknown would be the same as the approach used at the SwRI test impoundment. Modification to the survey approach, such as moving the position of electrodes, can easily be accomplished if necessary. This may be necessary if, for example, the leaks were not along a radial traverse.

The water depth and leak size and shape will likely influence leak signatures. Although the exact relationships are not known at this time, increasing depth appears to reduce the magnitude of the signature. This will become a limiting factor only if the signature strength falls below the background noise of the instrumentation. Background noise will be a site specific characteristic and therefore cannot be predicted.

ACKNOWLEDGEMENTS

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Evaluation and/or Monitoring Instrumentation System for Ponds, Using Liner Material as Indicator

An instrumentation concept is presented that is useful for evaluating synthetic pond lining material for resistance to contamination from organics. The method can also be used for screening for harmful effects from inorganic compounds, but the current data base for inorganics is not large (50 compounds) as compared to the organic compound data base (1500 compounds). The same techjnology that is used in the evaluation procedure, by using the same elastomer to build a detector as is used in the pond, can be used as a real time indicator to monitor the impounded material for conditions which would be harmful to the liner. By using selected, highly reactive elastomers for the detectors the containment site can be monitored for a broad range of organic leakage or contamination in free liquid, aqueous solution or vapor state. Supporting data are given and supporting references listed.

BACKGROUND

Anyone who has been involved for very long with the containment of industrial wastes or byproducts in lined ponds or pits has their own inventory of horror stories to tell. It is a rare individual in such responsible charge of facilities that has not lost some sleep considering new "what if" situations. While wastes of all classification have been known to be involved in misadventure in the past, liquid organics, or contamination from them (including substituted organics), seem to have been responsible for more than their fair share of problems. For this reason, it is the synthetic lining materials, polymers - etc., that will be of most concern to this paper rather than natural materials such as compacted earth, modified earth or clay blankets. This is not to say that the natural materials do not have misapplication or poor workmanship problems, just that synthetics seem to be involved in more problems with organics than do non-synthetics. There are exceptions to this generalization, however, as when natural materials are used material in so-called solid waste landfills. While the main emphasis of this paper will be the problems of organics with synthetic liners, secondary consideration is given to inorganic solutions with synthetic liners. Still further, tertiary treatment is given to applicable monitoring situations with liners of natural materials.

A key cause of leakage or failure of synthetic liners has been unintended contamination of the impounded material with a substance that the liner was never designed to survive. This happens because when the liner is specified it is economically impractical to require a liner is that is indestructible in all circumstances. What happens for then that an entirely reasonable decision is made to design the expected case of normal operations, with perhaps some consideration given to abnormal process excursions or possible gross contaminants. It is usually assumed tha minor contaminants will be dispersed or diluted to the point of no concern.

A few of the many possible scenarios that could frustrate the above design are as follows: 1) a trace contaminant that does not mix is introduced to the pond - eventually causing localized damage where it concentrates - at the surface line or on the bottom; 2) a trace organic that is accumulatively absorbed by the liner is introduced to the pond over a long period of time thereby causing slow swelling and weakening of the whole liner; 3) an unintended contaminant in conjunction with a known contaminant causes synergistic deleterious effects; 4) a major process upset or a natural disaster causes a large inflow of a harmful substance to the pond.

Other failures have been attributed to a changing composition of the impounded material. This could be increasing concentration, sludge disposition, or changing from an oxidizing to a reducing condition (or vice versa) on the bottom of a pond in the case of a liquid. In the case of a supposedly dry solid waste land fill, it could be the result of the formation of liquid or gaseous degradation products that were harmful to the liner.

All of the preceding scenarios addressed either something getting into the impoundment that shouldn't have or an unexpected change taking place in the material that was imponded. There is a third class of situation that could cause impoundment leakage or failure, however, that of external events such as earthquake, flooding, undercutting of or removal of base material by ground-water.

Leakage or failure of an impoundment lining system which was caused by either of the first two modes could be predicted and even circumvented if suitable monitoring instrumentation were in place which could monitor changing conditions in the influent (in the case of liquid) and/or various susceptible locations within the impoundment. Instrumentation with such placement would do nothing, however, to forewarn of impending leakage due to external events (natural disasters). It is possible though, to install instrumentation (similar to that used in the first two modes) external to the impoundment which would, if the initial leakage rate increased slowly enough, forewarn of a large scale failure. Such instrumentation could also monitor for regulation compliance.

INSTRUMENTATION CONCEPT.

Considering the recommendation for instrumental detectors in the influent and susceptible areas of the impoundment, many engineers would agree to the desirability of such action, but would get bogged down trying to decide which parameters were the important ones to monitor (pH conductivity, specific ion, redox potential, total organic carbon, specific organics, etc.).

The instrumentation concept that is being here proposed as a solution to the above problems would also eliminate the need to make decisions as to what parameters to monitor. This is accomplished by using a small coupon of the lining material in use as a detector/indicator. This coupon does not have to be from the actual liner in question - just be of the same formulation, i.e., duplicate the polymer, elastomer, thermo-plastic or other synthetic used in the liner, in a coupon of suitable shape and thickness.

The principal of this concept is that whatever happens to effect the liner will also effect the coupon in the same way, at the same time. The result of a deleterious effect will be either a positive or negative induced stress in the material. All one has to do is have a means of detecting and measuring this stress.

The case of using an elastomeric lining material is the easiest to describe. In this case, anything that is sorbed into the elastomer (organics are the most common) will cause it to swell in proportion to the amount sorbed causing a positive stress (for the purpose of this discussion, absorption and adsorption are considered to be interchangable and are referred to as sorption). Anything that is leached out or removed from the elastomer will cause it to contract in proportion to the amount leached causing a negative stress. Although these stresses are admittedly small in some cases, there are several ways in which they can be detected and measured. Several years of research have pointed out that, from the standpoint of level of desired signal and the harsh environment such a device must operate in, strain measurement using strain gages offers many advantages with few disadvantages.

A typical configuration for an elastomeric detector/indicator would consist of a rectangular slab of elastomer 15·30 mm, from 1.5 to 2.0 mm thick, with one side bonded to a flexible, but impermeable backing material of the same rectangular size. The backing material can be of any suitable material, stainless steel and reinforced plastic laminates having been used to date. A typical backing thickness would be anywhere between 0.1 and 0.4 mm. Before bonding the elastomer to the backing, a foil type strain gage is bonded to one side or the other of the backing. If the detector is to be used in a conductive medium, the gage mounted on the inside - the side that will be covered by the elastomer. A non-conductive medium application would allow the gage to be mounted on the outside (gaseous applications). The electrical leads from the strain gage have to be insulated and anchored according to the intended service. Very satisfactory connections and insulation have been effected by a variety of means.

In service, one face and the edges of the elastomer are free to react with the local environment while the underside of the elastomer is bonded to the impermeable backing. Thus, any stress induced in the elastomer, positive or negative, is translated to a bending moment in the backing material. This bending moment is transferred to the strain gage as either a compressive or tensile strain depending on which side of the backing the gage is mounted on and the sign of the original stress.

The detector size described above is the size we use in our laboratory for screening and evaluation work. Field detectors have been fabricated which are only 20% as large.

While the preceding detector fabrication technique is applicable for both cured and uncured elastomers, thin, sheet, non-elastic polymers present a different case.

With these materials, the best that can be done is to cover a mounted slab of one of the more reactive elastomers in such a way that any chemical damage to the liner material will cause an increase in permeation through the damaged liner (on the detector) which will be indicated by increasing stress in the underlying reactive elastomer. Such a scheme would not allow the detection of positive or negative stress in the actual liner - it would indicate only that some damage was occuring.

As an alternative to the above, the detector/indicators can be fabricated using an elastomer or group of elastomers known to be reactive to the same classes of contaminants that would be most harmful to the actual liner. These, then, would be installed in a manner similar to the case where the detectors were fabricated from elastomeric material duplicating the liner composition.

For the case of monitoring for compliance, as in down-gradient from a contaminant of any type, similar type detectors can be used - with the exception that the elastomer used should be one with broad spectrum reactivity, i.e., it should react to both aromatic and paraffinic organics. It is possible to build a single detector that would be responsive to almost any organic, without concern as to whether it was stratified at the bottom of the ground-water column, dissolved within the water, floating on top of the water or in a vapor state above the water. Such gages can be made to be very senesitive, also, with detection limits for most organics at a fraction of a part per million (ppm). This high sensitivity is achieved by the proper selection of materials (elastomer and backing) and the thickness of same along with the use of solid state strain gages having high gage factors. Gage factor, while determined from the relationship of many variables is essentially a measure of the sensitivity or output of a gage for a given set of conditions. The gage factors of foil type strain gages are typically 2 to 3 while those obtainable with solid state (transistor) type gages range from 50 to 300. Such high sensitivities, however, do carry an increased risk of overloading the gage.

Ways of minimizing the overload risk to the strain gage, and other features of the detector/indicators are the subject of patents pending and will be reported at a later date.

SUPPORTING TECHNOLOGY

Much of the technology used in the current indicators was the subject of a previous research project which was reported in reference (1) where it was shown that the aromatic reactivity of a hydrocarbon composition could be determined by exposing an elastomer of known composition to a liquid hydrocarbon and then measuring the resulting weight change or volume change of the elastomer after a specified time period. This technique was the basis of a 2 hour, manual assay procedure to determine the Benzene Aromatic Equivalent (BAE) of any hydrocarbon mixture of known or unknown composition. The method related the action of a given hydrocarbon on a particular elastomer to an equivalent volume percent of benzene in a non-reacting solvent that would cause an identical action on the same elastomer. The resulting BAE value of a mixture of unknown composition is predictive of the chemical and physical properties and the biological activity of that mixture.

It is interesting to note that the original equation for describing the BAE calibration curve, repeated here for your information:

$$Y = X^{a+B\ln x} \qquad \text{(eq. 1)}$$

Wherein:
Y = fractional volume percent BAE
X = fractional percent of a corrected and normalized weight gain of an elastomer test coupon
A = coefficient
B = coefficient
ln = natural logarithm
Note: full derivation of the equation is given in ref. (1)

which was based on probably 25 data points, had a coefficient of correlation, r^2, of about 0.97. As of now, using the manual assay method, we have determined the BAE values of over 1500 known chemicals, assayed an additional 2500 unknown mixtures, while accumulating an additional 1100 curve calibration points and the r^2 is still 0.97 (all data points included) for the same equation.

It is the above mentioned 1500 known chemical BAE values that is of pertinent value to this paper. A summary of the data and its meaning will be given in the next section.

The technology represented in reference (1) was useful in developing the detector/indicator instrumentation which is the subject of this paper. However, the BAE concept had limited usefulness as an industrial detector because it used elastomers that were only reactive to aromatic molecules. The aliphatic hydrocarbons had little or no effect on the elastomers used. This lead to further investigation of a large variety of cured elastomers, the results of which were first reported in reference (2). This reference reports, among much other data, tabulated response data concerning exposure of 31 separate sheet elastomers to 1 aromatic hydrocarbon, 1 aliphatic hydrocarbon, 2 mixtures of the aromatic and aliphatic hydrocarbon, 1 primary alcohol, 1 secondary alcohol and water. This data plus the previously unpublished BAE values of 1500 known chemicals mentioned above form the basis of interpreting the results of short term exposure tests to yield specific information on the suitability of a particular lining material for a given service. The same data and test results are useful in determining which elastomer(s) to use as detector/indicator active elements for a particular situation.

Within the group of 31 sheet elastomers tested were found several that responded to aromatics in preference to aliphatics, a couple that responded to aliphatics in preference to aromatics, several that responded to both equally (almost) and one that had almost no response to anything except the secondary alcohol. Most other test elastomers had very low to low response to either of the alcohols. Response to water varied from 0 to 1.43 weight percent absorption in two hours.

It was during the development of the data in ref. (2) that we also noted the greatly enhanced response of several elastomers to many halogenated hydrocarbons, i.e., two hour weight gains of up to 495 percent (in the case of chloroform with cured SBR) versus 120 percent for benzene.

Synergistic effects were noted for several combinations of liquids and many dissolved solid hydrocarbons where the BAE of the combination was greater than the BAE of either component. Methyl and ethyl alcohols and acetone are involved in a wide variety of synergistic combinations. A few of the dissolved solid hydrocarbon cases that were noted include: 3, 4 - Dichloroanaline with a calculated BAE of 127 when dissolved in ethanol and 241 when in benzene; 2 - Aminopyridine with a BAE of 36 when dissolved in water and 217 when in benzene; Phenol with a BAE value of 599 when dissolved in water and 338 when dissolved in benzene; HYdrocinnamic acid with a BAE of 611 when in water and a -3 when dissolved in benzene; and 3- chloropropionic acid with a BAE of 95 in water and 207 in benzene. These represent just a few of the ones we found and we were not really looking for any. I am sure that there are many more that have not been identified yet. The consequence of the existance of these effects is design uncertainty. How do you design to accomodate such effects if you do not know beforehand what combinations or ratios produce them?

BAE DATA AND ITS USE.

All that will be presented here is a summary of the data on 1500 chemicals. This is because the current tabular data on BAE value and desorption numbers of these chemicals occupy 75, 21.6 · 27.9 cm (8.5 x 11 in) sheets of paper. The complete set of data will be published in the future if we can find an appropriate forum.

All of the following BAE data was generatd using two hour exposures of an elastomer designated as JH-21, an acrylonitrile elastomeric polymer. The test coupons were approximately 15 · 30 mm with thickness ranging from 1.65 to 1.91 mm. Coupon weight was a nominal 1g. The tests were performed and correction factors were applied as per reference (1). The elastomer used was more reactive with aromatics than aliphatics with gross two hour weight gains being 118% for benzene and 0.08% for iso-octane. The response to ethanol under like conditions was 1.22%, methanol 2.13% and water 0.154%. Benzene and iso-octane were used for standards in the tests and were normalized to 100 and 0 respectively in the BAE calculations.

Tabulated below are the number of tested chemicals in a class, the name of the class, the range of BAE values for the class (obvious exceptions excluded), and a letter designation to indicate the ease with which desorption is accomplished: A = fully desorbed in 2 hours; B, fully desorbed in 4 hrs.; C, fully desorbed in 24 hrs.; D, half desorbed in 24 hrs.; E, 25% desorbed in 24 hrs.; F, desorbed not at all in 24 hrs.

Table 1. BAE and desorption by chemical class.

No. Tested and Class Name	BAE Range	Desorption Class
3 Acetals	33 - 711	C
27 Acids, Monobasic	5 - 47	A - D
9 Acids, Monobasic, Amino	-60 to 38	A
8 Acids, Monobasic, Halogen	55 - 174	C - D
4 Acids, Monobasic, Hydroxy	3 - 8	D - F
8 Acids, Monobasic, Other substituted	7 - 170	C - F
12 Acids, Polybasic	4 - 36	B - C
3 Acids, Polybasic, Amino	7 - 63	A - D
4 Acids, Polybasic, Hydroxy	-7 to 152	B - F

2	Acids, Polybasic, Other substituted	10 - 30	C
12	Acids Aromatic	-12 to 284	B - E
4	Acids, Aromatic, Amino	8 - 140	A - D
5	Acids, Aromatic, Halogen	0 - 111	A - D
7	Acids, Aromatic, Hydroxy	6 - 166	C - D
13	Acids, Aromatic, Other substituted	0 - 258	B - D
9	Acid Anhydrides	13 - 75	C - D
2	Acid Anhydrides, Aromatic	255 - 322	C
5	Acid Halides	23 - 108	E - F
34	Alcohols, Monohydric	2 - 33	B - D
10	Alcohols, Monohydric, Ether	5 - 29	C - E
4	Alcohols, Monohydric, Halogen	60 - 81	C - F
3	Alcohols, Monohydric, Other substituted	23 - 50	C - D
19	Alcohols, Polyhydric	0 - 4	B - E
3	Alcohols, POlyhydric, Ether	0 - 1.5	A - B
8	Alcohols, Aromatic	39 - 55	D - F
5	Alcohols, Aromatic, Other substituted	36 - 76	D - F
10	Aldehydes	7 - 80	D - F
2	Aldehydes, substituted	14 - 88	C - D
2	Aldehyde Derivatives	19 - 21	C - D
5	Aldehydes, Aromatic	60 - 105	D - F
4	Aldehydes, Aromatic, substituted	68 - 114	D - F
10	Amides	16 - 104	C - F
4	Amides, substituted	-1 to 57	A - C
6	Amides, Aromatic	31 - 171	C - D
4	Amides, Aromatic, substituted	15 - 218	C - D
27	Amines, Mono	0 - 131	C - D
17	Amines, Mono, Hydroxy	0 - 30	A - C
7	Amines, Mono, Other substituted	26 - 98	C - D
11	Amines, Poly	5 - 78	C - F
1	Amine, Poly, Hydroxy	3	F
23	Amines, Aromatic	44 - 392	A - F
3	Amines, Aromatic, Halogen	108 - 284	D
2	Amines, Aromatic, Hydroxy	34 - 37	C - F
8	Amines, Aromatic, Nitro, Nitroso	80 - 211	D - E
2	Amines, Aromatic, Other substituted	0 - 82	A - E
4	Amines, Quaternary	-19 to 20	B
4	Azo Compounds	63 - 282	C - D
5	Carbonates, Carbanilides	-38 to 299	C
2	Carbazides	7 - 25	D
6	Carbonates	14 - 358	C - D
2	Carbonates, substituted	103	D
3	Cyanates, Iso, Thio	60 - 139	D - F
36	Esters, Monobasic	7 - 93	B - F
7	Esters, Monobasic, Ether	36 - 70	C - E
8	Esters, Monobasic, Ether, Halogen	70 - 113	C - E
6	Esters, Monobasic, Ether, Hydroxy	1 - 32	C - F
5	Esters, Monobasic Ethers, Other substituted	30 - 1033	C - D
35	Esters, Polybasic	8 - 76	D - F
7	Esters, Polybasic, Ether	15 - 27	D - F
4	Esters, Polybasic, Hydroxy	6 - 17	E - F
1	Ester Polybasic, Other substituted	51	E
31	Esters, Aromatic	11 - 95	D - F
4	Esters, Aromatic, Ether	17 - 20	D - F
6	Esters, Aromatic, Hydroxy	14 - 79	C - F
1	Ester, Aromatic, Other substituted	51	F
3	Esters, Ortho	23 - 90	C - D
17	Ethers	8 - 131	C - E
3	Ethers, Halogen	64 - 128	C - F
20	Ethers, Aromatic	18 - 110	C - F
2	Guanidines	0 - 41	C - D
54	Halogen Compounds	4 - 364	C - F
10	Halogen Compounds, Olefinic	23 - 165	C - D
27	Halogen Compounds, Aromatic	41 - 180	C - E
17	Hetrocyelic Compounds, Nitrogen	30 - 246	C - D
1	Hetrocyelic Compound, Oxygen	135	C
9	Hetrocyelic Compounds, Sulfur	51 - 75	E
6	Hydrazines, Hydrazides	0 - 102	C - D
13	Hydrocarbons, Saturated, Paraffinic	2 - 49	B - C
5	Hydrocarbons, Saturated, Naphthenic	11 - 21	B - C
13	Hydrocarbons, Olefinic	9 - 53	C - D
22	Hydrocarbons, Aromatic, Benzenoid	23 - 100	C - E
10	Hydrocarbons, Aromatic, Polynuclear	62 - 247	C - F
1	Hydrocarbon, Aromatic, Aryl Olefinic	122	C
5	Imides	3 - 171	B - C
17	Ketones, Mono	42 - 114	B - D
4	Ketones, Di	11 -115	C - E
8	Ketones, Aromatic	55 - 174	D - F
4	Lactones, Lactams	2 - 197	C - D
3	Nitrates, Nitrites	50 - 115	C
7	Nitrites	15 - 133	C - F
3	Nitriles, Amino	3 - 70	C - D
3	Nitriles, Ether	4 - 61	C - E
4	Nitriles, Other substituted	5 - 103	C - D
10	Nitro, Nitroso Compounds	64 -113	C - D
4	Nitro, Nitroso Compounds, Halogen	123 - 247	C - D
3	Oximes	27 - 30	C
12	Phenols, Monohydroxy	5 - 23	C - E
5	Phenols, Monohydroxy, Amino	28 - 191	C
2	Phenols, Monohydroxy, Ether	48 - 87	C
7	Phenols, Monohydroxy, Halogen	105 - 405	C - D
5	Phenols, Monohydroxy, Nitro, Nitroso	152 - 272	C - D
11	Phenols, Polyhydroxy	-11 to 398	C
12	Phosphorous Compounds	11 - 78	C - F
3	Phosphorous Compounds, substituted	57 - 82	D - F
4	Quinones	65 - 222	B - C
3	Quinones, substituted	43 - 72	C - D
7	Sugars	0 - 17	A - C
5	Sulfates	0 - 59	A - E
5	Sulfides, Disulfides	60 - 118	C - F
16	Sulfo Compounds, Acids, salts	-91 to 60	A - F

3 Sulfo Compounds, Amides	133 - 198	C - D
3 Sulfo Compounds, Esters	46 - 115	C - F
4 Sulfo Compounds, Halides	72 - 181	C - F
4 Thiols	14 - 133	C - F
9 Ureas, Thioures	-100 to 89	B - C
510 Miscellaneous organic compounds	0 - 218	A - F

In using the above data for practical applications, reference (2) should be consulted. In general, however, when selecting a synthetic liner material to resist trace contamination from any of the above organics: to resist compounds with a low BAE value, neoprene, hypalon, nitriles and similar materials should be selected - butyl, EPDM and silicone rubber should not be used; to resist compounds of high BAE value, only Viton and other highly specialized materials can be used.

If elastomeric liners are contaminated with any of the above organics, those with desorption letter designations A through C will very likely desorb with no lasting ill effects if the contamination is removed promptly. Letter designations D and E imply that some accumulative effect is possible. Letter designation F indicates deffinite accumulative effects.

When selecting elastomers for use in detector/indicator applications the silicones, EPDM, or butyls should be chosen to detect chemical classes with low BAE values or those with a broad range of BAE's. These same materials are also useful for detection of high BAE value compounds, although, if aromatics only are to be detected, neoprene, hypalon, or better still, one of the nitriles should be selectd. When any of the cured elastomers are used in contact with a compound having an "F" desorption designation, detector applications will be one-time use only before requiring replacement. Detectors for most other organics are completely resuable and reversable with certain limitations.

While all of the BAE data presented in this paper were generated using the two hour manual assay, work is well under way to develop automatic instrumental methods of generating the data. In practice, one of the previously mentioned detector/indicators with an inside mounted solid state strain gage is used for a brief exposure. Only the first 60 seconds of the absorption slope is used, giving data that can be correlated with the current BAE data base. The actual corrlation awaits more data points, though.

References.

(1) Chambers, CC; U.S. Patent No. 4 350 496, "Mesuring the Aromatic Reactivity of a Hydrocarbon Composition", (Sept. 21, 1982).

(2) Chambers, CC; International Patent Application Published under the Patent Cooperation Treaty (PCT), "No. W O 82/02 342, "Removal of Hydrocarbon Compositions", (July 22, 1982).

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Applications of Fin Drain Leachate Cut-Offs in the United Kingdom

The disposal of waste by landfill is now well established. However, it is only in recent years that the associated potential environmental hazards have been more fully recognised. The prime hazard is the generation of toxic leachate within the fill that can subsequently escape to pollute local surface and groundwater regimes. In many cases the natural biodegradation of toxic leachate in the ground surrounding the landfill does not afford adequate protection. Thus some form of leachate control is required. Early techniques involved total containment using vertical slurry cut-off walls and horizontal geomembrane cut-offs. However, these techniques can lead to problems of over topping of the slurry walls or possibly more severe damage if the geomembrane beneath the landfill is ruptured. Since water is the main leachate solvent, a more positive approach to leachate control stems from the use of drainage installations which allow separation of clean and polluted water. This can be achieved by using a synthetic fin drain cut-off which is a laminar fin comprising a vertical water conducting core laminated between a filter fabric and a geomembrane. Using this arrangement liquid can enter the fin via the upstream filter fabric but cannot flow across the geomembrane on the downstream side of the core. Since the fin is vertical, or sub-vertical, the intercepted liquid flows down the core under gravitational drainage to be collected in a carrier drain at the base of the fin. The fin drain and fin drain cut-off are described in detail and their application to leachate control is briefly illustrated through three case histories. The first of these relates to remedial measures to control the hydrocarbon rich leachate from an above ground tip of gasworks waste including coal tar and coal dust. The second reviews measures designed to ensure safe operation of a large proposed tip involving the filling of a natural valley. To maintain natural drainage a fresh water brook running in the floor of the valley is culverted through a railway embankment which forms a barrage across the lower end of the valley and a boundary for the proposed filling operation. The final case history outlines the design and construction of remedial measures to the complex flow problem of groundwater and leachate in a limestone quarry used as a phased tip for domestic and commercial refuse.

INTRODUCTION

Aggregate filled drains, commonly known as "french" drains have been employed for many years to control groundwater. The water collected by these drains may be carried away by a perforated drainage pipe at the base of the trench. To prevent "silting up" of the carrier pipe by fines from the soil being drained, the grading of the aggregate fill is selected to act as a filter. As well as being problematical the construction of competent french drains, especially deep drains, is becoming progressively more expensive due to the increasing scarcity of suitably graded aggregates and the labour intensive nature of the operation. With this in mind the United States Joint Highway Research Advisory Council sponsored the development of a prefabricated fin drain in the late sixties. Research was conducted at the University of Connecticut culminating in the development of a fin drain comprising a thin vertical panel of water conducting core inserted into a longitudinally slotted pipe. The pipe and the core were encapsulated in a fine filter fabric which would allow the passage of groundwater whilst retaining soil fines that would otherwise clog the core. Water entering the vertical fin flows down the core under gravitational flow and enters the base pipe which carries the water to a suitable discharge point. In this early version of the fin drain, the filter fabrics used were either a polyester butterfly cloth with $O_{90} \cong 150$ µm or a polyamide chiffon with $O_{90} \cong 75$ µm. The water conducting core was a 3mm thick sheet of Armorweave expanded aluminium or more commonly vertical polyvinyl tube fencing, both with a vertical capacity of approximately 0.16 litres/sec/m. Although successful trials were conducted the fin drain did not appear to be a commercial success in the United States. In the early seventies, the fin drain notion was adopted by Ground Engineering Limited who developed the "Trammel" fin drain. This product was marketed and sold through the Laing subsidiary Geotextiles Limited. In 1983, Geotextiles Limited was purchased by Low Brothers of Dundee, who are now sole manufacturers of Trammel. On the home market, Trammel was followed by "Filtram" developed by ICI Fibres in collaboration with Netlon Limited. Many other fin drains have since been developed and marketed abroad. These include the Eljen drain and Miradrain from the USA, the Nylex Cordrain from Australia, the Culdrain from Japan, and the Enka drain from Holland. The latter is used largely for structural drainage and is marketed in the U.K. by MMG Erosion Control Systems.

THE TRAMMEL SYSTEM

Used as a french drain the system comprises a water conducting fin, which at its lower end encapsulates a perforated carried pipe, Figure 1. The fin is a lamina of a central water conducting core sandwiched between two geotextile filters. The trench in which the fin drain is installed is backfilled with the arisings from the trench. To ensure that the filter fabric can be correctly matched to the grading of the backfill four different mesh size fabrics are available. All four fabrics have a special high permeability woven structure with a polypropylene tape warp and monofilament weft, Figure 2. The mesh sizes, or nominal O_{90}, available are 150µm, 250µm, 430µm, or 640µm. Water permeability, normal to the plane of the geotextile filter, varies between 380 and 400 litres/sec/square metre under a driving head of 100mm of water. When associated with the core the permeability of the fabric drops by approxi-

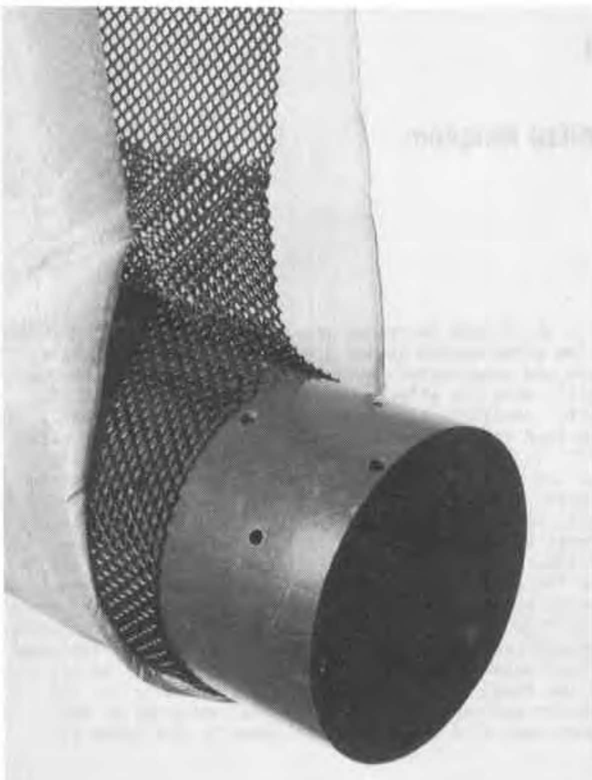

FIG.1. Revealed view of Trammel drainage system.

mately 30%. The core itself is a diamond pattern polyethylene geomesh comprising sets of parallel members approximately 6mm apart. The mesh has a special high capacity profile giving a total thickness of 3.5mm and a nominal transmissivity, or inplane flow rate, of 1 litre/sec. per metre run. As can be seen from Figure 3 the transmissivity decreases slightly as the lateral soil pressure increases. The Trammel system is delivered to site in component parts comprising rolls of core 2.0m wide and filter fabric, 3.1m wide, together with lengths of carrier pipe and fastening staples. This arrangement allows any convenient length and depth of drain to be made up on site as well as allowing the use of any convenient carrier pipe. Using this modular technique added versatility accrues since the fin drain can be easily adapted for use as a wall drain or a cut-off drain. When used as a cut-off, the geomembrane employed is generally 7.5m wide so as to reduce the number of joints.

THE FILTRAM SYSTEM

Again the system comprises essentially a water conducting core sandwiched between two sheets of filter fabric. In the case of Filtram the core is a sheet of polyethylene Netlon with a diamond shaped mesh formed of sets of parallel members approximately 10mm apart. The filter fabrics employed are generally Terram 1000 or the heavier gauge Terram 2000. Both are non-woven fabrics formed from continuous polypropylene/polyethylene filaments, which are thermally bonded to form a coherent structure, Figure 4. The O_{90} sizes are 100µm and 50µm with water permeabilities of the fabrics being 50 and 30 litres/sec. per square metre, respectively. Again the permeability of the composite is reduced by approximately

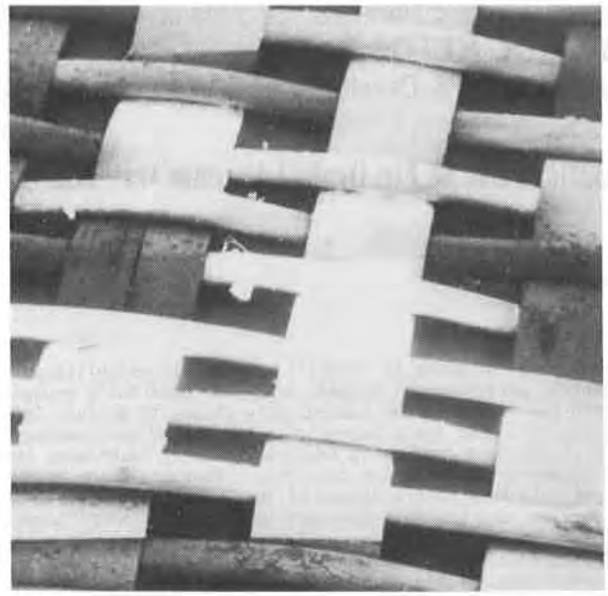

FIG. 2. Micrograph of Woven Monofilament on Tape Filter Fabric.

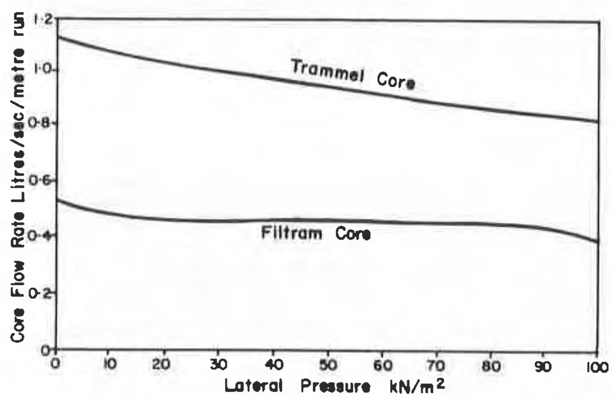

FIG. 3. Variation of Core Transmissivity with Lateral Pressure.

30% due to the presence of the core. Filtram is completely prefabricated with the Terram filter fabric being factory bonded to the Netlon core. The total thickness of the structure is 4.5mm which is associated with a nominal transmissivity of 0.5 litres/sec. per metre run. As with Trammel, the transmissivity decreases somewhat with increasing lateral soil pressure, Figure 3. The Filtram fin is delivered to site completely prefabricated either in rolls 25m long and 1.6m wide, or as 1.6m wide panels nominally 6m long. One of three methods may be used to give hydraulic continuity between fin and carrier pipe. Most simply the fin panels are slid into a precut slit running the length of a section of a non-perforated Polydrain. Although this is simple there is always a possibility of the split pipe crushing during backfilling. To obviate this the fin can be rested on top of an intact length of perforated pipe and subsequently connected to it by enveloping the pipe in a short

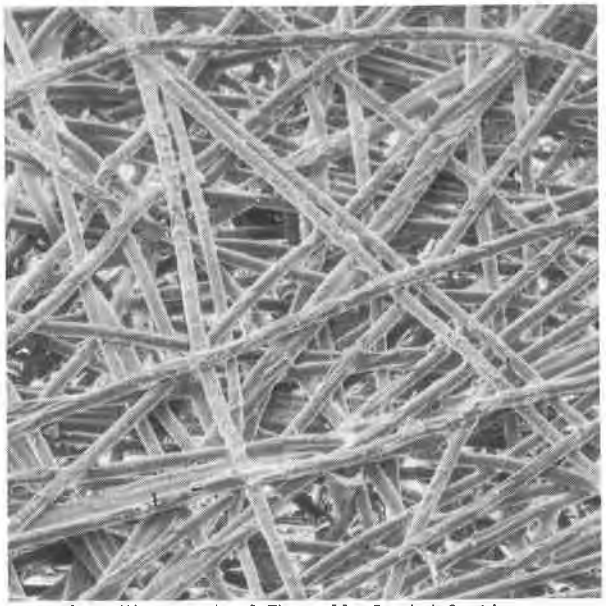

FIG. 4. Micrograph of Thermally Bonded Continuous Filament Filter Fabric.

length of Terram which is run a short distance up either side of the fin and connected to the fin by fixing pegs. The third option is to bed perforated pipe and the lower section of the fin in free draining granular material. In this fashion hydraulic continuity between the fin and the carrier pipe is via the granular fill. Despite factory prefabrication of the fin, the Filtram system is still versatile since the fin can be supplied with filter fabric on one side only, as might be used for wall drainage or with filter fabric on one side of the core and geomembrane on the other. This arrangement might be used as a cut-off drain or for wall drainage.

THE FIN DRAIN CUT-OFF

The disposal of waste by landfill is now well established. However, it is only in recent years that the associated potential environmental hazards have been fully recognised. The prime hazard is the generation of leachate within the fill that can subsequently escape to pollute local river and groundwater regimes. In many cases the natural "dilute and disperse" mechanisms in the ground surrounding the landfill do not afford adequate protection. Thus some form of leachate control is required. Early techniques involved total containment using vertical slurry cut-off walls and horizontal geomembrane cut-offs. However, these techniques can lead to problems of over topping of the slurry walls or possibly more severe damage if the geomembrane under the site is ruptured. Since water is the main leachate solvent a more positive approach to leachate control stems from the use of drainage installations which allow the separation of clean and polluted water. This can be achieved by using the fin drain cut-off where the cut-off is similar to the simple fin drain except the downstream filter fabric is replaced by a geomembrane, Figure 5. With this arrangement liquid can enter the fin via the upstream filter fabric but cannot flow across the downstream face of the drain. In this form the fin drain is capable of preventing surface water run-off, groundwater flow and spring issues from entering the landfill thus reducing the quantity of leachate produced. Also it may be installed around sites to drain leachate that would otherwise flow horizontally from the landfill

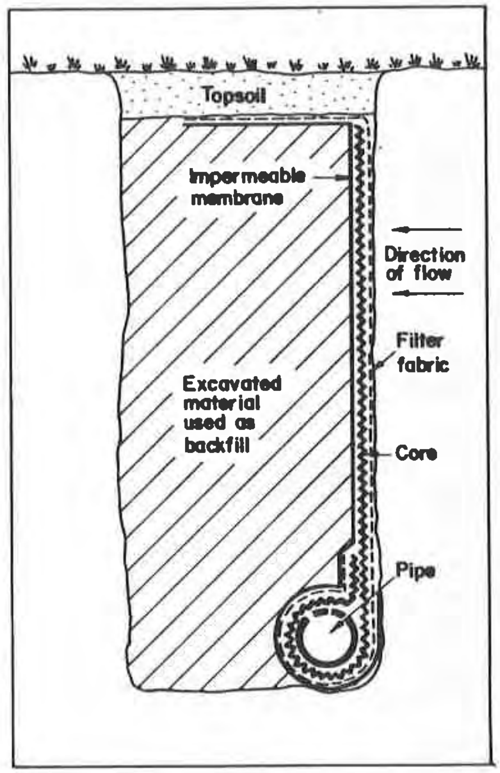

FIG. 5. Principle of Fin Drain Cut-Off

into the surrounding environment. The successful operation of a cut-off drain lies in the correct siting of the system, the ability to filter liquids from soils without clogging, and the matching of component materials to withstand known chemicals in the pollutant. An investigation of the geology and hydrogeology of the site is of prime importance in planning the overall drainage system layout. This should produce reliable information such as seasonal variation in water table contours, presence of geological sequences and logging of boreholes to determine the major hydrogeological controls over movement of any contamination and an estimate of the leachate likely to be formed. From this information is derived the positioning of the cut-off, required system performance data, depth of installation, design of filter and method of disposal or treatment of collected liquid.

The selection of the impermeable membrane will depend upon the character of the waste, soil conditions, performance required, design life and availability. The potential for interaction between many substances within a landfill which could contribute to leachate composition is large. It is, therefore, important that chemical analysis of the leachate be carried out in order to identify any chemical species that has known deleterious effects on certain polymers in order to select the appropriate material. So far as knowledge extends today, crystalline polymers have good all round resistance to chemical attack. An indication of the resistance fo chemical attack of common geomembrane polymers is given in Table 1.

TABLE 1 - Effects of Various Chemicals in Common Polymers

Chemical	Poly-ethylene	Polyvinyl-chloride	Poly-propylene
Acids (diluted)	U	U	U
Acids (concentrated)	U	U	U
Alkalies (diluted)	U	U	U
Alkalies (concentrated)	U	U	U
Ammonia liquid (0.88)	U	SA	U
*Metal salts	U	U	U
Benzene	F	F	F
Bleach	U	U	U
Brine	U	U	U
Carbon tetrachloride	F	F	F
Chloroform	F	F	F
Chlorine water	U	U	U
Cresols	SA	SA	SA
Diesel Oil	U	U	SA
Detergents	U	U	U
Developers, photographic	U	U	U
Edible oils	U	U	U
Fatty acids	U	U	U
Fuel oils	SA	SA	SA
Glucose	U	U	U
Glycerine	U	U	U
Lead tetraethyl	U	U	U
Lubricating oils	U	U	U
Methyl cellosolve	U	U	U
Mineral oils	SA	U	SA
Pesticides	U	U	U
Petrol (aromatic)	F	F	F
Phenol 90%	F	U	F
Selenium salts	U	U	U
Trichlorethylene	F	F	F
Turpentine	SA	U	U

* Arsenic, Cadmium, Chromium, Copper, Iron, Lead, Manganese, Mercury, Nickel, Zinc.

U = Unaffected
SA = Slight attack
F = Fails

N.B. Combinations of certain chemicals, although unaffecting plastics when in solution, may cause degradation when in an emulsion.

CASE HISTORIES

To illustrate the use of fin drain leachate cut-offs in the United Kingdom, three case histories are presented briefly.

1). This is a very simple case history relating to a low above ground tip where waste from a nearby gasworks was dumped. The tip was constructed by first stripping topsoil to a depth of approximately 200mm and stock-piling this for later capping of the tip. Waste was then tipped and spread to a thickness of approximately 1.5m. Once filled to capacity, the tip was capped off and seeded for use as a playing field. Several years after completion of the project it was found that grass over arable land adjacent to the southern boundary of the tip was dying for a distance up to 100m from the boundary with the waste tip. An investigation of the tip revealed waste comprising coal dust, coke and rubble. In large areas of the tip, especially close to its southern boundary, extensive deposits of coal tar were found layered within the coal dust, Figure 6. Excavation of trial pits at the boundary revealed a perched

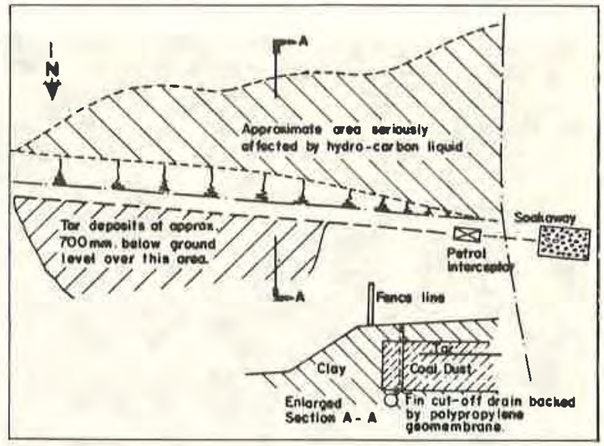

FIG.6. Plan and Section of Gasworks Waste Tip

water table within the tip which was seeping out at the toe of the tip. Since the tip was above original ground level, leachate seeping from the toe was free to flow over the surface of the arable land which fell to the south. Undoubtedly, the leachate was generated by surface water infiltration through the capping layer. Chemical analysis of the leachate revealed it to be rich in hydrocarbons for a distance of approximately 100m from the boundary. To remedy this problem, a 2m deep fin drain cut-off was installed along the southern boundary of the tip down slope of any tar deposits. The geomembrane, which was installed on the down slope side of the fin, was a 250µm thick LDPE with a unit weight of 230 grms/m². This extended from the surface of the tip down to the carrier pipe of the fin drain which was bedded in clay beneath the waste, Figure 6. The carrier pipe discharged into a soakaway at the western end of the pipe run. However, a petrol interceptor was installed immediately upstream of the soakaway to collect the hydrocarbon rich fraction of the leachate. The cut-off drain has performed satisfactorily since its installation in 1977 with the effects of the leachate on the adjacent arable land abating within twelve months of installation.

2). This relates to the drainage measures designed to prevent leachate escape from a proposed new tip for domestic refuse. The topography of the land, Figure 7, consisted of a natural river valley bounded at its lower end by a disused railway embankment. Down the middle of the valley ran a small brook, which was culverted through the railway embankment. The proposals for the new tip envisaged raising the ground level in the valley to that of the top of the embankment, a maximum of 15m at the middle of the valley.

The purpose of the drainage measures was to isolate the clean water in the brook from any contaminated water in the tip area. The primary method by which this was achieved was to culvert the stream throughout the entire length of the tip area and through the embankment. Although satisfactorily isolating the brook, this also removed the drainage path for the lower end of the tip area, thus making the area susceptible to flooding behind the railway embankment. This would have had three detrimental effects:

i) difficulties in moving plant in a waterlogged area

ii) leaching of contaminants from already tipped material with subsequent possible filtration through and under the embankment and back into the brook downstream

iii) a reduction in factor of safety of the embankment due to the high hydrostatic pressures.

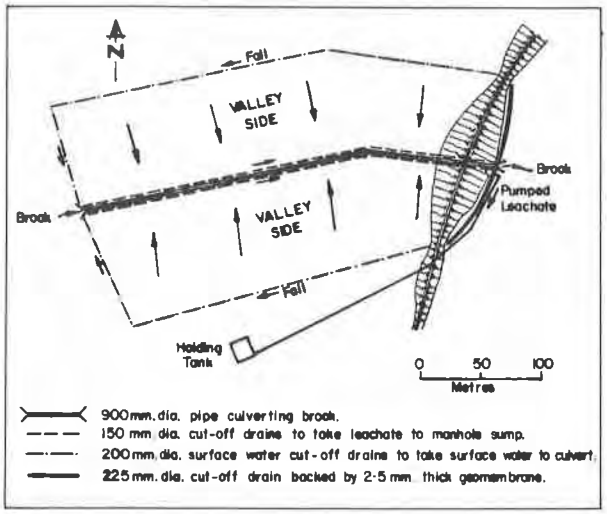

FIG. 7. Plan of proposed tip.

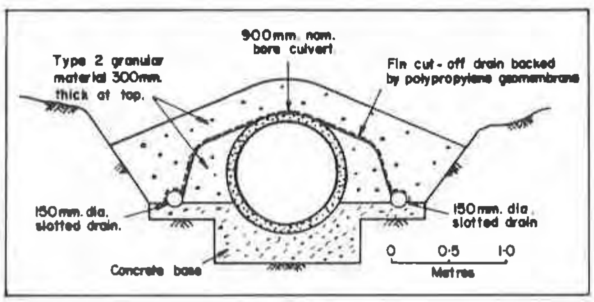

FIG. 8. Detail of cut-off drains to culvert.

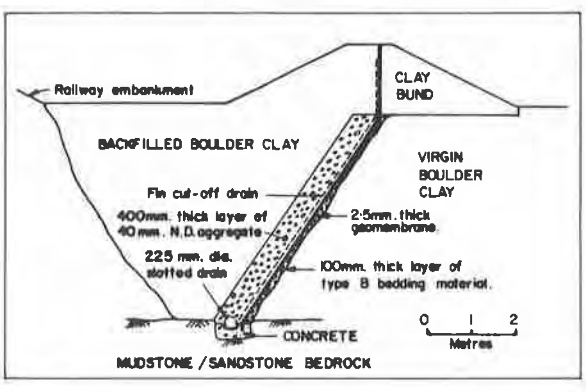

FIG. 9. Detail of cut-off drains to toe of railway embankment.

As a first measure the 900mm diameter culvert was draped in a fin drain cut-off incorporating a 200μm thick polypropylene geomembrane on the culvert side, Figure 8. The purpose of this was to intercept any natural groundwater flow to the piped stream, and hence prevent tracking directly along the outside of the culvert. This was to be avoided so as to prevent ingress of pollutants through any defects in the culvert, and also to halt any flows of water through the embankment along the outside of the culvert. The carrier pipes from the fin cut-off drain were fed into a manhole, from which the contaminated water could be pumped away.

The second preventative measure consisted of forming a complete vertical cut-off at the downstream end of the site. The ground conditions beneath this part of the site comprised approximately 4-6m of boulder clay over laminated mudstones and silty sandstones. It was considered that the mudstones/silty sandstones would provide a suitably impervious layer to vertical filtration, but that the boulder clay, with large lenses of sand and gravel, would allow filtration of pollutants beneath the embankment and back into the water course. It was, therefore, decided to place an impermeable curtain through the boulder clay down to bedrock. This consisted of a 2.5mm thick high density polyethylene geomembrane toed and concreted into the bedrock with a fin drain on the upstream side, Figure 9. The geomembrane was torch welded at the joints to ensure a good seal. The carrier pipes for the fin drain were also led into the manhole, from which the contaminated water could be pumped away.

The final preventative measure was to place a shallow fin drain with surface water interceptor along the valley sides above the tip area. The purpose of this was to intercept any surface water coming down the valley sides and redirect it into the brook just before it entered the culvert.

3). The final case history relates to a large limestone quarry where there is phased infilling of the worked out areas. Since the quarry is still active, there is simultaneous filling in abandoned areas whilst quarrying operations continue on the active faces. The quarry, which is generally 15m deep, is in highly fractured oolitic limestone underlain by fullers earth clay, which is assumed impermeable. An idealised plan of the quarry is shown in Figure 10.

Problems arose when toxic leachate was found to be springing from the lower sections of the active quarry workings in the east of the quarry. There was also ponding of leachate on the exposed quarry floor which led to the development of a large lagoon at the NW quarry face. The leachate in this lagoon was seen to be entering the fractured limestone through which it ultimately flowed to a nearby watercourse causing severe pollution. This situation compelled the quarry owners to commission an investigation and design of remedial works.

An investigation was duly carried out and this revealed that the fractured limestone dipped towards the north as did the surface of the underlying clay. Several piezometers were installed and these indicated a general northerly groundwater flow through the limestone which entered the Phase I landfill. This was confirmed by several series of dye traces which exposed three primary seepage paths for the leachate. Firstly there was seepage of leachate directly into the quarry face where the landfill and quarry face were in direct contact. This was one source of downstream pollution. Secondly, leachate was travelling down through the poorly capped landfill under gravitational flow until it met the impermeable clay. From here the leachate flowed horizontally crossing the exposed quarry floor, to form a leachate lagoon, Figure 10. As the lagoon filled it finally entered the foot of the NW quarry face, Figure 11, to flow on towards a water course to the north. The final seepage path was from the eastern corner of the Phase I fill, through the spur of limestone carrying the access road, and out through the face of the quarry workings whence it flowed across the quarry floor to discharge into the lagoon which was a low point in the quarry floor.

The proposed remedial works comprised three phases. In the first phase a deep trench was constructed to expose

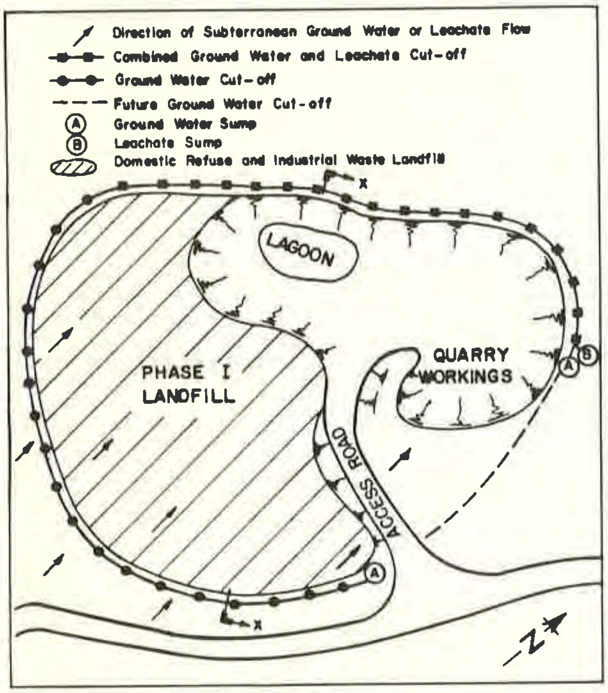

FIG. 10. Plan of Quarry.

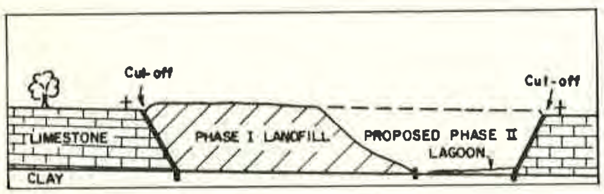

FIG. 11. Idealised Cross-section of Quarry.

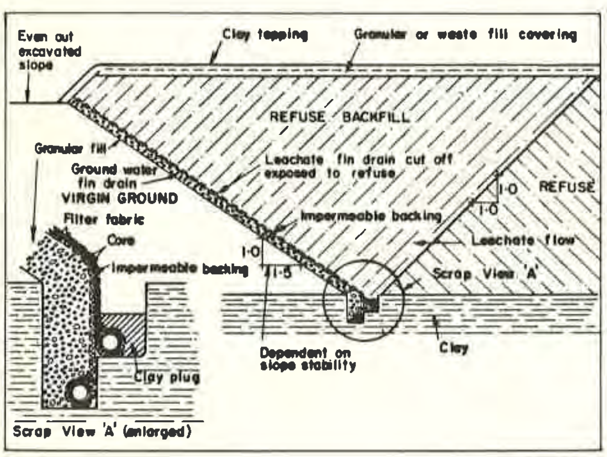

FIG. 12. Detail of leachate and groundwater cut-offs.

the southern quarry face buried beneath the Phase I landfill. The trench was taken down to the level of the fullers earth clay with the Phase I landfill to the north battered back at 1 : 1. The exposed quarry face was trimmed and battered back at 1 : 1.5 following which it was dressed with 300mm of free draining granular fill. This face was covered with a monofilament on tape filter fabric, with a mesh opening of 240µm, followed by a water conducting core which was finally covered with a 230 grms/m² LDPE geomembrane. This arrangement acted as a groundwater interceptor cum cut-off with the groundwater being conducted down the fin to a carrier pipe. The carrier pipe was laid in a 1.5m deep slit trench in the underlying clay. This drain run, carrying the intercepted fresh groundwater, was led to a sump which finally discharged into a conventional soakaway. The excavated landfill was replaced and capped with clay underlain by a layer of granular material.

A similar operation was carried out on the NW quarry face most of which was still exposed. The installation of the groundwater interceptor here was to cut-off any groundwater that might deviate from the general northerly flow. Once this cut-off had been installed, a second water conducting core was laid on the geomembrane followed by a layer of filter fabric, Figure 12. The function of this drain was to intercept the leachate from the existing Phase I landfill and Phase II landfill when placed. This arrangement of a groundwater cut-off backing onto a leachate cut-off extended round the whole NW quarry face and part of the NE face as shown in Figure 10. Where the fin drain cut-off was exposed on the yet unfilled quarry face, it was covered with a 300mm thickness of free draining granular fill to prevent ultraviolet degradation. The flow from the two separate cores were collected in two separate carrier pipes. The inner carrier pipe, Figure 12, took the leachate and therefore the pipe was surrounded with a plug of puddle clay to prevent leakage into the outer carrier pipe which contained fresh water from the groundwater cut-off. The groundwater was led to a sump which eventually discharged via a conventional soakaway. The leachate was led to a separate sump and it is planned to spray this on the surface of the landfill so aerating the leachate. It is understood that this repeated aeration of the leachate leads to degradation of the leachate so reducing toxicity to an acceptable limit. The fact that the vast majority of the leachate solvent was groundwater entering through the southern quarry face means that the recurring leachate can only be recharged by any vertical seepage through the capping layer placed over the surface of the landfill. The final section of the quarry is currently being worked out. Once this is achieved, a final section of groundwater cut-off is to be installed at the eastern face as shown in Figure 10.

Although the final performance of the remedial measures cannot be assessed until the quarry is finally filled to capacity and completely enveloped by the various fin drain cut-offs, it appears that the groundwater interceptor cum cut-off has been totally successful. This is reflected by the fact that the leachate lagoon, which was recharged by leachate produced by groundwater and surface water penetrating the landfill, has now virtually dried up. As a corollary to this, the leachate pollution observed downstream of the quarry has abated completely.

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Denver, U.S.A.

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Design of Drain Beneath Geomembranes: Discharge Estimation and Flow Patterns in the Case of Leak

The use of geomembranes in hydraulic structures necessitates estimation of the discharge and of the flow pattern when a leak occurs.

In a first part, it is supposed that any drain is layed beneath the geomembrane. This hypothesis may be consider according to the nature of the underlaying soil and the security conditions. Generally it is better to design a draining layer to follow the seepage loss evolution and to evacuate it. Different boundary conditions are considered and charts give the discharge Q as a function of the geometric parameters (water depth, size of the leak, thickness of the soil below the geomembrane, etc.).

In a second part, the seepage loss is estimated when the impermeable membrane is layed on a drain. The flow across the geomembrane is depending not only of the size of the leak but also of the permeability of the drain. Charts are given and permit designing the drain transmissivity according to a maximum porewater pressure allowed inside it.

INTRODUCTION

When waterproofing of hydraulic structures has to be realized, geomembranes are used frequently for reasons of commodity and for their very low permeability. But due to their small thickness, leak might occur easily as a result of rupture or imperfect welding and thus the sheet does not give guaranties of impervious barrier. The existence of a leak may be catastrophic for soil pollution or for structure stability. Also it is necessary to anticipate risks of leak and to be able to estimate the discharge and the flow width when it occurs.

Two cases are considered : either the geomembrane is layed directly on the soil (first part) or placed on a draining layer (second part).

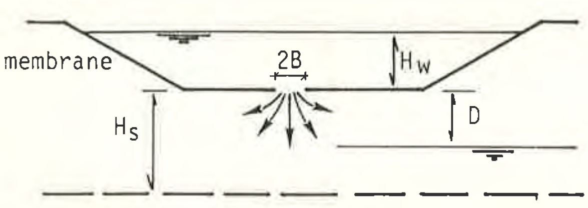


Fig. 1 : Considered problem and parameters definition

I. GEOMEMBRANE LAYED DIRECTLY ON THE SOIL

In this part is presented the case of a slot, of small width 2B and of infinite length (two dimensionnal problem), in an impermeable sheet at the horizontal bottom of a basin containing a water height H_w (fig. 1).

The flow takes place in the soil of thickness H_s below the geomembrane and will depend of the permeability of the soil, the depth of the water table and the boundary conditions : very permeable or imprevious barrier at depth H_s.

Calculations considering steady state flow in the saturated part of the soil* permit easily treatment of both cases.

I.1. Homogeneous soil limited by a very permeable front (fig. 2)

The permeability of the underlaying soil (at depth greater than H_s) is sufficient ot assurme that its upper limit, with the soil supporting the geomembrane, is an isobaric line of constant pressure $u_o = H_s - D$. The hydraulic head difference Δh between the upper line of the flow (at crack level) and the lower line (at depth H_s) is given by :

$$\Delta h = H_w + D$$

* Generally, the soil is not saturated at the principle of the flow. The effects of the no-saturation have been studied and presented before (1) : the discharge is more important but decreases rapidly when water depth H_w is large.

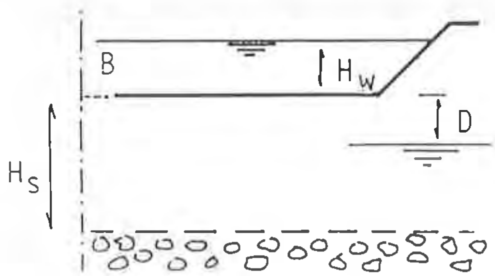


Fig. 2 : Very permeable boundary at depth H_s

The obtained results are presented on the figure 3. They show the variation of the discharge Q with the width 2B of the slot :

For $H_w/H_s > 5$, the ratio $Q/k_s\Delta h$ is depending only of the parameter B/H_s : the discharge is proportional to Δh, whatever the value of H_w/H_s may be.

a. Influence of the width of the slot (fig. 3) :

the width of the slot has a very small effect on the discharge when the leak is small and particulary if $H_w/H_s > 1$: $Q/k_s\Delta h$ varies from 0.4 to 1 when B/H_s is multiplicated by 100 (from 10^{-3} to 10^{-1}). Therefore it is not necessary to know accuracely the value of B and this is very interesting because it is not easy to estimate it.

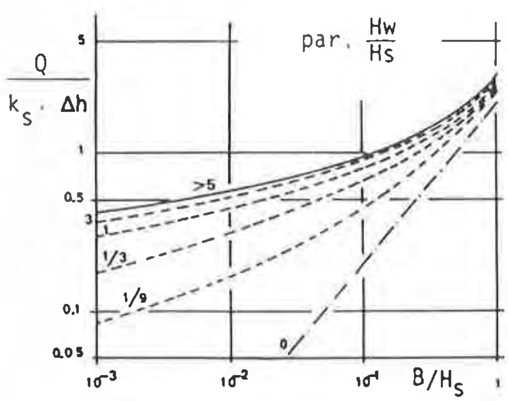


Fig. 3 : Discharge Q as a function of B for different ratio H_w/H_s

b. Influence of thickness H_s of soil under the geomembrane (fig. 4)

When increasing H_s, the discharge decreases rapidly to a value corresponding to an infinite thickness soil (that is verified elsewhere (2)).

Consequently, it is not necessary to realize important thickness of soil below the geomembrane in order to reduce the seepage loss : a minimum value of H_s/H_w about 0.2 (for a thin slot) will reduce the flow as much as an infinite thickness.

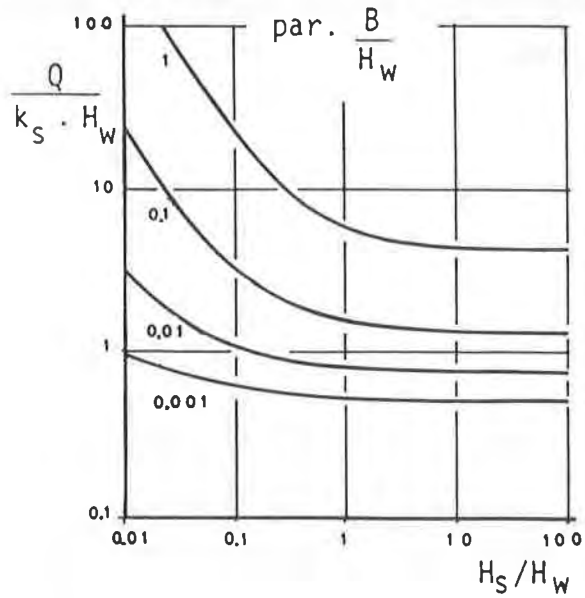


Fig. 4 : Influence of thickness H_s

c. Influence of the depth of the ground water table (fig. 5) :

the depth of the ground water table has a little influence on the discharge. It may be supposed that $D/H_s = 1$.

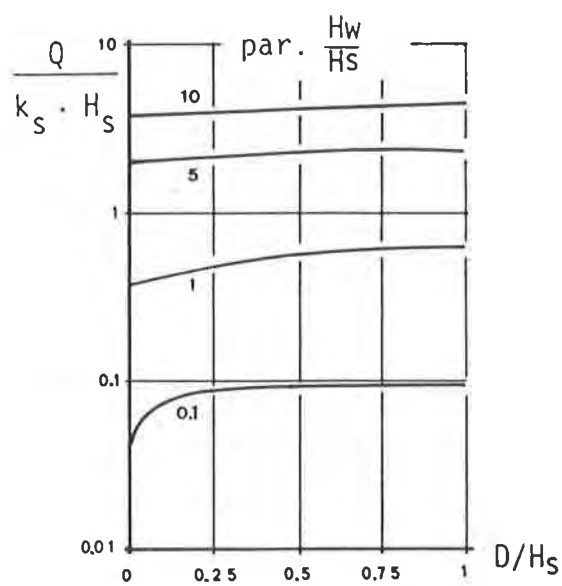
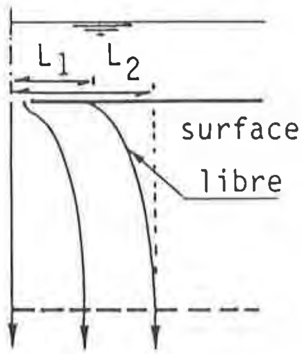


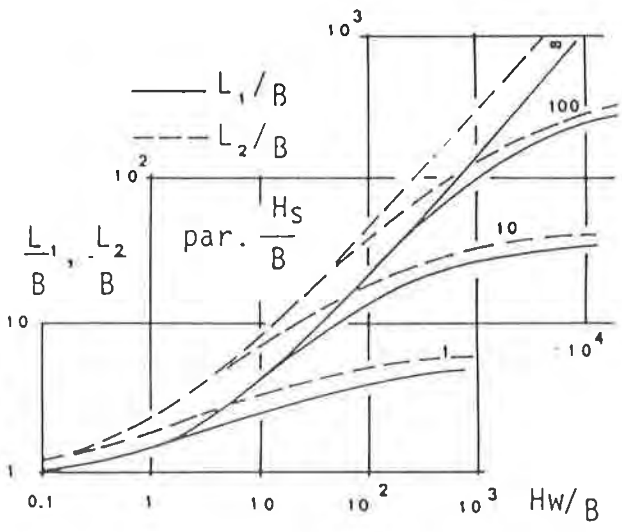
Fig. 5 : Influence of depth of groundwater table ($B/H_s=10^{-3}$)

d. Widths of the flow (fig. 6) :

the half widths L_1 of the flow, below the geomembrane, and L_2 at pervious boundary may be determined from fig. 6 when there is no ground water ($D/H_s = 1$).



a. Definition of L_1 and L_2



b. Variations of L_1 and L_2

Fig. 6 : Widths of flow

1.2. Homogeneous soil limited by an impervious stratum

In order to simplify the problem, only flows like that presented on figure 7 is considered : entirely saturated soil, symmetrical flow from the leak, vertical equipotential at a distance L.

a. Influence of the length L on discharge Q (fig. 8) :

increasing L the discharge tends assymptotically to the value of a monodirectional flow of width H_s and of length L, particularly for small leaks. Also in this case, the discharge is little affected by the width of the slot.

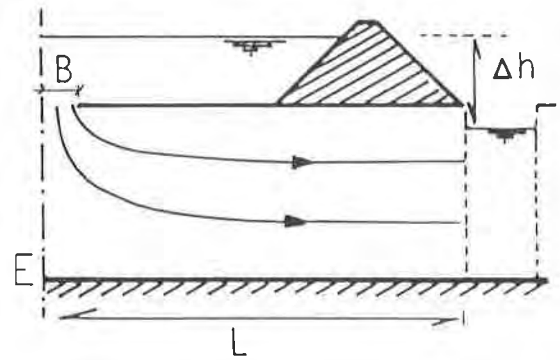


Fig. 7 : Impervious stratum limiting the soil beneath the geomembrane

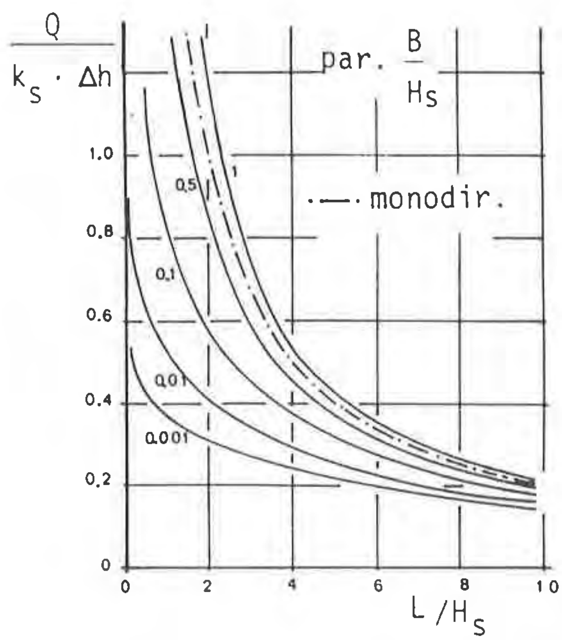


Fig. 8 : Influence of length L on discharge Q

b. Hydraulic head at point E (cf. Fig. 7)

the hydraulic head at point E is done (2) by the relation

$$h_E = -D + \Delta h . G(B, L, H_s)$$

where $\Delta h = H_w + D$
and G a function of the geometric parameters.
The graph of G is presented on the figure 9 as a function of B/H_s and L/H_s.
G tends to 1 when L/H_s increases : it confirms that the flow becomes monodirectionnal.

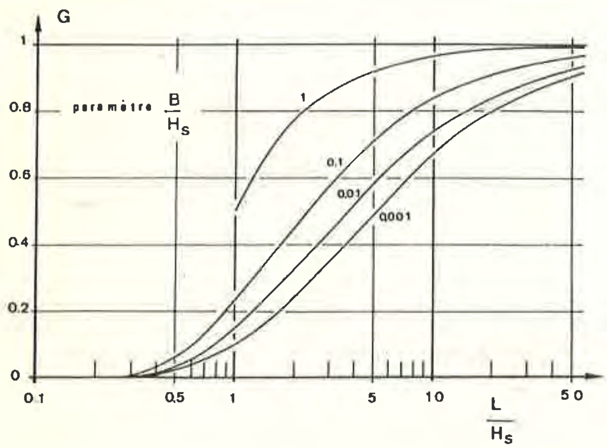


Fig. 9 : Variation of G function giving hydraulic head at point E

II. GEOMEMBRANE PLACED ON A DRAINING LAYER :

In the first part, the seepage loss has been estimated when the draining layer is missing. It is preferable to place one : it reduces the risk of seepage in the soil and consequently the risk of pollution or instability of the structure. Mainly it allows to follow the seepage loss variations and may prevent an accident.

The draining layer may be of granular medium or a geotextile (or a grid). Geotextile, as geomembranes, have the advantage to be placed easily on slopes.

The drain is determined by its transmissivity θ (θ = k.H : k permeability and H thickness of the drain) and have to be much more permeable than its supports in order to drain efficacely. That is obtained really if the support of the draining layer is a geomembrane. But this second sheet could be no more impervious than the first. The seepage in the soil below is the less important as the pore water pressure is low in the draining layer. It is the same problem when the drain is set on the soil directly. Giroud (3) proposed to determine the transmissivity such as the pore water pressure should be zero in the drain.

Classical draining system can work like "perfect drain" (infinite permeability) by the side of a fine soil or a geomembrane. But, in the case of a leak, the water-head loss inside the drain is not always negligible compared with that of the opening. The discharge have to be limited by the leak itself and not by the drain transmissivity.

This aspect is presented here : under what conditions the water head loss in the drain is negligible for a given opening and what is the required transmissivity.

II.1. Discharge through an opening

The flow rate through an opening of area A can be calculated by the relation :

$$Q/A = \sqrt{2g\Delta h}$$

Δh : piezometric difference between the two sides of the leak.
The water head loss by friction on the opening sides is neglected.

II.2. Slot of infinite length (bidimensionnal problem) :

The geometric parameters are presented on figure 10 :

H_w : water height in the reservoir
2B : slot width at depth z
θ : transmissivity of the drain (thickness H and permeability k)
L : distance between the leak and the collector pipe.

The collector pipe is supposed at the same height as the basin bottom.

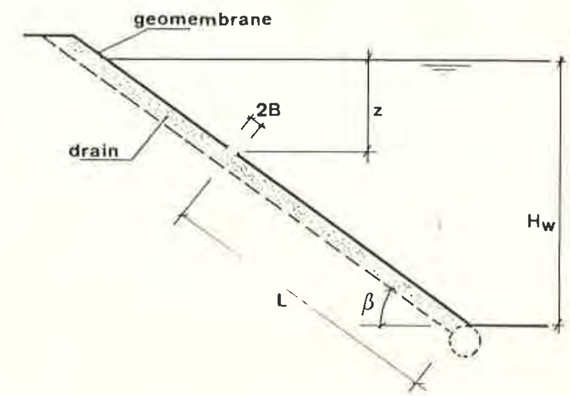


Fig. 10 : Draining layer beneath the geomembrane

Water flow in the drain : generally, the thickness of the drain is very small in comparison with the distance L (H << L). According to § I.2 the flow is quite equivalent to a monodirectionnal flow. Its discharge is given by :

$$Q = \theta\ (H_w - \Delta h)/L \text{ per unit length of leak}$$

it is equal to the discharge passing through the slot

$$2B\ \sqrt{2g\Delta h} = \theta\ (H_w - \Delta h)/L$$

If we express the maximum pore water pressure u in the drain at the exit of the opening) :

$$u/\gamma_w = z - \Delta h$$

γ_w : specific weight of water

we obtain

$$\frac{\theta \sin \beta}{2B\sqrt{2gH_w}} = \frac{1}{1 + \frac{u}{\gamma_w (H_w - z)}} \left[\left(1 - \frac{u}{\gamma_w z}\right) \frac{z}{H_w} \right]^{\frac{1}{2}}$$

The figure 11 represents the variation of this highest porewater pressure u for different depths z of the leak :

Deeper the slot is, more rapidly decreases u when increases the transmissivity.

A drain with a porewater pressure equal to zero (flow due to gravity only) in the drain is designed by :

$$\theta = \frac{2B\sqrt{2gz}}{\sin\beta}$$

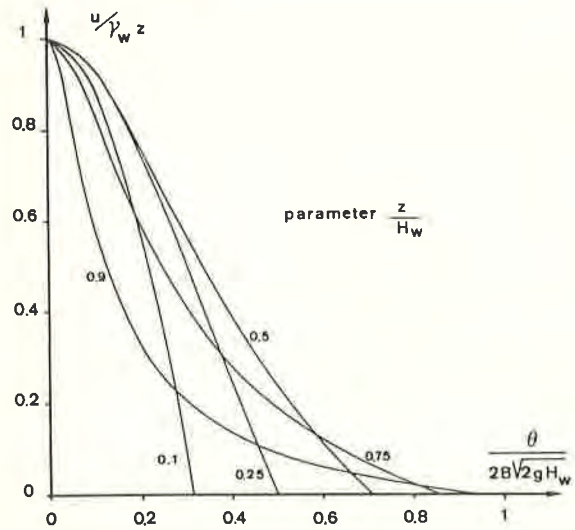


Fig. 11 : Variation of pore water pressure u
(slot of width 2B)

II.3. Circular leak :

The discharge in the drain can be calculated using flownet of a source (diameter 2B) towards a linear well (fig. 12)

$$Q = 2\pi k.H \frac{H_w - \Delta h}{L_n(2L/B)}$$

(once more, supposing collector pipe at depth H_w).

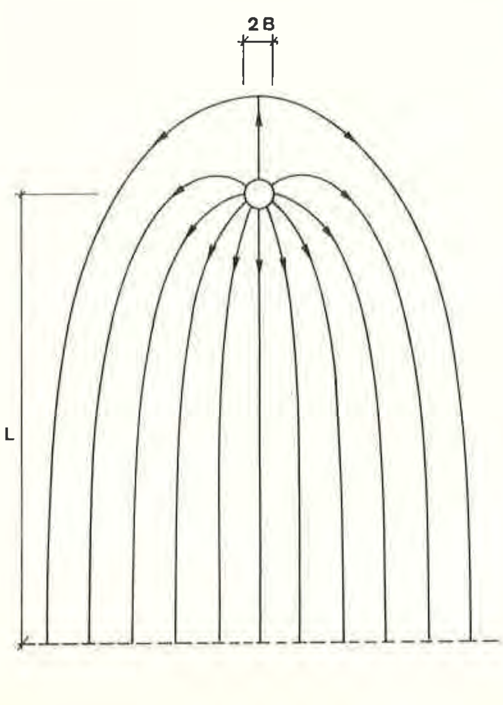


Fig. 12 : Flow from a circular source to a linear well

The transmissivity is given by :

$$\theta = \frac{B^2\sqrt{2g\Delta h}}{2(H_w - \Delta h)} \cdot L_n \frac{2(H_w - z)}{B \sin\beta}$$

Charts presented on figure 13, allow the determination of θ as a function of z and B for a given water height H_w, for two cases : u = 0 (Δh = z) and $u = \gamma_w z/2$ (Δh = z/2). There is a ratio of about 10 between the required transmissivities. This chart has been calculated for β = 30°, but this parameter have not a great influence on θ value.

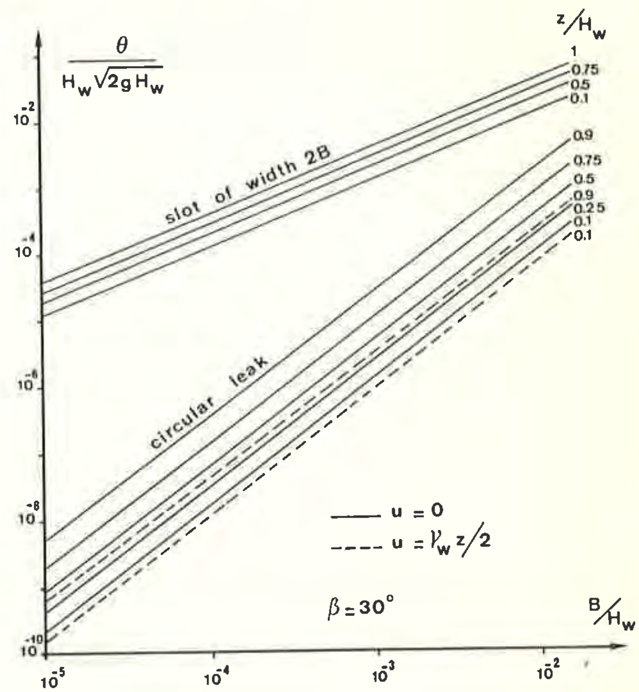


Fig. 13 : Required transmissivity θ for u = 0 and $u = \gamma_w.z/2$

CONCLUSION

Results presented here allow to provide for the discharge when a leak occurs in a geomembrane used with different conditions : geomembranes directly placed on the soil or on a draining layer. The remaining problem is the determination of the opening shape (as a slot or an circular aperture) and the probability that leaks exist and how many. Nevertheless, some remarks aid to understand the mecanism and the parameters influencing the seepage loss :

- when the leak is small, the size of the opening have a little influence on the flow ;
- a minimum thickness of soil under the geomembrane is sufficient to reduce seepage as much as an inifinite thickness ;
- when a draining layer is placed below the geomembrane, deeper the slot is, more rapidly decreases the pore water pressure when increases the transmissivity ;
- For an opening of definied shape, charts give the transmissivity of the drain in such a way that pore water pressure would be equal to zero.

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Design of Geomembrane Liner for the Proton Decay Experiment

Design considerations and theoretical analyses of a water reservoir for the Proton Decay Experiment are presented. The design embodies the site location/conditions, operational considerations, and requirements imposed by the physics experiment itself. Compatibility of the geomembrane with the ultra-pure water used for the experiment led to the selection of HDPE. The reservoir location (600 m underground in a salt mine) imposed stringent conditions on the maximum leakage allowed due to the risk of salt solution. Hence, a double HDPE lining system with a monitored double drainage system was designed, installed, and put into operation. The double liner concept and the associated drainage system are discussed in detail. Theoretical analyses were performed to anticipate the mechanical behavior of the liner. Laboratory tests, which simulated the expected stresses on the liner, complemented the results of the theoretical analyses. Consequently, the final reservoir design proposed measures to alleviate stresses.

INTRODUCTION

Matter may not be forever. The lifespan of protons may be 10^{32} years according to a theory for which the Nobel Prize in physics was granted in 1979. To verify this theory, a group of physicists at the Universities of California and Michigan have set up the Proton Decay Experiment, sponsored by the U.S. Department of Energy. The experiment consists of monitoring a 18m × 24m × 20m deep reservoir containing 8500 m^3 (2.2 million gallons) of water, i.e., approximately 10^{33} protons. If the theory is correct, only a few of the protons in the reservoir would spontaneously disintegrate, or "decay", each year, giving off ultraviolet light detected by 2048 photomultiplier tubes immersed in the water. Transparency is essential and the water is permanently recycled and purified by a sophisticated facility adjacent to the reservoir. Detection of proton decay should not be disturbed by natural radioactivity and cosmic radiation. Hence, the reservoir has been excavated in salt, where natural radioactivity is low, and is located 600 m (2000 ft) underground where cosmic radiation is minimal.

Concrete and steel reservoirs were considered first, but were eliminated because of the cost associated with difficult access to the reservoir. Consequently, a geomembrane liner was considered. The design of the geomembrane lining system for the Proton Decay Experiment reservoir was an exceptional challenge due to the stringent requirements discussed in Section 1 of the paper. Sections 2 and 3 discuss the two most delicate aspects of the design: the leakage collection/detection system and the mechanical behavior of the liner.

1. DESIGN APPROACH

1.1 Operational Requirements

The following requirements were dictated by the physics experiment:

• Water Purity. The degree of water purity required by the Proton Decay Experiment is several times more stringent than current distilled water standards.

• Water Level. During the operation of the experiment, the water level should remain nearly constant with a tolerance of 0.2 m (8 in.), during several years. Emptying the reservoir for repair or maintenance would not be critical to the experiment but would require a considerable amount of time (it takes five weeks to empty and five weeks to fill the reservoir through a pipe going to the ground surface).

• Reservoir Depth. The volume of the reservoir (8500 m^3 or 11000 cu.yd.) was dictated by the number of protons to be stored. The shape of the reservoir needed to be massive (i.e., almost equal in the three directions) to optimize measurements. Consequently, the depth of the reservoir had to be of order 20 m (70 ft). As a result, a high water pressure was exerted on the liner.

The above operational requirements resulted in the following requirements for the liner:

• Geomembrane/Water Compatibility. Release of chemicals (eg. plasticizers) by the geomembrane in contact with the extremely pure water should be minimal. Tests conducted at the University of California-Irvine led to the selection of a high density polyethylene geomembrane.

• Maximum Leakage. The rate of leakage through the liner should not affect the water level more than indicated above. It was determined that leakage rate should be under 40 liters (10 gallons) per minute to be compatible with the pumps evacuating leakage to the ground surface and the capacity of the purification facility to recycle leakage.

1.2 Site Conditions

In addition to the stringent requirements dictated by the physics experiment, equally stringent requirements were dictated by the following site conditions:

• Risk of Salt Solution. A large portion of the cavity walls consist of massive crystalline salt. Pure water in contact with salt could result in serious damage to the walls or even collapse of the cavity. Consequently, any water leaking through the liner should be collected and prevented from direct contact with the salt.

• No Emergency Storage Capacity. If it appears necessary to empty the reservoir, for repair or any other emergency, this can be done only through a pipe going to the ground surface, 600 m (2000 ft) above. It would have been extremely expensive to excavate another large cavity only for emergency purposes and it was not possible to spill water into the adjacent salt mine. Consequently, the conceptual design had to be such that even if a major leak occurred, the leaking water would remain contained in the reservoir area for the duration of the emptying operation.

• Irregular Shape of the Cavity. The cavity was being excavated when the liner design started. It was too late to change the shape of the cavity and the method of excavation. Consequently, the following geometrical conditions were imposed for the design of the liner: (i) cubical shape with rather angular corners (rounded corners generate more uniform stresses); (iii) wall irregularities caused by the excavation technique and large irregularities at the connection between the walls and the bulkhead which closes the access ramp used by excavation equipment; and (iii) chainlink mesh rockbolted to the walls to prevent spalling of salt or shale.

1.3 Design Considerations and Requirements

The above operational requirements and site conditions lead to three major design considerations:

• Geomembrane Selection. As mentioned above, geomembrane selection resulted primarily from pure water/geomembrane compatibility.

• Leakage Collection. Zero leakage is impossible to guarantee with any type of liner. Therefore, to meet the above mentioned stringent requirements regarding leakage, the following was done: (1) through liner quality control, ensure that leakage from the reservoir is under 40 liters (10 gallons) per minute; and (2) design a leakage collection system which: (i) prevents water/salt contact, (ii) prevents external "dirty" water from coming into the reservoir, and (iii) is able to contain leaking water for thirty days if leakage rate exceeds pumping capacity. Leakage collection design is discussed in Section 2.

• Liner Mechanical Behavior. The liner is subjected to high stresses as a result of wall irregularities and high water pressure. A design was conducted to evaluate stresses, propose measures to alleviate excessive stresses, and verify that the selected geomembrane would withstand remaining stresses. Mechanical behavior of the lining system is discussed in Section 3.

2. LEAKAGE COLLECTION

2.1 Double Liner Concept

A lining system should always be designed assuming that there will be a leak (1). A single liner was not deemed adequate because it would have been unable to prevent leaking water from being in contact with salt.

Description of Double Liner. A double liner with two drainage systems was selected (Fig. 1). The internal drainage system collects water leaking from the reservoir through the inner liner. A pump, located in the "clean sump", sends this water to the purification facility. If this pump is operated on a regular basis, water does not accumulate in the clean sump or the internal drainage system. As a result, the outer liner is subjected to negligible water pressure. Leakage through the outer liner should therefore be negligible, unless there are large holes at the bottom of the outer liner where water leaking through the inner liner flows.

The external drainage system collects: water that might have leaked through the outer liner; ground water; and water that could leak from the supply pipe or the adjacent purification facility. The water collected by the external drainage system is too dirty to be recycled. A pump located in the "dirty sump" sends it to the ground surface. Being promptly evacuated, the external dirty water is thus prevented from entering the internal drainage system through holes in the outer liner.

Functions of the Double Liner. The double lining/double drainage system was selected because it nearly meets all design requirements mentioned in Section 1.3: viz. (i) it prevents almost completely clean water from being in contact with salt; and (ii) it prevents dirty water from reaching the internal drainage system and the reservoir.

In addition, the vertical shafts surmounting the two sumps would hold water in case of contingency. If leakage through the inner liner exceeds the recycling capacity,the water level would rise in the "clean shaft" to the same level as in the reservoir, thereby balancing pressure and stopping leakage. However, leakage could then occur through the outer liner because it would become subjected to water pressure. This leakage would be collected by the external drainage system and conveyed to the dirty sump. If this leakage exceeds the capacity of the "dirty pump", the level of water would rise in the "dirty shaft", stopping all leakage but putting a large amount of water in contact with the salt. In that case, the reservoir should be emptied and water in contact with the salt should be evacuated. In this situation, the dirty water can be pumped back into the reservoir during the emptying operation, thereby preventing or minimizing damage to the salt cavity.

Therefore, it was recognized that a double liner did not provide a guarantee that the reservoir will never be emptied due to leakage. However, as mentioned in Section 1.1, interrupting the experiment is not critical. The double lining system also provided for leakage detection. This has proved to be essential (2, 3).

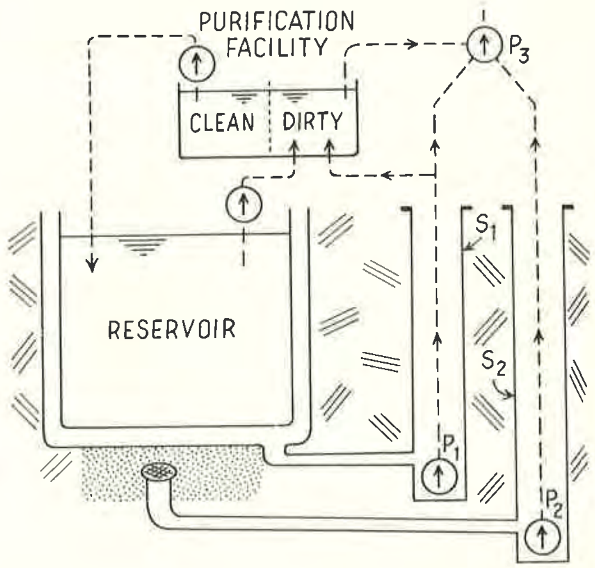

Fig. 1. A double lining system. Legend: P1, pump in "clean sump"; P2, pump in "dirty sump"; P3, pump rejecting dirty water to ground surface; S1, "clean shaft"; S2, "dirty shaft".

2.2 Leakage Evaluation

The following values of leakage through the inner liner were determined using equations given in (4).

- 0.06 liters/minute due to geomembrane permeability (assuming a coefficient of permeability of $k=10^{-13}$ m/s which is probably conservative for HDPE). The thickness of the geomembrane is 2.5 mm (100 mils).

- 15 liters/minute if there is one pinhole (diameter 0.1 mm) per m^2 ,i.e., a total of 2100 pinholes.

- 50 liters/minute for one 10 mm diameter hole and 5 liters/minute for one 3 mm diameter hole, located in the lower half of the reservoir. These two values were calculated assuming a free draining medium outside the inner liner. They should be reduced, considering that such flow rates would locally saturate the internal drainage system.

Therefore, it was concluded that quality control of the installation, particularly the seams, would be important to ensure that the maxiumum allowable leakage rate of 40 liters/minute is not exceeded.

2.3 Selection of Drainage Material

To design the internal drainage system, it was conservatively assumed that leakage through each of the four wall liners would be 25% of total allowable leakage,i.e., 10 liters/minute (1.7×10^{-4} m^3/s) through each wall liner (in reality a fraction of the leakage could have been expected through the floor liner). Therefore, the required hydraulic transmissivity of the drainage system on a 18 m wide wall is :

$$1.7 \times 10^{-4} \div 18 = 1 \times 10^{-5} \text{ m}^2/\text{s}.$$

On the floor, where the slope was approximately 4%, the required hydraulic transmissivity at the base of the wall opposite to the sump is:

$$1 \times 10^{-5} \div 0.04 = 2.5 \times 10^{-4} \text{ m}^2/\text{s};$$

and the required hydraulic transmissivity near the sump, where the flow concentrates, was found to be 4×10^{-3} m^2/s.

Gravel was considered as a draining material for the internal drainage system on the floor, but gravel was deemed too radioactive and too cumbersome to transport to the underground reservoir. Synthetic drainage layers such as geotextiles, plastic mats and plastic nets were considered for the floor and the walls. Geotextiles did not have enough hydraulic transmissivity and mats were too compressible, losing a large fraction of their initially high hydraulic transmissivity under the design compressive stress (200 kPa, i.e., 20 m of water). The high density polytehylene net tested exhibited little compressibility and had a hydraulic transmissivity of 4×10^{-4} m^2/s. Tests to evaluate hydraulic transmissivity under compressive stress had been conducted at the University of Michigan, using an apparatus similar to the one described in (5).

Comparing required and measured hydraulic transmissivities, it was decided to use the following amount of plastic net: strips 0.15 m (6 in) wide every 1.5 m (5 ft) on the walls; a number of layers on the floor progressively increasing from 2 at the toe of the wall opposite to the sump to at least 10 in the sump area.

Nets, protected from salt and shale dust by a geotextile filter, were also used under the outer liner for the external drainage system. Also, reclaimed

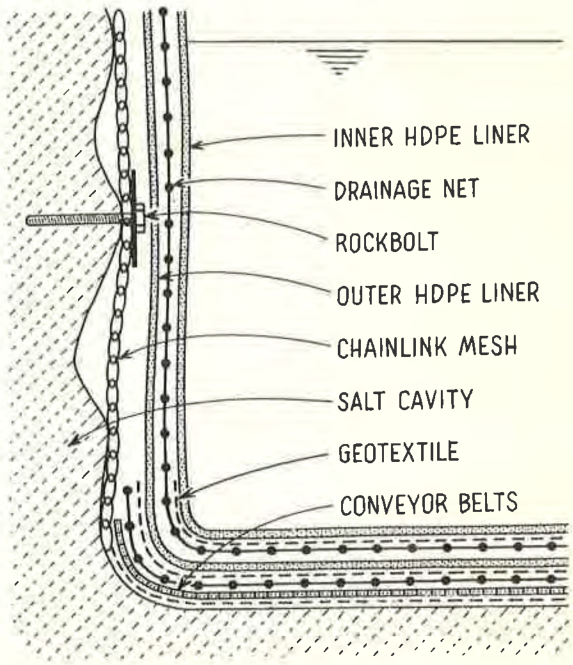

Fig. 2. Cross section showing layers forming the double lining system and typical chainlink mesh rockbolted to the wall (Not to scale).

conveyor belts were placed on the floor of the cavity under the outer liner to ensure a smooth support for the liner which is subjected to high water pressure. A summary of the materials used to form the lining system is shown schematically in Fig. 2.

Net strips used on the walls between the two liners as part of the internal drainage system were spot welded to the outer liner. Also, nets placed on the floor (external and internal drainage system) were spot welded together to prevent movement during liner installation. The drainage system was kept open at top for free escape of air entrapped between or under the liners.

2.4 Outlet Design

The "clean' and "dirty" outlets are part of the same concrete structure presented in Fig. 3. This structure is separated from the concrete structure which includes the two sumps by a 50 mm polystyrene foam-PVC waterstop joint. This joint allows for flexibility between the two structures. The outlet concrete structure is under the reservoir while the sump structure is under the bulkhead closing the access ramp used by excavation equipment. For the sump structure, the rock and the lowest bulkhead timber are solidly connected by the concrete. The outlet concrete structure is separated from the rock by 100 mm foam joints on 3 sides to allow the structure to withstand a significant reduction in the size of the salt cavity due to salt creep. The foam loses approximately 70% of its thickness under a compressive stress of 1 MPa. Stresses of that magnitude would be expected in the case of salt creep. Hence, the foam joints could absorb 14 cm (i.e., 2 x 10 x 70%) in the direction of the length of the outlet structure and 10 cm (15 x 70%) in the direction of the width, without damage to the structure. According to salt mine specialists, these displacements are smaller than salt displacements likely to occur over several years.

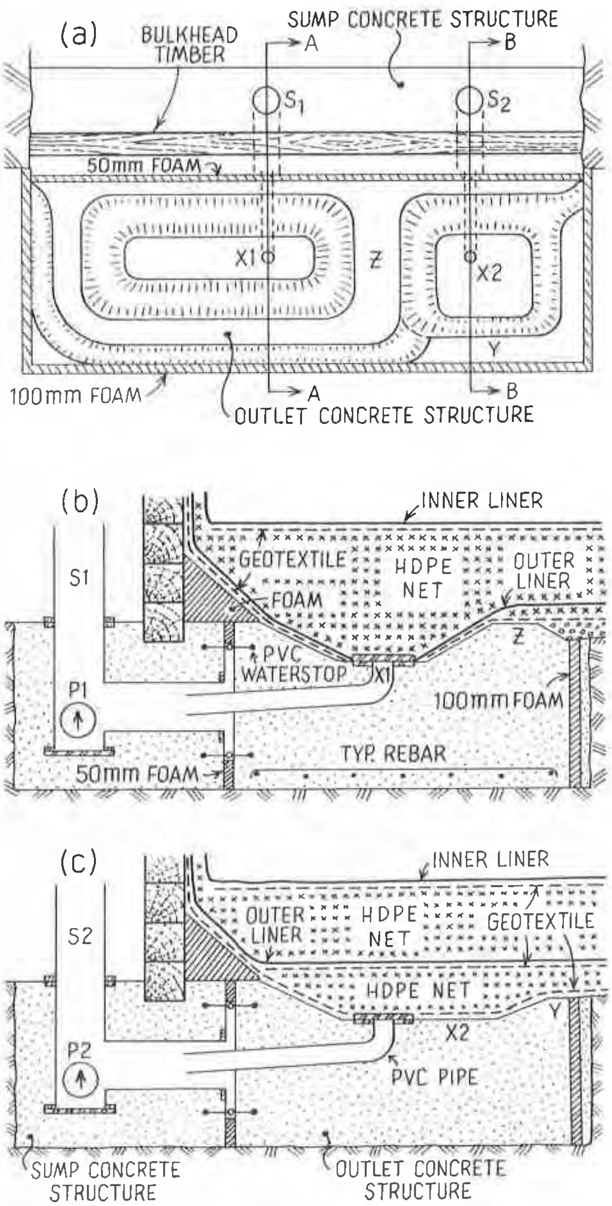

Fig. 3. **(a)** Plan view of the concrete structures embodying the clean and dirty outlets and sumps. S1 and S2 are the clean and dirty sump shafts which rise 20 m to the top of the reservoir. The 50 mm foam joint separates the two structures. The 100 mm foam joint on 3 sides of the outlet structure separate it from the rock. X1, X2, Y, and Z indicate various elevations of the outlet structure shown in section views A-A and B-B. **(b)** Section view A-A: internal drainage outlet, sump, and clean pump, P1. **(c)** Section view B-B: external drainage outlet, sump, and clean pump, P2. Indicated on the figures are the relative arrangements of the 2 HDPE geomembranes, HDPE drainage nets, geotextiles, bulkhead timbers, and foam-waterstop joints.

3. LINER MECHANICAL BEHAVIOR

3.1 Presentation of the Problem

Prior to first filling the reservoir, the double liner (including the intermediate plastic net spot-welded to the outer liner) is hung at the top and is subjected only to gravity forces. At this stage, the liner is in contact with the irregular walls of the cavity in a few points. Elsewhere, the liner is at a various distances from the wall.

The study described below was conducted to predict the behavior of the liner when subjected to water pressure and to recommend measures to alleviate stresses likely to be detrimental to the liner. Approximately 30 days are necessary to fill or empty the reservoir. The progressive application of water pressure on the liner during filling raised the following questions: (i) Would the water pressure be large enough to force the liner to follow the irregular shape of the walls? (ii) During the movement of the liner towards the walls, would the length of the liner increase (i.e., elongation of the liner) and/or would the top of the liner move downwards (provided it is not rigidly clamped)? and (iii) Would the liner withstand the stresses associated with the movements mentioned above? These questions are addressed in Section 3.2.

In all places where the liner would come in contact with the walls as a result of water pressure, another question arose: Would the liner burst through openings of the chain link mesh covering the walls? This question is addressed in Section 3.3.

Stresses likely to be caused by other mechanisms, such as gravity when the reservoir is empty, temperature difference between water and walls, and differential settlement between bottom and walls, were evaluated. These stresses appeared to be negligible as compared to the stresses generated by water pressures discussed in the next two sections.

In the analyses presented in the next two sections, the tensile behavior of the liner is the important characteristic to consider. The 2.5 mm (100 mils) thick geomembrane used for both liners had a tensile behavior typical for HDPE. The yield characteristics were approximately: 10% elongation, 20 MPa for the stress and, consequently, 50 kN/m for the force per unit width. The elongation at failure is of the order of 800%.

3.2 Liner Displacement Caused by Reservoir Filling

Survey of the Walls. To evaluate what would be the distance between the liner and the cavity walls, 52 vertical profiles of the walls were surveyed prior to liner installation. The 12 profiles surveyed on the East wall are shown in Fig. 4a. Also, the shape of the corners between walls and between wall-floor intersections were recorded. Typically, the radius of curvature at the corners was 0.6 to 0.9 m (2 to 3 ft). At the wall floor intersection, typical curvatures were 0.3 to 0.7m.

In the analysis, the likely location of the liner at the end of installation was determined by assuming that the liner was in contact with a number of protruding zones on the walls and was bridging the relative depressions between the protruding zones (Fig. 4b). The depressions were identified and recorded with their length, width, depth (relative to the assumed geomembrane location), and altitude. Also, the corners of the cavity were considered as depressions. This assumption proved to be correct, since during the installation it was not possible to place the liner in close contact with the wall at all points along the vertical corners.

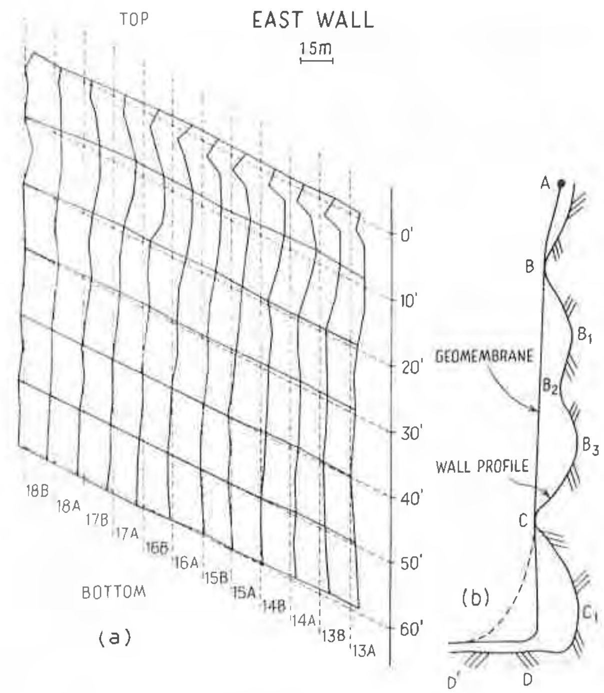

Fig. 4. **(a)** Surveyed shape of the East wall of the cavity after excavation. **(b)** Assumed shape of of the geomembrane after installation.

Maximum Vertical Displacement. The first step of the analysis consisted of evaluating the maximum vertical displacement of the top of the liner during the first filling. Assuming that the liner would be pushed against the cavity wall by the water pressure, the predicted vertical displacement was the difference between wall length ($BB_1B_2B_3CC_1D$, in Fig. 4b) and liner length (BCD, in Fig. 4b). Values between 0.3 and 0.5 m (12 to 20 in.) were obtained. As a result, it was decided to hang the liner from adjustable chains instead of the traditional rigid clamping considered initially.

The predicted values of the vertical displacement mentioned above were believed to be upper boundaries because part of the movement of the liner towards the wall results from liner elongation rather than vertical displacement. However, vertical displacements observed after the first filling were slightly larger than predicted (max. observed value ~0.6 m). Possibly, this could be due to the curved shape of the liner near the floor (CD' instead of CD in Fig. 4b). Also, depressions against which the liner exhibits elongation without vertical displacement are small depressions not shown in Fig. 4b. These small depressions were not considered in the evaluation of the wall profile length.

Pressure-Deflection Relationship. A preliminary study was conducted using the chart presented in (6). This chart had been established assuming that the liner would not move outside the depression area. This worst case situation gives the maximum liner elongation. The pressure-deflection analysis showed that the pressure exerted by 0 to 3 m (10 ft) of water would be sufficient to place the liner in contact with typical large depressions of the wall, but, in several cases, would cause yield of the liner. This analysis confirmed that it was important to attach the top of the liner in such a way that it could move as freely as possible.

To better model the behavior of the liner, an analysis was made, assuming friction between the liner and the wall (Fig. 5). The lengthy analysis cannot be reproduced here. However, it led to the chart presented in Fig. 5c. Using this chart for typical depressions of the wall showed that the water pressure necessary to push the liner in contact with the wall did not depend

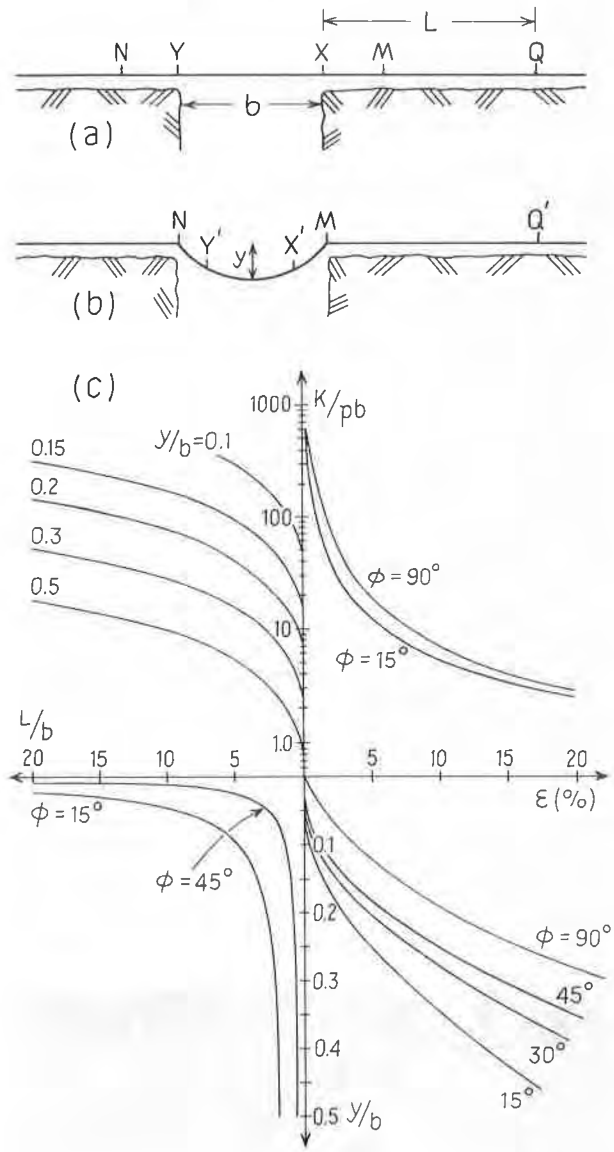

Fig. 5. Theoretical analysis of a geomembrane sliding with friction when pushed into a depression by water pressure: **(a)** Initial position of the geomembrane; **(b)** Final position of the geomembrane after sliding (beyond a certain length L, tensile stresses cannot overcome friction); **(c)** Chart. Knowing the geomembrane tensile stiffness (i.e., modulus x thickness), the water pressure p, and the width b of the depression, the chart gives the deflection y and the length L defined above (Note: for Φ=90°, L=0).

significantly on the friction angle between the liner and the wall. Therefore, this analysis confirmed that a pressure of 0-3 m (10 ft.) of water would be sufficient to place the liner in contact with large depressions (typically 2 m wide) of the wall. Other findings of the analysis were as follows:

• A length of typically 2 to 4 m (6 to 13 ft) of liner could move towards a typical large depression (2-3 m), depending on the friction angle. Comparing this length to the above mentioned height of water shows that, in most cases, the portion of the liner located above a large depression would move toward the depression before there is enough water pressure to mobilize friction and block downward movements.

• The situation appeared more critical for the portions of liner located against vertical corners between two walls because horizontal movements of the liner are not as free as vertical movements and because water pressure is higher at the level of a depression than above. Stresses in the liner appeared to be close to yield stress if the liner was placed at distances from the wall of the order of 0.6 m (2 ft) or more. It was therefore judged extremely important to place the liner as close as possible to the wall in the corners.

• The same analysis applied to smaller depressions (typically 0.5 m wide) showed that a fraction of the movement of the liner towards the depression would result from elongation of the liner. This occurs because the height of water necessary to push the liner into the depression is of order 10 m (33 ft) which causes friction to block vertical movements of the liner.

3.3 Liner/Support Interaction

Openings of the chain link mesh were 55 mm x 55 mm. A theoretical analysis was conducted to study the behavior of the liner. Preliminary calculations were made to determine in what stress condition the liner would be. It appeared that the deflection was too large to consider pure bending and too small to consider pure tension. A combined bending-tension analysis was conducted and led to the conclusion that deflection of the liner over a chain link mesh opening would be about 3 mm and stresses would be about 10 MPa (1450 psi), which is approximately one-half the yield stress. This margin of safety was judged insufficient because of the potential additional stresses likely to result from elongation of the liner to reach the chain link mesh and because of the decrease of yield stress with time due to creep. Therefore, it was decided to cover the lower 3 m (10 ft) of the walls with polyethlyene or other protective

Fig. 6. Photograph of the testing apparatus.

plates placed on the chain link mesh. Also, polyethylene plates were placed on all rockbolts to prevent direct contact between the liner and the rock bolts.

Laboratory tests were conducted at The University of Michigan to evaluate the effect due to water pressure pressing the HDPE geomembrane against the chainlink mesh and rockbolts on the cavity walls. The actual testing apparatus is shown in Fig. 6. These tests verified that the HDPE liner would not be damaged.

CONCLUSION

When the Proton Decay Experiment reservoir was designed, some of the analyses conducted were considered very sophisticated, even excessively sophisticated. In fact, the very careful monitoring (2,3) of this reservoir shows that the actual mechanical behavior of the liner is even more complex than considered at the design stage. With the better knowledge gained today it would be possible to include in the design: (i) superposition of stresses resulting from a small depression into a large depression; (ii) combined effects of creep and yield; and perhaps, (iii) failure mechanisms linked to molecular reorientation of the high density polyethylene. On the other hand, the conceptual and detailed design of the leakage collection system would probably be done today identically. The use of plastic nets which was then a premiere for leakage collection and detection is now becoming a widely accepted practice.

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Leakage Monitoring of the Geomembrane Liner for the Proton Decay Experiment

The Proton Decay Experiment reservoir contains 8500 m^3 of ultra-pure water and is located 600 m underground in a salt mine extending under Lake Erie. Site conditions required that the reservoir achieve zero leakage of water to the surrounding salt cavity. Leakage monitoring systems which include underwater inspections and underwater repairs of the HDPE geomembrane liner are described. Four problems, i.e., leaks, encountered with the liner are documented. Stress concentration at geomembrane thickness discontinuities (eg. seams and HDPE manufacturing ribs) are believed to be the primary cause of the observed failures. Although the reservoir continues successful operation, long term leakage monitoring and contingency procedures are necessary to avoid damage to the salt cavity and lining system.

INTRODUCTION

Located in a salt mine extending under Lake Erie, this reservoir (18m x 24m x 20m) contains more than 8500 m^3 of ultra-pure water. The more than 10^{33} protons in the water are being monitored by 2048 light sensitive photomultiplier tubes immersed in the water. The purpose is to perform a physics experiment to search for the first observation of the spontaneous disintegration, or "decay", of the proton. Spontaneous proton decay is predicted by a new category of physics theories known as Grand Unification Theories (1,2). These theories attempt to unify the fundamental forces of nature into a single set of equations. Accurate detection of proton decay can be disturbed by natural radioactivity and cosmic radiation. Therefore, the reservoir was installed in a cavity excavated from salt, where natural radioactivity is low, and is located 600 m underground, where the amount of cosmic radiation is 20,000 times less than on the earth's surface.

Ultra-Pure Water Requirement. Since the detection technique of the experiment looks for weak transmissions of ultraviolet light, transparency of the water is essential. Hence, all materials immersed in the water as well as the geomembrane liner should not contaminate the water by the release of chemicals, plasticizers etc. Tests were conducted at the University of California-Irvine to assess the physio-chemical compatibility of various materials with the ultra-pure water. The only acceptable geomembranes found were high density polyethylene (HDPE) and some hypalons (absent of "filler" materials). HDPE was chosen for the geomembrane liner because of its mechanical behavior, and hypalon was used to fabricate a floating cover for the reservoir.

Other materials, such as PVC pipes used between the purification facility and the reservoir; plexiglas, PVC, and glass used to make watertight housings for 2048 photomultiplier tubes used by the experiment; and PVC electrical cables used to power the tubes, had to be "oven-baked" at 60 °C for several hours in order to be compatible with the ultra-pure water.

Reverse Osmosis Purification Facility. The ultra-pure water is produced by a large scale reverse osmosis (RO) purification facility located underground and adjacent to the reservoir. In the RO technique, water is forced at high pressure through 3 stages of semipermeable membranes. The water is filtered to the 10^{-3} µm level with a very low concentration of biological molecules remaining. Following RO filtration, the water is irradiated with intense ultraviolet light to kill algae and other microorganisms.

To maintain the ultra-pure water status of the reservoir the following is done: (i) the reservoir is sealed off from the salt mine by use of vinyl airdome structures; (ii) "clean room" conditions and work procedures are maintained over the reservoir; (iii) the water is permanently recycled at 750 liters/minute through 1 µm carbon filters; and (iv) the water is permanently recycled through the RO membranes at 300 liters/minute.

Zero Leakage Requirement. The salt mine location of this reservoir is the primary reason for the geomembrane lining system to have zero leakage. A large portion of the cavity walls behind the lining system consists of massive crystalline salt. Due to the high solubility of salt, even small amounts of leakage water could result in serious damage or even collapse of the cavity. Hence, following a series of theoretical analyses and considerations, double geomembrane lining and double drainage systems were designed (3) and installed.

Although zero leakage from a lining system may be required, it is extremely difficult to achieve. Secure long term containment of liquids (water in the case of this reservoir or hazardous chemicals, wastes, etc. in other lined facilities) places strict requirements on all phases of the lining system --from the initial planning to long term monitoring.

Section 1 of this paper discusses the monitoring and quality control procedures imposed on the installation of the double geomembrane lining system. The leakage monitoring system and procedures presently in operation are described in Section 2. Overall performance of the lining system, problems encountered, and a discussion of the cause of the problems are given in Section 3. The successful underwater patching of the HDPE liner is described in Section 4.

1. LINING SYSTEM INSTALLATION

1.1 Lining System and Drainage Design

A careful design based on theoretical analyses, operational requirements and site conditions resulted in choosing a double HDPE geomembrane lining system and double drainage system for this reservoir. Details are given in (3). In summary, the design provides: (i) a geomembrane system with properties sufficient to withstand high water pressure (200 kPa), high stresses resulting from wall irregularities, and chainlink mesh rockbolted to the cavity walls; (ii) a leakage collection system between the two liners such that water leaking from the inner liner can be quickly pumped away preventing the build up of water pressure on the outer liner; (iii) a watertight outer liner to give an increased margin of safety in the event that the inner liner develops serious leaks; (iv) a leakage and ground water collection system outside the two liners; and (v) pumps to recycle leakage water away from the salt and back into the reservoir if large leaks develop in both liners.

1.2 Cavity Preparation

Prior to the installation of the outer geomembrane liner, the excavated salt cavity was surveyed and prepared as follows: (i) HDPE protective plates were placed over all rockbolts on the cavity walls; (ii) large HDPE panels were attached to the chainlink mesh along the bottom 3 m of the cavity walls where the water pressure would be highest; (iii) the cavity floor was covered with layers of reclaimed conveyor belts, geotextiles, and HDPE drainage nets, (see Fig. 2 in (3)).

1.3 Liner Fabrication Procedure

Before being brought underground into the reservoir area, all rolls of the HDPE geomembrane were laid out and inspected for defects. Once underground, the HDPE panels (5 m x 22 m) were seamed by machine and hand welders. Two panels, large enough to cover the entire area of the 4 cavity walls (22 m x 45 m), were fabricated on the floor of the reservoir. The large fabricated sections were hoisted up the 20 m high vertical walls of the cavity and were pushed as close as

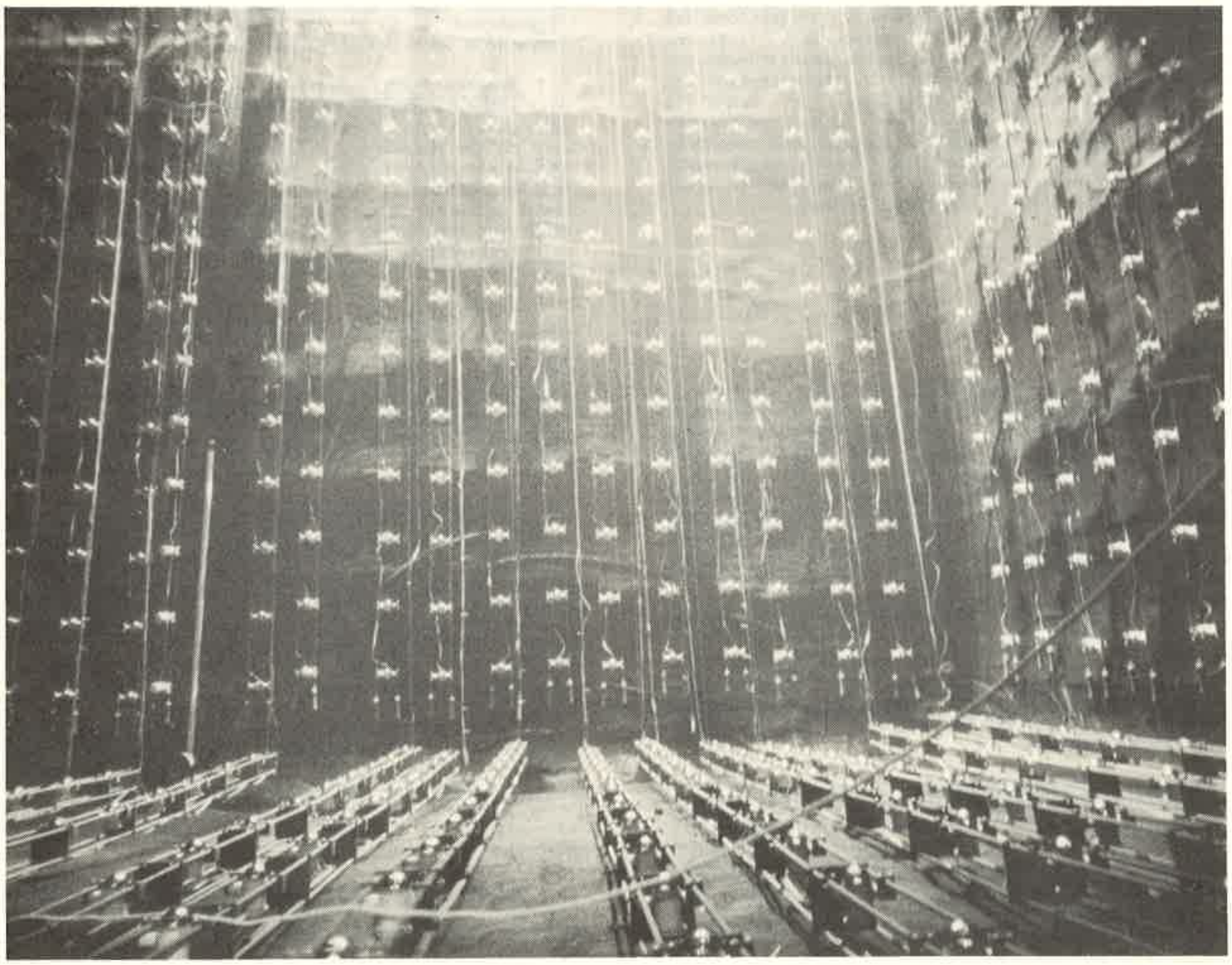

Fig. 1. Underwater photograph of the Proton Decay Experiment and reservoir. The photomultiplier tubes shown in the photograph are located along all six sides of the reservoir and are spaced 1 m apart. The extreme clarity of the ultra-pure water is apparent. The irregularly shaped walls of the cavity can be seen imprinted upon the HDPE geomembrane. Careful examination of the geomembrane on the far wall shown in the photograph reveals wrinkles and folds, which may be unavoidable in double lining systems.

possible to the walls especially in the corners. Closure seams were made with a hand-held extrusion welder which lays a bead of molten HDPE at the overlapping edge of the geomembranes. A large floor panel was fabricated similarly and joined to the wall panels by the hand-held extrusion welder. Following a series of quality control procedures and tests described below, layers of HDPE drainage netting were placed on the outer liner floor. The nets were spot welded together to prevent movement and covered with a thin geotextile which served as a working platform for fabrication of the inner liner. Also, HDPE net strips were spot welded to the outer liner walls as part of the drainage system. The inner liner was fabricated and assembled in the same manner. Serious problems encountered during the first attempt to fill the reservoir and the resulting remedial action are described in (4).

1.4 Quality Control Procedures

HDPE Seams. Liner "fabrication" seams were made by an automated machine which controls all parameters for extruding molten HDPE between two overlapping "heated" surfaces of the HDPE sheet and then pressing them together. If the temperature of the extrudite and HDPE surfaces at the time of seaming are sufficient for HDPE fusion, then the weld will be good. However, quality control of the seam is checked only by measuring the total thickness of HDPE at the seam by using an ultrasonic transmitter and receiver. With the ultrasonic procedure, deviations from the expected minimum thickness (greater than twice the thickness of a single HDPE sheet, i.e., >2 x 2.5 mm), indicate weld defects. In the assembly of this lining system there were >700 m of machine made extrusion welds. Crews tested 100% of the welds ultrasonically and found <0.5% of the total length of seams were defective. During the filling of the reservoir, 10 cm of "open" machine welds were found. Hence, about 3% of the actual defective seam length had gone undetected ultrasonically.

The extrusion welds made by the hand-held welder ("hand-welds") present more of a problem for quality control, since important welding parameters (4) are controlled by the operator rather than automatically. "Hand-welds" were used in several important areas in the assembly of this liner, viz., (i) closure of the large wall sections, (ii) cap strip seams, (iii) attachment of the floor panel to the wall panels, and (iv) connection to the drainage outlet. Furthermore, 90% of the "hand-welds" are located in the bottom 2 m of the reservoir.

Quality control of the "hand-welds" involves the use of a vacuum box device. Soap film is first spread on the weld, and then a vacuum box (with a gasket to fit tightly over the raised weld ribbon) creates soap bubbles in areas containing even small holes.

It is important to note that testing welds with the ultrasonic or vacuum box devices does not guarantee that the weld is good. A common failure mode for the HDPE extrusion welds (4) occurs when the HDPE extrudite or heated surfaces are too cool for HDPE fusion. In this case a "cold weld" joint will result which will have the appearance of a good weld and will pass seam quality control, however, under impact or stress, it fails by the clean separation of the weld ribbon from the geomembrane surface. An observation of a failure of this type is presented in Section 3.2.

Trial Fillings. The outer liner was filled with 2 m of water and monitored for 2 days before installation of the internal drainage system and inner liner. In this test, leakage water was detected in the external drainage system. SCUBA divers located the leaks by techniques described in Section 4. The water was pumped out and repairs were made. After fabrication and installation of the inner liner, a similar test was conducted.

2. LEAKAGE MONITORING

The absolute requirement that leakage water from this reservoir does not come into contact with the surrounding salt cavity, demands that the leakage be continously monitored. The present monitoring system has been in operation for over 20 months. With this system, no damage having detrimental effects on the lining system has accompanied the problems described below in Section 3.2.

2.1 Automatic Monitoring System

Two drainage systems were installed to handle leakage water from the reservoir: (i) an internal drainage system between the two geomembranes which collects water leaking from the inner liner; and (ii) an external drainage system which collects water leaking from the outer liner. Both systems have separate outlets, sumps, and evacuation pumps (3).

The amount of leakage water processed by each drainage system is measured by flowmeter-odometers and is recorded on a stripchart and by a computer. With each data reading the computer determines if a new leak has appeared in the lining system and assesses the ability of the sump pumps to handle the leakage water. In addition to leak detection, the computer monitors several other safety devices associated with the underground laboratory and water purification facility, e.g., fire and gas alarms, pipe pressures, status of key pumps, reservoir water level, electrical power status, etc. Via an autodial modem, the computer transmits messages detailing the nature of any detected problem to a video terminal at ground surface in the mine security office (manned around the clock). Security personnel notify the appropriate emergency or maintenance crews.

2.2 Underwater Monitoring

If the security office receives a message that the reservoir has developed a leak, a SCUBA diving team is called in to locate and patch the leak underwater using techniques described in Section 4.

Technical SCUBA Diving Team. The diving team consists of 4 certified divers. They function similar to firemen, i.e., most of the time they maintain their gear and perform routine maintenance, however, if the leak condition warrants, they can be in the reservoir within 6-10 hours. Emergency procedures have been established and practiced. Typically, two divers are in the reservoir concurrently. Team members rotate between diving and tendering since the maximum allowable time working at a depth of 20 m without decompression is about 50 minutes. All required gear for diving in the reservoir is kept permanently at the reservoir site. The gear consists of: (i) full body dry suits made of latex rubber; (ii) full head commercial masks; (iii) self contained air tanks; and, (iv) a communication tether. The diving gear was carefully selected to minimize contamination of the ultra-pure water by the presence of the divers.

Underwater Maintenance. Aside from responding to leaks in the liner, the diving team performs a standard maintenance program about every 3 weeks. This program entails: (i) inspection and adjustment of the underwater apparatus used for the Proton Decay Experiment itself; (ii) vacuuming particulate debris which has settled to the reservoir floor; (iii) inspection and repair of patches covering holes previously located in the liner; and, (iv) monitoring wrinkles, voids, and areas of the liner showing signs of elongation.

3. PERFORMANCE OF THE LINING SYSTEM

As of February 1984, the reservoir has been operated continuously for 20 months. Although leaks in the inner liner have periodically occurred (Section 3.2), they have been located quickly (with the monitoring systems described in Section 2) and have been successfully patched underwater (Section 4). Hence, reservoir operation has not been interrupted and no dertimental damage has resulted from those leaks. The internal drainage system has evacuated the leakage water promptly, thereby preventing the buildup of water pressure on the outer geomembrane liner. The reservoir's performance with respect to its ability to maintain ultra-pure water is presented in Section 3.1. Problems (i.e., leaks) encountered with the liner are presented in Section 3.2 and an evaluation of the causes of the problems is given in Section 3.3.

3.1 Experience with Maintaining Water Purity

A number of measurements are made routinely to assess the purity of water in the reservoir. In one measurement, ultraviolet light produced by a laser is shined into the water and the distance that the light travels before being attenuated to 50% of its original intensity is measured by the photomultiplier tubes. Attenuation lengths of 40-50 m are maintained. These values can be compared with a typical attenuation length for ordinary tap water of 4 m. This extreme transparency of water is apparent in the photograph shown in Fig. 1.

Periodic measurements of the amount of dissolved solids in the water gives values in the range 2-10 ppm. Also, particulates suspended in the water have been observed by the divers. Although the particulates do not significantly degrade the light attenuation length, they are "food" for the growth of microorganisms. A species of bacteria (Pseudomonas sp.) has been found in the water, mainly attached to the PVC and glass surfaces of the photomultiplier tube housings. To inhibit bacterial growth, the reservoir water is occasionally shock treated with chlorine. Also, the divers vacuum particulates which have settled to the bottom of the reservoir as part of their routine maintenance.

3.2 Liner Problems Encountered

Problem 1. The first leak in the reservoir was detected July 26, 1982 near the completion of the initial filling with water. An underwater photograph of the failure is shown in Fig. 2. The white moustache-shaped area near the center of the photo is an elongated region where the HDPE liner has exceeded the yield point. The dark vertical line (above the arrow in the photo) is a hole in the geomembrane. The water depth above this hole is 13.5 m; the hole and yielded areas were ~20 mm^2 and ~450 mm^2 respectively; and the measured rate of leakage was 28 liters/minute. The elongated area was monitored for several weeks while patching techniques were being developed. The following was observed: (i) after 3 weeks, 4 vertical holes similar to the one shown had developed; (ii) after 6 weeks, 11 similar holes had developed; and (iii) after 3 months, about 70% of the original elongated area had become a few large holes through the geomembrane. In all cases the holes were vertical in shape indicating that the HDPE failure was perpendicular to the direction of elongation. Hence, molecular reorientation of yielded HDPE may be a factor. Inspection of the geomembrane in the vicinity of the elongated area revealed widespread grinder marks (remnant of the HDPE welding process) as far as 20 cm away from the nearest seam. Hence, the geomembrane was somewhat "thinner" due to the grinding, but was also adjacent to a "thick" field seam. This condition establishes a region of concentrated stress (see Section 3.2 and (5)) which may be a cause for the failure.

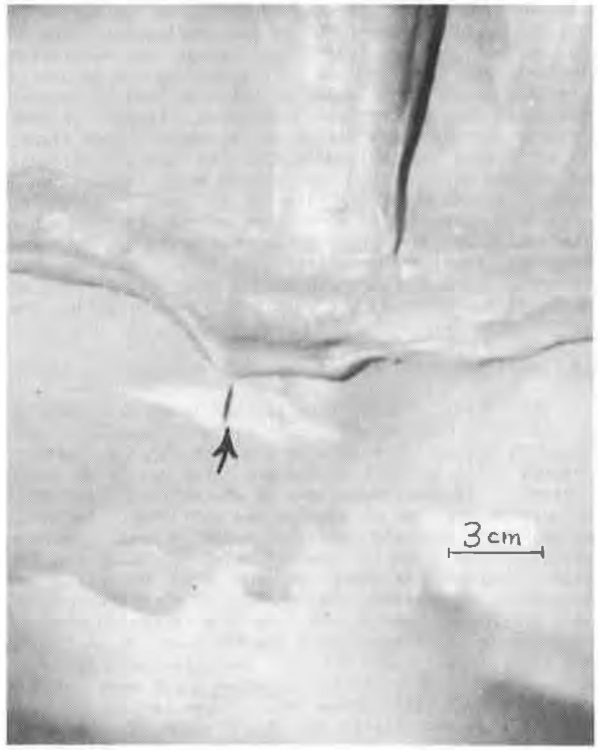

Fig. 2. The white "moustache" shaped area near the center of the underwater photograph is yielded HDPE. The arrow points at a vertical slit hole in the HDPE geomembrane, (Problem 1).

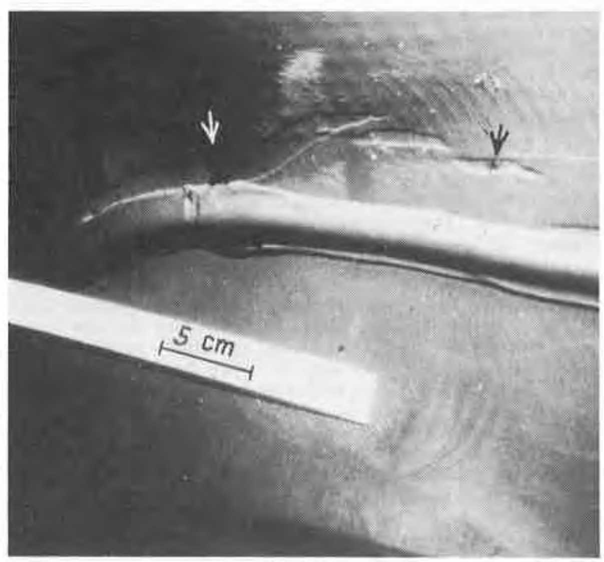

Fig. 3. Elongation of the HDPE liner with arrows indicating holes in the geomembrane. The photo shows only a portion of the yielded area which extended 1.5 m along the edge of the HDPE weld ribbon, (Problem 2).

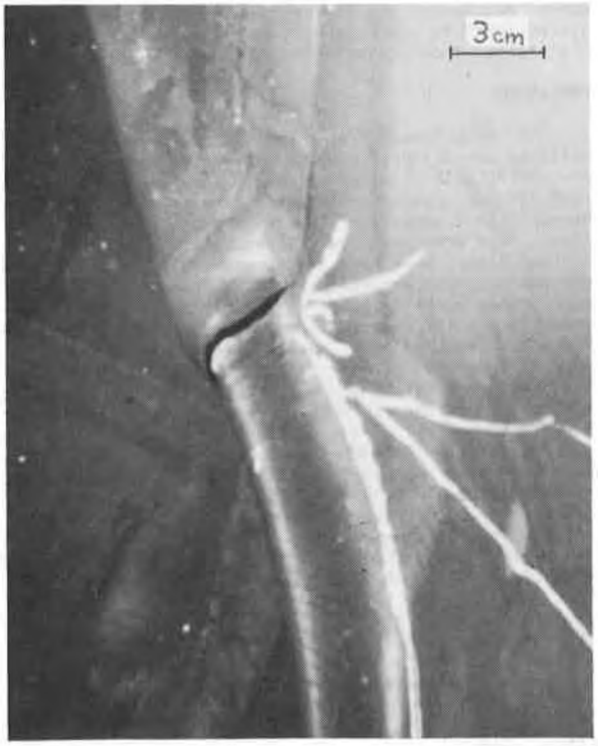

Fig. 4. This photograph shows a HDPE "hand-weld" failure in a region where the weld crosses over a wrinkle on the inner liner. The clean separation of the weld ribbon from the geomembrane is indicative of a "cold" weld. The wrinkle is typical of many on the inner liner,(Problem 3).

Problem 2. On September 30, 1982 the internal drainage system detected a new leak. The diving team located an extensive area of HDPE elongation beyond the yield point. A total of 8 holes were found in the elongated area which extended for 1.5 m adjacent to a "hand" extrusion seam joining the liner floor panel to the west wall panel. A portion of the elongated liner with 2 holes visible is shown in Fig. 3. Data associated with this problem are: (i) 19.5 m water depth above; (ii) total area of holes ≅ 175 mm^2; (iii) yielded area ≅ 11250 mm^2; and (iv) measured leakage rate >40 liters/minute. This failure caused great concern because of its relatively large size. Earlier observations of the Problem 1 failure indicated that a yielded region of this type could eventually develop into large holes which could produce more leakage water than could be handled by the pump system. A large patch placed over the entire elongated area by the SCUBA divers stopped the leakage. This initial patch has not been removed, hence, no new data is available on the development of additional holes. However, the divers were able to pinpoint a void region (by tapping and listening) behind the failure region. This void was about 20 cm wide and 1 m long. Voids of this type may be inherent in double lining systems and are due to a separation (or mismatched contouring) of the two nested liners.

Problem 3. On October 26, 1982 another leak was detected. A search by the diving team located the condition shown in Fig. 4. A failure of an HDPE extrusion seam had occurred in a region where the "hand" weld crossed over a wrinkle on the inner liner. Further inspection revealed that the weld is a cap strip weld indicating that 2 failures had occurred: (i) the cap strip weld; and (ii) a defect covered by the cap strip. The water depth above the hole is 17 m and the measured leakage rate was 8 liters/minute. The most significant factor about this failure was the "clean" separation of the weld from the HDPE geomembrane which is indicative of a "cold" HDPE extrusion weld (4). Current quality control procedures may be inadequate to avoid this type of seam failure.

Problem 4. More than one year passed before the next failure occurred on January 25, 1984. Again, there was HDPE yield accompanied by a hole in the geomembrane. Characteristics of this failure are quite different from previous failures in that it is not associated with grinding, welding, or any known defect. However, the elongation is located adjacent to a manufacturing rib (spread coating overlap) in the HDPE. A void area between the two liners (15 cm wide and 1 m long) runs parallel to the rib. The water depth above the failure is 18 m; the yielded area is 325 mm^2 and the hole area is 10 mm^2. The failure mechanism (i.e., concentrated stress) may be the same as for Problems 1 and 2, however, in this case the geomembrane thickness discontinuity is smaller and not associated with seaming. Since 20 months passed with the liner under the same stress condition before this failure, HDPE creep may also be a factor.

3.3 Evaluation of the Problems

Seam Defects. Problems 1,2,3 described above may be due (at least in part) to the HDPE extrusion welding process coupled with stresses in the geomembrane . In Problem 1, there is excessive grinding of the geomembrane in the region of the failure. The purpose of grinding the geomembrane surface prior to welding is to remove oil, dirt, and other substances which may inhibit a properly "fused" HDPE weld. However, grinding leaves the geomembrane thinner. A thin region adjacent to the relatively thick weld ribbon is problematic , particularly if the liner is under stress. (See the discussion in (5) for the detrimental effects of concentrated stresses). Also in Problem 2, the yielded area occurred in the grinded area adjacent to the weld ribbon. In this case, the grinding did not seem to be excessive, however, the pattern of elongation partially follows the circular grinder striations (Fig. 3).

Very likely the weld failure of Problem 3 was initiated by the wrinkle in the geomembrane. However, the clean separation of the weld from the liner is indicative that it was a "cold" HDPE extrusion weld.

Wrinkles and Voids. Numerous wrinkles on the inner liner and void spaces between the 2 liners have been observed in the double geomembrane lining system of this reservoir. Wrinkles and voids may be common for double lined facilities due to the difficulty of fabricating two liners with equal dimensions. In addition, differential movements due to thermal gradients, subsidence, and other phenomena can cause wrinkles to appear, disappear, or move. In this reservoir, divers have documented wrinkles which have moved by several meters. These movements occurred during filling and shortly after as the reservoir water temperature came into equilibrium with the cavity walls.

Problems 2 and 4 were accompanied by void regions (15-20 cm wide) behind the areas of elongation. At the design stage of the lining system, a theoretical analysis of the tension membrane behavior of a 2.5 mm thick HDPE liner was conducted. The studies indicated that gaps of these dimensions could be bridged by the geomembrane under a water pressure of 200 kPa (20 m deep). Nevertheless, the failures occurred. In the case of Problem 2, grinding may have been a factor. However, Problem 4 was not associated with any weld or known defect in the geomembrane.

Concentrated Stresses. Discontinuities in geomembrane thickness can cause stress concentration at the discontinuity (5). As a result of these stresses, a geomembrane can elongate and fail under forces smaller than otherwise might be expected for uniform thickness. Hence, the primary reason for the failures discussed above may be concentrated stresses. For example, in Problems 1 and 2, elongation and failure occurred adjacent to a thick weld ribbon. In these cases, the discontinuity in HDPE thickness was enhanced by the grinding associated with surface preparation prior to welding. In Problem 4, elongation and failure occurred adjacent to a rib remnant of the manufacturing process. Also, this failure occurred 20 months after the reservoir was filled, which suggests that it may be due to HDPE creep accelerated by stress concentration.

4. UNDERWATER REPAIR OF A GEOMEMBRANE

4.1 Locating a Leak Underwater

There are two basic techniques for locating holes underwater in the HDPE liner, viz., "listening" and "looking". For this reservoir, it has been found that holes which leak more than 10 liters/minute can be heard clearly by the divers if extraneous noise is eliminated (i.e., if pumps are shut off, etc.). The sound level of the leak usually allows the divers to locate the general vicinity of the hole. Bright lights are used to search the area localized by sound. Yielded HDPE has a white appearance, which under bright lights, stands out in contrast to the black geomembrane.

It should be noted that the "listening" technique is relatively easy in this reservoir because there is little extraneous noise in the vicinity due to the underground location. A more sophisticated technique, using a parabolic reflector and hydrophone, are being tested.

4.2 Underwater Patching

It is not possible to fusion weld HDPE underwater. Also, we know of no chemical resins such as marine epoxies or silicon cements which will bond permanently with polyethylene --in or out of water. Nevertheless, the need existed to plug the various holes appearing in the HDPE geomembrane without the expense and lost time of emptying the reservoir. Therefore, over a period of several months, various underwater patching schemes were tested and evaluated. The result is that for the first time to our knowledge, successful underwater repair of a geomembrane has been accomplished.

Although the patches used in this reservoir range from a few square centimeters up to 1.0 m^2, we believe the technique could also work over larger areas. The patches are not permanent repairs! The technical diving team must inspect and repack the patches periodically, although patch lifetimes greater than 6 months have been achieved. Given the relative ease of dealing with these patches, it has been decided to continue this mode of operation for several years rather than empty the reservoir to make permanent repairs.

There are various layers of materials used for the underwater patches. The basic scheme is to plug holes in the geomembrane with a sticky or gum-like substance but prevent or inhibit the substance from being extruded through the hole by the water pressure. We have found that butyl rubber works well for the gum-like material and that at least two layers of fine mesh (0.5 mm polypropylene strands with 5 strands/cm in both directions forming a diamond pattern) between layers of butyl rubber are needed to inhibit extrusion. The entire layered patch is covered by kapton tape to prevent the butyl rubber from contaminating the reservoir water.

CONCLUSION

The underground Proton Decay Experiment reservoir continues operation with negilible water leaking to the surrounding salt cavity. Leakage monitoring systems which include underwater inspections have contributed to the successful underwater repair of the HDPE geomembrane liner. Four problems, i.e., leaks, which have developed in the liner have been documented. In two cases, failures occurred adjacent to HDPE seams where the thick weld ribbon constitutes a large discontinuity in the geomembrane thickness. Stress concentration at these HDPE steps coupled with narrow void spaces behind the liner may be the primary reason that the geomembrane elongated and failed at a lower average stress than expected. Another failure appeared adjacent to a HDPE manufacturing rib, also a discontinunity in thickness. However, nearly 20 months passed with the reservoir under the same stress condition before this failure which suggests that HDPE creep was a factor, possibly accelerated by stress concentration. Also, a HDPE seam failed where it crossed over a wrinkle in the liner. This failure suggested: (i) wrinkles or folds can initiate failures at weak points in the geomembrane by local stress; and (ii) current quality control procedures of HDPE seams may be inadequate to avoid welds with weak adhesion, i.e., "cold welds". The reservoir continues successful operation, however, the necessity of long term leakage monitoring and contingency procedures has been demonstrated.

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Analysis of Stresses and Elongations in Geomembranes

Two unexpected effects of stresses upon geomembranes are analyzed by the author. The extent to which the tensile behavior of an unreinforced geomembrane is affected by a non-uniform thickness is discussed in the first portion of the paper. Behavior of both geomembranes with yield and without yield is examined for several cases of non-uniform thickness, with portions of the analyzed geomembrane having lack of thickness and/or extra thickness. The author then explores transfer of stress through seams for both reinforced and unreinforced geomembranes. The importance of peel tests in evaluation of the stress capability of seams is emphasized. Conclusions drawn from these analyses and recommendations for designers of liners are then presented.

INTRODUCTION

Stresses in geomembranes result from a number of phenomena at different scales: stresses affecting large areas of the geomembrane may result from gravity, thermal expansion-contraction, and shrinkage; stresses affecting localized areas of the geomembrane usually result from differential behavior between two areas of the soil or structure supporting the geomembrane, such as differential settlement between earth dike and concrete structure, cracks in the supporting concrete or soil, and subsidence of a small area of the supporting soil.

Experienced designers are aware of the above mentioned stresses, but there is much less awareness, even among specialists, of concentrated stresses likely to occur at a much smaller scale due to discontinuities in geomembrane thickness. Such stresses might be the cause of unexplained failures. These stresses and the resulting elongations are discussed in the first part of this paper.

Also, stress transfer through a seam between two adjacent geomembrane sheets may not be as simple as usually assumed, and seam failure may be caused by forces smaller than expected. The influence of seam width on stress transfer through seams is discussed in the second part of this paper.

The two parts of this paper have in common the influence that geometrical characteristics such as geomembrane thickness and seam width have on stresses.

1 INFLUENCE OF A NON-UNIFORM THICKNESS ON GEOMEMBRANE TENSILE BEHAVIOR

1.1 Presentation of the Problem

It is usually implicitly assumed that geomembrane thickness is uniform. In fact, the thickness of a geomembrane may not be uniform for a variety of reasons such as: (i) manufacturing process (e.g., spread coating overlaps); (ii) insufficient control of manufacturing process; (iii) scratches on, or abrasion of the geomembrane, done during transportation or installation; (iv) grinding of the geomembrane for seaming preparation; (v) seaming; (vi) cap strips and patches; and (vii) abrasion of the geomembrane by materials in contact. A geomembrane can exhibit localized extra thickness as a result of (i), (v), and (vi), localized lack of thickness as a result of (iii), (iv), and (vii), and either one from (ii).

The purpose of the study presented hereafter is to evaluate the extent to which the tensile behavior of a geomembrane can be affected by a non-uniform thickness. The analyses are related exclusively to unreinforced geomembranes. The tensile behavior of most reinforced geomembranes is essentially governed by the reinforcing fabric and, for practical purposes, is not affected by a non-uniform thickness of the polymeric coating.

1.2 Tensile Behavior of a Uniformly Thick Unreinforced Geomembrane

A tensile test consists of recording the tensile force F necessary to progressively increase the length L of a geomembrane sample. Normalized results are obtained by dividing the force F by the sample width B to obtain the force per unit width α, and dividing the sample length increase ΔL by the initial length L to obtain the strain ε (also called elongation). As shown in Fig. 1, the force per unit width/elongation curves can be of two types for unreinforced geomembranes: (i) curves without yield are typical of PVC or butyl rubber geomembranes; and (ii) curves with a yield peak are typical of HDPE geomembranes.

For unreinforced geomembrane samples of uniform thickness T, it is possible to further normalize test results by dividing the force per unit width by the initial thickness to obtain the stress σ ($\sigma = \alpha/T$). A stress/elongation curve related to a given polymeric compound is valid for any geomembrane made of this polymeric compound, regardless of its thickness, while a force per unit width/elongation curve is valid only for one type of geomembrane (which includes thickness and polymeric compound). Using the stress/elongation curve, the secant modulus E at elongation ε of a polymeric compound can be defined as the ratio between the stress and the corresponding elongation [$E = \sigma/\varepsilon = \alpha/(\varepsilon T)$].

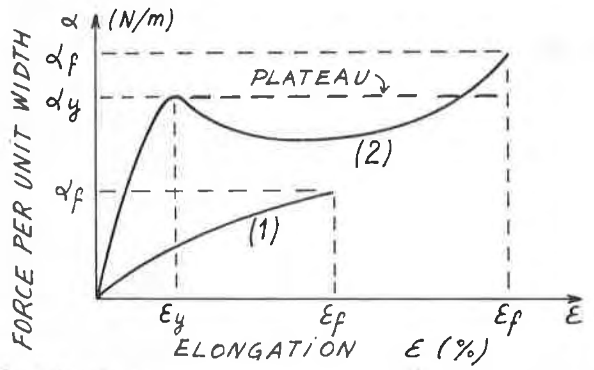

Fig. 1 Force per unit width/elongation curve without yield (1) or with yield (2). Notation: α_y = force per unit width at yield; α_f = force per unit width at failure; ε_y = elongation at yield; ε_f = elongation at failure. Note: depending on the polymeric compound and the thickness of the geomembrane, curve (1) can be above or below curve (2).

1.3 Elongation of a Non-uniformly Thick Geomembrane

The geomembrane samples considered in the theoretical study are defined in Fig. 2. When any of these samples is subjected to a tensile force, the force per unit width α is the same in any cross section parallel to the width B, but the stress σ in the portion of thickness T and the stress σ' in the portion of thickness T′ are different as shown by the following equation which results from the definition of σ given above:

$$\alpha = \sigma T = \sigma' T' \qquad (1)$$

The stress/elongation curve is unique for a given polymeric compound. Therefore, as a result of the difference between σ and σ', the elongation ε in the portion of thickness T and the elongation ε' in the portion of thickness T′ are different. However, an average elongation ε_a can be defined as the ratio between the sample length increase Δ L and the original length L. This results in:

$$L(1 + \varepsilon_a) = (L - L')(1 + \varepsilon) = L'(1 + \varepsilon') \qquad (2)$$

Hence:

$$\varepsilon_a = \varepsilon(1 - L'/L) + \varepsilon' L'/L \qquad (3)$$

Combining Eq. 1 with the definition of the modulus given in Section 1.2 gives:

$$\alpha = E \varepsilon T = E' \varepsilon' T' \qquad (4)$$

Eliminating ε or ε' between Eqs. 3 and 4 gives the following two equations:

$$\varepsilon_a / \varepsilon' = L'/L + (1 - L'/L)(T'/T)(E'/E) \qquad (5)$$

$$\varepsilon_a / \varepsilon = (1 - L'/L) + (L'/L)(T/T')(E/E') \qquad (6)$$

The elongation of the thin portion of the geomembrane sample is larger than the elongation of the thick portion. Therefore, the thin portion will reach the yield elongation ε_y (if any) and the elongation at failure ε_f before the thick portion. Therefore, the behavior of the sample is governed by the thin portion. Consequently, Eq. 5 will be used when T′ is less than T, and Eq. 6 when T is less than T′.

The principle of the discussions presented in the next two sections is to compare the actual behavior of the geomembrane sample, i.e., the measured force per unit width and the measured elongation in a tensile test, with the nominal behavior of the geomembrane, i.e., the behavior of a sample of uniform thickness T (characterized by its force per unit width/elongation curve, (1) or (2) in Fig. 1).

1.4 Behavior of a Geomembrane Without Yield

Lack of Thickness In this case, ε' is larger than ε and failure occurs when $\varepsilon' = \varepsilon_f$. According to Eq. 5, the average elongation at failure is:

$$\varepsilon_{af} = \varepsilon_f [L'/L + (1 - L'/L)(T'/T)(E'/E)] \qquad (7)$$

The actual force per unit width at failure α_{af} is the force per unit width causing failure of (i.e., generating stress σ_f in) the portion of thickness T′. It is related to the nominal force per unit width at failure α_f by:

$$\alpha_{af} = \sigma_f T' = \alpha_f T'/T \qquad (8)$$

Extra Thickness In this case, ε is larger than ε' and failure occurs when $\varepsilon = \varepsilon_f$. According to Eq. 6, the average elongation at failure is:

$$\varepsilon_{af} = \varepsilon_f [(1 - L'/L) + (L'/L)(T/T')(E/E')] \qquad (9)$$

The actual force per unit width at failure α_{af} is the force per unit width causing failure of the portion of thickness T. It is therefore equal to the nominal force per unit width at failure:

$$\alpha_{af} = \alpha_f \qquad (10)$$

1.5 Behavior of a Geomembrane With Yield

In the analysis presented in this section, the nominal force per unit width/elongation curve of the geomembrane (curve (2) in Fig.1) is approximated by a curve with a plateau after the peak. This simplifies the analysis without practically affecting the results.

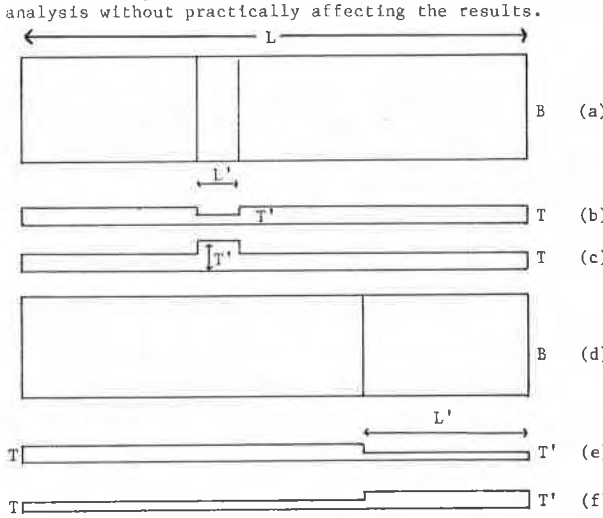

Fig. 2 Geomembrane samples considered in study: (a) Plan view for samples (b) and (c); (b) Sample with an indentation over length L′; (c) Sample with extra thickness over length L′; (d) Plan view for samples (e) and (f); (e) Sample with lack of thickness over length L′; and (f) Sample with extra thickness over length L′.

Lack of Thickness In this case, ε' is larger than and yield occurs when $\varepsilon' = \varepsilon_y$. According to Eq. 5, the average elongation at yield is:

$$\varepsilon_{ay} = \varepsilon_y \,[L'/L + (1-L'/L)(T'/T)(E'/E)] \qquad (11)$$

The actual force per unit width at yield α_{ay} is the force per unit width causing yield of (i.e., generating stress σ_y in) the portion of thickness T'. It is related to the nominal force per unit width at yield α_y, by:

$$\alpha_{ay} = \sigma_y \, T' = \alpha_y \, T'/T \qquad (12)$$

This force per unit width remains constant beyond the yield point. Therefore, the elongation ε in the portion of thickness T remains constant, while the elongation ε' keeps increasing until it reaches the elongation at failure ε_f. The constant value of ε corresponds to the force per unit width α_{ay} exerted on the portion of thickness T:

$$\varepsilon = (\alpha_{ay}/T)/E = \varepsilon_y \; (T'/T)(E'/E) \qquad (13)$$

The average elongation beyond the yield point can be determined by combining Eqs. 3 and 13:

$$\varepsilon_a = \varepsilon_y \; (1-L'/L)(T'/T)(E'/E) + \varepsilon' L'/L \qquad (14)$$

Hence, at failure:

$$\varepsilon_{af} = \varepsilon_y \; (1-L'/L)(T'/T)(E'/E) = \varepsilon_f \, L'/L \qquad (15)$$

Extra Thickness In this case, ε is larger than ε' and yield occurs when $\varepsilon = \varepsilon_y$. According to Eq. 6, the average elongation at yield is:

$$\varepsilon_{ay} = \varepsilon_y \,[(1-L'/L) + (L'/L)(T/T')(E/E')] \qquad (16)$$

The actual force per unit width at yield α_{ay} is the force per unit width causing yield of the portion of thickness T. It is therefore equal to the nominal force per unit width at yield:

$$\alpha_{ay} = \alpha_y \qquad (17)$$

This force per unit width remains constant beyond the yield point. Therefore, the elongation ε' in the portion of thickness T' remains constant, while the elongation ε keeps increasing until it reaches the elongation at failure ε_f. The constant value of ε' corresponds to the force per unit width α_y exerted on the portion of thickness T':

$$\varepsilon' = (\alpha_y/T')/E' = \varepsilon_y \; (T/T')(E/E') \qquad (18)$$

The average elongation beyond the yield point can then be determined by combining Eqs. 3 and 18:

$$\varepsilon_a = \varepsilon_y \; (L'/L)(T/T')(E/E') + \varepsilon \, (1-L'/L) \qquad (19)$$

Hence, at failure:

$$\varepsilon_{af} = \varepsilon_y \; (L'/L)(T/T')(E/E') + \varepsilon_f \, (1-L'/L) \qquad (20)$$

1.6 Numerical Examples

To illustrate the use of Eqs. 7 through 20, typical cases of lack of thickness or extra thickness have been considered. Values of the actual force per unit width at failure α_{af} and of the average elongation at failure ε_{af} have been calculated for two types of geomembranes: (i) a typical PVC geomembrane (without yield) with characteristics at failure α_f = 20 kN/m and ε_f = 400%; and (ii) a typical HDPE geomembrane, exhibiting yield, whose characteristics are α_y = 30kN/m and ε_y = 10% at yield and α_f = 30 kN/m and ε_f = 700% at failure. In this latter case, the actual force per unit width and the average elongation at yield were also calculated. Results are presented in Table 1.

The behavior of a geomembrane without yield is not significantly affected by typical thickness variations. At working elongations (typically 0 to 50%), the change in behavior is negligible.

The behavior of a geomembrane exhibiting yield is significantly affected, especially in two cases, one of lack of thickness, one of extra thickness:

- A narrow scratch or crack reducing the thickness of the geomembrane by 25% does not affect the strength significantly, but does affect the elongation at yield which becomes 7.5% instead of 10%. Even more affected is the elongation at failure which becomes 14% instead of 700%. This result does not correspond

Table 1 Influence of lack of thickness [cases (a) and (b)] and extra thickness [cases (c), (d) and (e)] on the behavior of typical geomembranes without or with yield. Definitions of L, L', T and T' are given in Fig. 2. Calculated values are: actual force per unit width at failure α_{af} and average elongation at failure ε_{af}, for a geomembrane without yield (Section 1.4); and actual force per unit width at yield α_{ay} and at failure α_{af}, and average elongation at yield ε_{ay} and at failure ε_{af}, for a geomembrane with yield (Section 1.5).

		Given Geometry		WITHOUT YIELD Calculated with α_f= 20 kN/m ε_f= 400%		WITH YIELD Calculated with α_y=α_f= 30 kN/m ε_y= 10% ε_f= 700%		
		L'/L	T'/T	α_{af}	ε_{af}	α_{ay} = α_{af}	ε_{ay}	ε_{af}
		(-)	(-)	(kN/m)	(%)	(kN/m)	(%)	(%)
(a)	Scratch, Crack	0.01	0.75	15	301	22.5	7.5	14
(b)	Lack of thickness over a large area	0.5	0.9	18	380	27	9.5	354
(c)	Seam	0.2	2	20	360	30	9	561
(d)	Patch, Cap strip	0.5	2	20	300	30	7.5	352
(e)	Extra thickness over a large area	0.5	1.1	20	382	30	9.5	354

to an extreme academic case but to a likely situation. In fact, failures of this type have been observed (1). Consequently, the following important recommendation can be made: the elongation to be considered when designing a liner using a geomembrane exhibiting yield should be smaller than an allowable elongation obtained by dividing the elongation at yield by a certain factor. Further studies should be undertaken to determine this factor. Tentatively, using a value of 2 for this factor can be considered to include: (i) the above mentioned reduction of elongation at yield; (ii) the effect of creep which usually affects the behavior of geomembranes exhibiting yield; (iii) the possible smaller elongation in the field biaxial situation as compared to a laboratory uniaxial test; and (iv) a factor of safety.

- An extra thickness over a large area (e.g., a patch or a cap strip) also tends to concentrate stresses in the portion of the geomembrane having the nominal thickness. As a result the average elongation at yield is significantly affected. This is important because this elongation governs the ability of the geomembrane to withstand field situations where a given elongation is imposed to the geomembrane.

Finally, although the study presented above has been restricted to unreinforced geomembranes, a similar approach can be used to analyze the behavior of a reinforced geomembrane after failure of the reinforcing fabric. A large elongation occurs in the then unreinforced small portion, while the elongation remains small in the portion where the reinforcing fabric is still intact.

2 STRESS TRANSFER THROUGH A SEAM

2.1 Seam Tests

Two types of tests are used to evaluate geomembrane seam strength: the peel test and the shear test (Fig. 3).

Peel Test The force per unit width causing a seam to fail in a peel test does not depend on the geomembrane thickness: its value is typically between 1 and 3 kN/m (5 and 20 lb./in.). Such values are small, compared with the force per unit width at failure in a tensile test on a geomembrane sample without seam for which typical values are between 4 and 70 kN/m (25 and 400 lb./in.), depending on the thickness of the geomembrane. However, there is an exception: seams in HDPE geomembranes do not normally fail in peel. In other words, the peel adhesion of a HDPE geomembrane is at least equal to its tensile strength, regardless of the thickness.

In addition, reinforced geomembrane may have a small ply adhesion, i.e., a small adhesion between the reinforcing fabric and the polymeric compound. Many reinforced geomembranes have a ply adhesion smaller than peel adhesion. Such geomembranes may fail by delamination next to a seam when the seam is subjected to a peel test.

Shear Test The force per unit width causing a seam to fail in a shear test is usually 80 to 90% of the force per unit width at failure in a tensile test on a geomembrane sample without seam. However there is an exception: for HDPE geomembranes, the above ratio is usually 100%.

The special behavior of HDPE geomembranes in peel and shear tests suggests that the two tests may not be independent. In other words, the performance of seams of

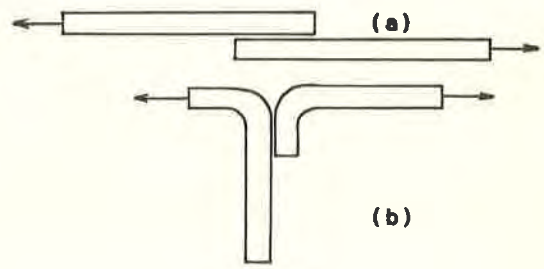

Fig. 3 Seam tests: (a) shear test; (b) peel test.

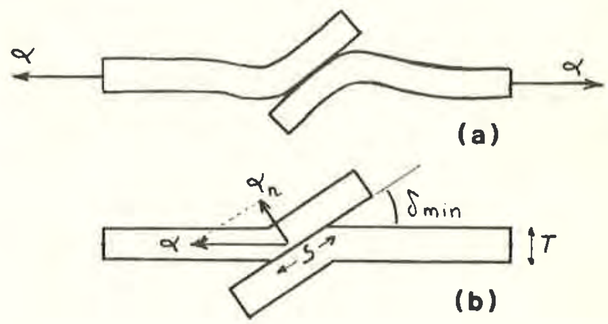

Fig. 4 Sample with a seam subjected to a shear test: (a) likely shape of the seam; (b) approximate shape used in calculations.

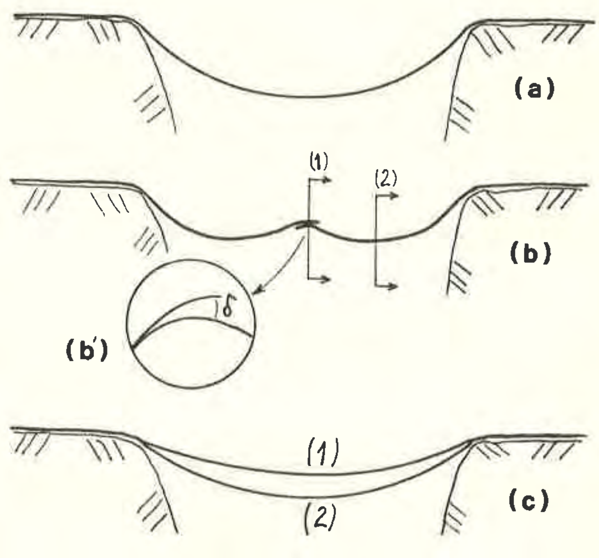

Fig. 5 Shape of a geomembrane bridging a depression: (a) cross section of geomembrane without a seam; (b) cross section perpendicular to the seam of a geomembrane with a seam; and (c) cross sections along the seam (1) and parallel to the seam (2) showing less deflection of seam due to double stiffness resulting from double thickness. Double stiffness has only a little direct effect on cross section (b) because it affects only a small portion of this cross section, but the geomembrane shape in cross section (b) is significantly affected by continuous stiffness of the seam in cross section (c1). Such tridimensional effect would not exist if the depression was an infinitely long trough perpendicular to cross section (b).

geomembranes other than HDPE when subjected to a shear test may be affected by their small peel adhesion. This is discussed in the following sections.

2.2 Peel Component in a Seam Shear Test

When a geomembrane sample with a seam is subjected to a shear test, the center of the seam tends to align itself with the direction of the applied force (Fig. 4a). As a result, the seam rotates and probably does not remain plane. Therefore, it is difficult to define an angle of rotation of the seam. However, as shown in Fig. 4b, a minimum value δ_{min} of the angle of rotation can be defined as follows:

$$\delta_{min} = \sin^{-1}(T/S) \qquad (21)$$

where: T = geomembrane thickness; and S = seam width.

As shown in Fig. 4b, the force per unit width α applied to the sample has a component α_n normal to the plane of the seam:

$$\alpha_n = \alpha \sin \delta_{min} \qquad (22)$$

The seam may fail by peeling or by delamination if the normal component α_n is larger than peel adhesion. To minimize the risk, the angle δ_{min} should be as small as possible. Combining Eqs. 21 and 22, it appears that at least the following condition should be satisfied to ensure that α_n is smaller than ply adhesion:

$$S/T > \alpha_f / \alpha_p \qquad (23)$$

where: S = seam width; T = geomembrane thickness; α_f = force per unit width at failure in a tensile test on a geomembrane sample without seam; and α_p = peel adhesion or ply adhesion, whichever is less.

Eq. 23 is necessary but not sufficient since δ_{min} defined by Eq. 21 is only a minimum value for the angle between the plane of the seam and the direction of the forces (as discussed in section 2.3). For a given geomembrane (characterized by T, α_f and α_p) a minimum seam width can be calculated using Eq. 23. Typical examples are presented in Table 2. The following comments can be made:

- The minimum seam width calculated according to Eq. 23 varies over a wide range, from 0.6 mm to 65 mm (0.02 in. to 2.58 in.).
- The required seam length increases when the thickness and the tensile strength of the geomembrane increase, the main factor being the tensile strength.
- Reinforced geomembranes require wider seams than unreinforced geomembranes due to their high tensile strength and low peel or ply adhesion. This is particularly true for heavily reinforced geomembranes because they have a low ply adhesion value as a result of small strike through (i.e., small openings of the reinforcing fabric, through which the polymeric compound on one side of the fabric can adhere to the polymeric compound on the other side of the fabric). In many instances seam width for reinforced geomembranes is 25 mm (1 in.) which is not sufficient for heavily reinforced geomembranes according to Eq. 23, as shown in Table 2.
- Seam width required for unreinforced geomembranes is smaller than 12 mm (1/2 in.). Typical 25 mm (1 in.) wide seams can therefore be considered as satisfactory for unreinforced geomembranes. From the point of view of Eq. 23, HDPE geomembranes constitute the ideal case where peel adhesion is at least equal to tensile strength which results in a very narrow required seam.

The above comments should not be isolated from their context. Stress transfer through a seam is a complex phenomenon and Eq. 23, based on simple assumptions, can be considered only as an approach leading to qualitative results. Values presented in Table 2 and discussed above should be used only to identify trends, such as the need for using wider seams for reinforced geomembranes than for unreinforced geomembranes.

While a thorough theoretical analysis of stress transfer through a seam would be complex, testing seams is relatively simple. Systematic testing of seams should be carried out to evaluate experimentally the influence of parameters identified in the above discussion.

2.3 Peel Components in Field Situations

The analysis presented in the previous section considered only forces in the cross section of a seam (Fig. 4). Such bidimensional analysis is not sufficient in some field situations where forces perpendicular to the considered cross section play an important role. A geomembrane bridging a depression is a typical example. The shape of the geomembrane with and without a seam is different, as explained in Fig. 5, due to seam stiffness in the direction perpendicular to the considered cross section.

As a result of the shape of the geomembrane shown in Fig. 5b, the two geomembrane sheets tend to exhibit an angle δ (Fig. 5b′) larger than the minimum value δ_{min} defined by Eq. 21 and Fig. 4. The observed value δ_{max} between the loose flap of the seam and the geomembrane can be considered as an upper boundary for δ . A value of the order of 30^0 has been observed by the author for δ_{max} on the seam of an unreinforced geomembrane bridging the corner of a reservoir after having undergone shrinkage. It may therefore be assumed that values of the order of 5 to 10^0 for δ are possible for unreinforced geomembranes. Values for δ are expected to be smaller for reinforced geomembranes than for unreinforced geomembranes since reinforced geomembranes are less deformable and consequently exhibit less deflection when bridging a cavity. Expected values for δ should be compared with the angle δ_ε to which a seam can resist when subjected to a force per unit width corresponding to a certain elongation of the geomembrane. According to Eq. 22, this angle does not depend on the seam length and its value is given by:

$$\delta_\varepsilon = \sin^{-1}(\alpha_p/\alpha_\varepsilon) \qquad (24)$$

where: α_p = peel adhesion or ply adhesion, whichever is less; and α_ε = force per unit width in the geomembrane at a given elongation ε .

Values of δ_{10} corresponding to a 10% elongation are given in Table 2. It appears that heavily reinforced geomembranes can withstand only small angles (2 to 4^0) resulting from three dimensional phenomena such as the one depicted in Fig. 5. This results from the low adhesion compared with the high tensile forces existing in a reinforced geomembrane, even at small elongation. On the contrary, unreinforced geomembranes can withstand very large angles. Even at a 50% elongation, the accepted angle would still be larger than 30^0 for all unreinforced geomembranes considered in Table 2.

2.4 Conclusions Regarding Stress Transfer Through Seams

Due to small values of peel adhesion of all geomembranes except HDPE, seams are the weakest part of a geomembrane because in field situations, and even in a laboratory seam shear test, seams are subjected to peel

Table 2 Required seam width calculated using Eq. 23, and maximum seam angle at 10% elongation of the geomembrane calculated using Eq. 24. Tensile strength, and peel and ply adhesion values are from NSF standards (2).

REINFORCEMENT			UNREINFORCED				LIGHTLY REINFORCED		HEAVILY REINFORCED			
Polymeric Compound			PVC			HDPE	CSPE (Hypalon)					EIA
Thickness	T	mm mils	0.25 10	0.75 30	1.14 45	2.5 100	0.75 30	0.91 36	0.91 36	1.14 45	1.5 60	0.75 30
Tensile Strength	α_f	kN/m lb/in	4 23	12 69	18 104	26 150	17.5 100	21 120	35 200	35 200	52.5 300	70 400
Minimum (Peel, Ply) Adhesion	α_p	kN/m lb/in	1.75 10	1.75 10	1.75 10	26 150	1.75 10	1.75 10	1.2 7	1.2 7	1.2 7	1.4 8
Tensile Strength/Minimum Adhesion	α_f/α_p	-	2.3	6.9	10.4	1	10	12	29	29	43	50
Required Seam Width (Calculated with Eq. 23)	S	mm in	0.6 0.02	5.2 0.21	12 0.47	2.5 0.10	7.5 0.30	11 0.44	26 1.03	32 1.29	65 2.58	38 1.50
Force per unit width at 10% elongation	α_{10}	kN/m lb/in	0.2 1	0.5 3	0.9 5	26 150	9 50	10.5 60	17.5 100	17.5 100	26 150	35 200
Maximum Seam Angle at 10% elongation (Calculated with Eq. 24)	δ_{10}	°	90	90	90	90	12	10	4	4	3	2

stress components. The theoretical study presented above is based on simple assumptions and the obtained numerical values should only be considered as a means to express qualitative trends. It appears that the weakening of seams caused by insufficient peel adhesion is more marked for heavily reinforced geomembranes than for lightly reinforced or unreinforced geomembranes. Further studies including refined theoretical analyses and systematic laboratory testing should be undertaken to verify the findings of the preliminary approach presented herein. If these findings were confirmed, it could be concluded that seams for heavily reinforced geomembranes should be wider than for other geomembranes.

As for now, it is suggested that the peel test be considered not only as an index test for seam quality control but also as a means to evaluate the ability of a seam to withstand actual stress components in the field.

CONCLUSIONS

The study presented in the first part of this paper, although it could be further refined, leads to a clear conclusion: geomembranes exhibiting yield should always be used in situations where the elongation is smaller than an allowable elongation obtained by dividing the elongation at yield by a factor which may be of the order of 2. Ignoring this recommendation can lead to failures, just as lack of recognition of this phenomenon has led to failures in the past. With the increasing use of yield and creep-prone geomembranes, which is justified by some other desirable characteristics, a thorough knowledge of geomembrane mechanical behavior is required on the part of designers.

The study presented in the second part of this paper does not lead to clear-cut conclusions because the assumptions considered may be simplistic. Nonetheless, a trend has been identified: heavily reinforced geomembranes seem to require wider seams than lightly reinforced or unreinforced geomembranes. Due to the importance of the subject and the difficulty of theoretically analyzing stress transfer through a seam, a systematic testing program is recommended.

The two studies presented in this paper confirm that common sense in engineering is as reliable as tossing a coin, and more dangerous since it is more believable. Common sense dictates that a geomembrane exhibiting 700% elongation cannot fail in actual situations where imposed elongations rarely exceed 50% and only in limited areas. However, such a geomembrane can indeed fail under a 15% elongation as shown by analysis and experience. Again, common sense dictates that the stronger the reinforcement, the higher the factor of safety. However, in the cases discussed in this paper, a strong reinforcement may decrease the safety of seams. The same may occur in cases not discussed in this paper, such as differential settlements, where a stiff reinforcement is clearly detrimental.

It is hoped that the discussions presented in this paper will increase designers' awareness of the need for a thorough knowledge of geomembrane mechanical behavior to meet the challenges of liner design. It is also hoped that more extensive research work will be undertaken to better understand the mechanical behavior of geomembranes.

This paper results from personal and independent work done by the author, using published information and personal observations. The technical content of this paper has not been discussed prior to publication and is therefore the author's sole responsibility. Since some of the results and comments presented in this paper may have important practical consequences, the author requests that quotations from this paper not be isolated from relevant context.

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Design of Earth and Concrete Covers for Geomembranes

In many cases, geomembranes installed on slopes are covered by concrete or a layer of earth. Earth covers are subject to slip along the geomembrane as well as internally. Concrete covers subject the geomembrane to gravitational as well as thermal expansion-contraction stresses. In this paper, the authors present design methods for predicting earth cover stability considering an internal failure as well as a geomembrane interface failure. They also present an analysis of the stresses induced in geomembranes by concrete cover differential movements. In both the earth and concrete cover sections, the effects of geotextiles installed adjacent to the geomembrane are also discussed.

INTRODUCTION

In many instances, geomembranes used to line reservoirs, dams, and canals are protected by an earth or a concrete cover from damage caused by weather, shocks, waves, current, vandals, etc. Movements of covers can cause problems.

Large movements resulting from instability of earth cover on a slope can affect the integrity of the cover and damage the geomembrane. Two types of such movements are considered herein: (i) slide within an earth cover; and (ii) slide at earth cover/geotextile/geomembrane interface.

Small differential movements between a concrete cover and a geomembrane which result from thermal expansion-contraction induce tensile stresses in the geomembrane.

Three analytical studies are presented in this paper, devoted respectively to the two types of instability and the differential movements mentioned above. These studies were made for the design of actual projects and they are presented here as design examples.

In sections 2 and 3, the effects of geotextiles used in association with the geomembrane are discussed as related to earth slide along the geomembrane and differential concrete-geomembrane movements.

1 SLIDE WITHIN AN EARTH COVER

1.1 Presentation of the Problem

A typical earth cover is shown in Fig. 1. Instability may result from a slide along the geomembrane, as discussed in Section 2, or a slide within the earth cover, as discussed below.

When the liquid level in the reservoir is kept constant for a long period of time, the earth cover is saturated to the same level. Then, if the liquid level in the reservoir is lowered, the liquid level in the earth cover moves down. Assuming that liquid flow in the earth cover is parallel to the slope, the maximum velocity at which the liquid level in the earth cover is moving down is:

$$v = k \sin^2\delta \qquad (1)$$

where: k = coefficient of permeability of the earth cover (m/s); and δ = slope of the earth cover.

If the drawdown of the reservoir is faster than the maximum drawdown velocity, v, in the earth cover, the earth cover exhibits excess pore water pressure caused by the level difference between liquid in earth cover and liquid in reservoir. As a result, the earth cover may become unstable. This situation, known as "rapid drawdown", may exist not only when a reservoir is emptied, but also, locally, when waves cause repeated temporary drawdown of small amplitude.

The rapid drawdown situation is usually the most critical for earth stability. Therefore it is recommended to design earth covers considering the rapid drawdown situation. A classical conservative approach consists of assuming that drawdown is instantaneous. It is classical geotechnical engineering knowledge that liquid flow inside the earth is horizontal immediately after such drawdown, and becomes parallel to the slope after excess pore pressure dissipates.

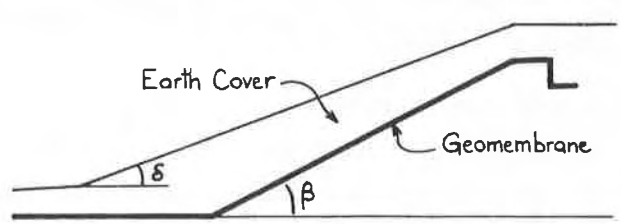


Fig. 1 Cross section of a typical earth cover. The slope of the earth cover is not necessarily the same as the slope of the geomembrane.

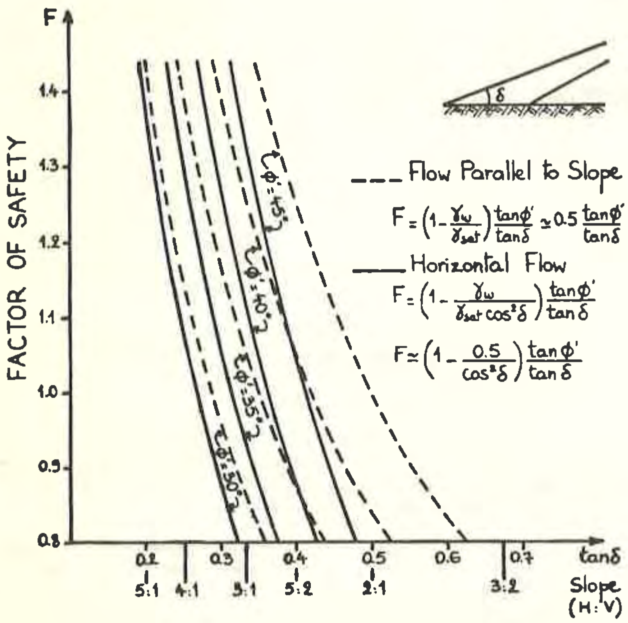


Fig. 2 Chart giving the factor of safety against sliding inside an earth cover during a rapid drawdown. Equations are classical for an infinitely high slope. Notation: Φ' = effective angle of internal friction of the earth; δ = earth cover slope; γ_w = unit weight of water (9810 N/m^3); and γ_{sat} = unit weight of saturated earth (N/m^3).

1.2 Evaluation of the Factor of Safety

The chart presented in Fig. 2 gives the factor of safety of the earth cover against sliding within the earth. This chart has been established with the assumption that the earth cover is infinite, thereby neglecting the buttressing effect of the soil at the toe of the cover. With the same assumption, the factor of safety with no flow, i.e., if the reservoir level is kept constant, would be $F = \tan\Phi'/\tan\delta$.

A factor of safety of the order of 1.3 is typically recommended. In a reservoir where ore processing plant tailings with $\Phi' = 35^0$ were used as cover material, the chart showed that a slope of 4/1 was necessary to ensure a factor of safety of 1.3 in case of rapid drawdown. By considering only the case of a constant liquid level, a slope of $1.9/1 = 1.3/\tan\Phi'$ would have given the same factor of safety. An earth cover with such an external slope would have been unstable in case of rapid drawdown.

To complete the evaluation of the stability of an earth cover, the risk of a slide along the geomembrane should be considered.

2 SLIDE ALONG THE GEOMEMBRANE

2.1 Presentation of the Problem

The geomembrane located under the earth cover, or the geotextiles, if any, placed under or on the geomembrane can act as slip surfaces thereby facilitating a slide of the earth cover. Therefore, the design of an earth cover should evaluate the risk of sliding at any of the cover/geotextile/geomembrane/geotextile/dike interfaces. A method is presented hereafter.

2.2 Principle of the Method

The method consists of evaluating the forces involved in the stability of the earth cover, which leads to one of the following three results: (i) the earth cover is stable; (ii) the earth cover is unstable and slides on the liner; and (iii) the earth cover tends to slide with certain elements of the liner (e.g., the geotextile, if any, in contact with the earth cover; or the geotextile and the geomembrane). In the latter case, the earth cover is stable if the dragging force on the liner elements moving with the geomembrane is smaller than (i) the tensile strength of the these liner elements, and (ii) the pull out resistance of the system (such as an anchor trench) anchoring the liner at the top of the slope.

2.3 Description of the Method

The method is described using the example of an actual reservoir. The cross section of the dike, the geotextile/geomembrane liner, and the earth cover is shown in Fig. 3a. The geomembrane is secured on the crest of the dike by the weight of a layer of compacted fill (zone 4 in Fig. 3a) used as a road. This way of securing the geomembrane is at least as effective as a classical anchor trench and has the advantage of preventing infiltration of liquid through the crest, which was essential in the considered reservoir.

To simplify the analysis, the earth pressure between zone 4 on one hand, and zones 2 and 3 on the other hand, is neglected. The only link between zone 4 and the earth cover on the slope is the force per unit width in the geotextile at the top of the slope. The force per unit width in the unreinforced geomembrane is neglected. If a reinforced geomembrane had been used instead, its force per unit width would have had to have been taken into account in cases where slide occurs at the lower interface of the geomembrane.

2.4 Pull-out Resistance at the Top of the Dike

The maximum value, α_{max}, of the force per unit width in the geotextile is given by Eq. 2 in Fig. 3. This equation expresses the equilibrium of zone 4. The value of α_{max} is represented by the curve (4) in Fig. 3b. If the force per unit width, α, induced in the geotextile by the downward movement of the earth cover sliding along the slope exceeded the maximum value, α_{max}, the geotextile would be pulled out of the crest of the dike.

2.5 Stability Analysis

The force per unit width, α, in the geotextile can be derived from a stability analysis of the earth cover on the slope. A classical wedge analysis has been used. The earth cover has been divided in two wedges by a vertical line located at the toe of the slope. The wedge located on the right of this vertical line (Fig. 3a) has been divided in two zones: rock layer (zone 2) used as a wave protection zone, and transition zone (zone 3) between the rock layer and the liner. The equilibrium of wedges 1 and 2-3 is represented by the force-per-unit-width diagram in Figure 3c. The lower part of the diagram is related to wedge 2-3 and leads to Eq. 3. The upper part of the diagram is related to wedge 1 and leads to Eq. 4. Combining Eqs. 3 and 4 and eliminating the unknown earth pressure P across the vertical separation between the two wedges yields Eq. 5. This equation gives the value of the force per unit width, α, induced in the geotextile by the earth cover on the slope. If the value of α is larger than the maximum value α_{max}, the earth cover is unstable.

The value of α has been determined in three cases discussed in the following two sections.

2.6 Stability During Filling of the Reservoir

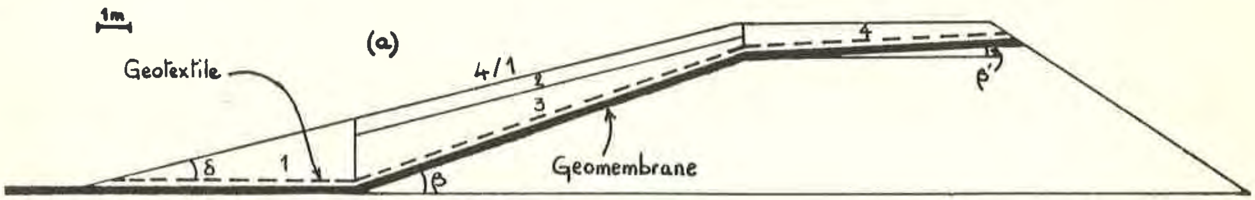
The most critical case during filling occurs when the level of liquid reaches approximately the top of wedge 1 (Fig. 3a) because: (i) the resistance opposed to sliding by wedge 1 is small since it then results from the submerged weight of the wedge; and (ii) the forces that tend to cause sliding are high since most of wedge 2-3 is then not submerged.

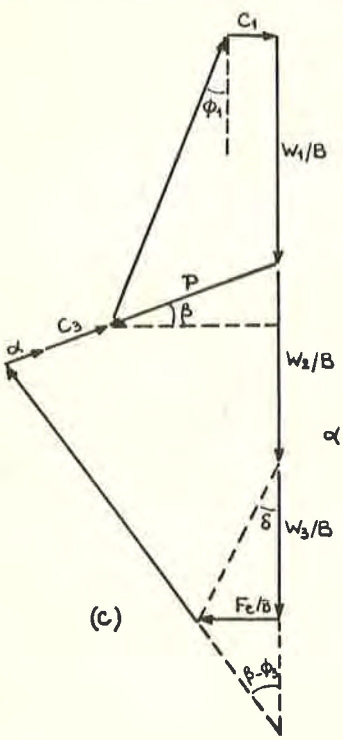
The force per unit width, α in the liner (i.e., geotextile and geomembrane) is given by curve 1 in Fig. 3b, obtained using Eq. 5 with no flow force (since there is no flow in the cover) and with the assumption that interfaces are cohesionless ($C_1 = C_3 = 0$) and that $\phi_1 = \phi_3$. It appears that:

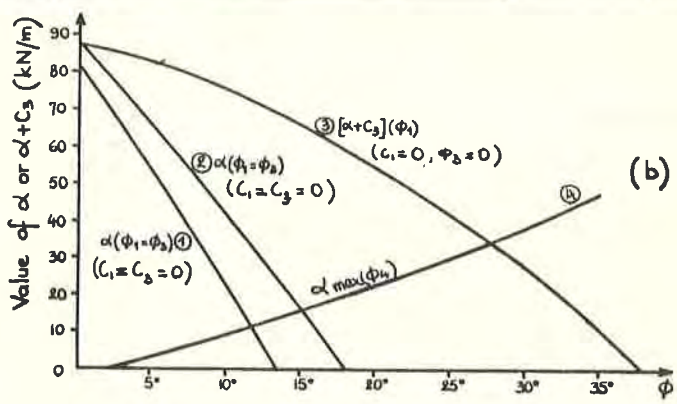
- If all interfaces (i.e. earth cover/geotextile, geotextile/geomembrane, and geomembrane/dike) have friction angles larger than 13.5^0 the earth cover is stable and does not induce tensile stresses in the liner.

- If the friction angle between earth cover and geotextile is less than 13.5^0, the earth cover is unstable.

- If one of the interface friction angles below the geotextile (i.e., geotextile/geomembrane and geomembrane/dike) is smaller than 13.5^0, the earth cover is stable if (i) the tensile force in the geotextile is smaller than the maximum force given by the curve (4) and geotextile tensile strength; and (ii) if the earth/geotextile friction angle is large enough to permit stress transfer between earth and geotextile and geomembrane.







$$\alpha_{max} = \frac{W_4}{B}\left(\cos\beta'\tan\phi_4 - \sin\beta'\right) \qquad (2)$$

$$\left[\frac{W_2}{B}+\frac{W_3}{B}\left(1+\frac{\tan\delta}{\tan(\beta-\phi_3)}\right) - (P+\alpha+C_3)\sin\beta\right]\tan(\beta-\phi_3) - (\alpha+C_3)\cos\beta = P\cos\beta \qquad (3)$$

$$\frac{W_1}{B}\tan\phi_1 + P\sin\beta\tan\phi_1 + C_1 = P\cos\beta \qquad (4)$$

$$\alpha + C_3 = \frac{\left[\frac{W_2}{B}+\frac{W_3}{B}\left(1+\frac{\tan\delta}{\tan(\beta-\phi_3)}\right)\right]\tan(\beta-\phi_3) - \left(\frac{W_1}{B}\tan\phi_1 + C_1\right)\frac{\sin\beta\tan(\beta-\phi_3)+\cos\beta}{\cos\beta-\sin\beta\tan\phi_1}}{\cos\beta+\sin\beta\tan(\beta-\phi_3)} \qquad (5)$$

(d)

Fig. 3 Slide along the geomembrane: (a) Cross section of the considered geomembrane-lined dike; (b) Chart valid only in the particular case of the dike presented above; curve (1) is related to the stability during filling and curves (2) and (3) to the stability during rapid drawdown; the method consists of comparing α with α_{max}, value of pull-out resistance at the crest of the dike; (c) Force-per-unit-width diagram (F_e is the flow drag force in zone 3 in case of rapid drawdown assuming horizontal flow; $F_e = W_3 \tan\delta$ if the submerged density of zone 3 is one half its saturated density which is usually approximately the case); (d) Equations where: W_1, W_2, W_3, W_4 = weight of zones 1,2,3, and 4 respectively; Φ_1 and Φ_3 = friction angle along the geomembrane in zones 1 and 3, respectively; C_1 and C_3 = cohesion multiplied by length of geomembrane in zones 1 and 3, respectively; P = unknown force across the vertical line between wedge 1 and wedge 2 - 3; α = force per unit width in the geotextile (the force per unit width in the unreinforced geomembrane being neglected); and B = width perpendicularly to the plane of the cross section. Note: in Eq. 5, $F_e = 0$ is equivalent to $\delta = 0$.

2.7 Stability During Rapid Drawdown

The worst case of drawdown occurs when the liquid level is drawn down from the maximum level to approximately the level of the top of wedge 1. The forces are the same as in the case of filling at the same level with, in addition, flow drag forces in zone 3.

In this case, Eq. 5 gives curve 2 in Figure 3b if the interface is assumed cohesionless, and curve 3 if the interface is assumed frictionless. ($C_3 \neq 0$, $C_1 = \Phi_3 = 0$). The latter case can occur if the clay which constitutes the dike is saturated (perhaps as a result of leakage through the geomembrane). This case is interesting because the stability in case of leakage should always be verified since it may be the worst case. In fact, for this particular dike, it is not the worst case, as can be deduced from Fig. 3b. Curve (3) shows that even if $\Phi_1 = 0$ (which is a very conservative assumption), the required value of $\alpha + C_3$ is 87 kN/m. The undrained cohesion between geomembrane and clay was estimated as 15 kN/m², hence C_3 = 15 x 11.8 = 177 kN/m (11.8 m being the length of the geomembrane on the slope). The factor of safety is 177/87 = 2, without any tensile stress in the geotextile ($\alpha = 0$), and would be higher with $\alpha \neq 0$.

2.8 Conclusion

Although the evaluation of the stability along the geomembrane has been presented using an actual example, the method is general and a chart such as Fig. 3b can be established in each specific case.

3 DIFFERENTIAL MOVEMENTS

3.1 Presentation of the Problem

Cracked concrete canal linings can be rehabilitated using geomembranes protected by concrete covers. Differential movements between the concrete cover and the old cracked concrete lining can cause undesirable stresses in the geomembrane. A reinforced geomembrane can be selected to withstand these stresses. An alternate (or complementary) solution consists of placing a geotextile between an unreinforced (or a reinforced) geomembrane and the concrete cover, and/or between the geomembrane and the cracked concrete lining. The study presented hereafter shows by which mechanism stresses in the geomembrane can be alleviated by a geotextile placed between the geomembrane and the cracked concrete lining.

3.2 Stresses in Geomembrane in Contact with Concrete

The situation to be analyzed is presented in Fig. 4. The width of a crack, initially negligible, increases to b as a result of a symmetrical lateral displacement of the concrete lining on both sides of the crack. Equations presented in Fig. 4 are related to the behavior of the geomembrane on the right side of the crack.

The shear stress, qf_{ms}, on the lower face of the geomembrane is larger than the shear stress, $q'f_{cm}$, on the upper face of the geomembrane because the normal stress q under the geomembrane (which results from water pressure plus submerged weight of concrete cover) is larger than the effective normal stress q' on the geomembrane (which results from the submerged weight of the concrete cover). As a result, the geomembrane is dragged toward the right by the moving concrete lining. The value of the force per unit width, α, in the geomembrane results from a straight forward integration. The force per unit width decreases linearly from a maximum value, α_{max}, at the edge of the crack, to zero at a distance L from the crack. The value of L increases with increasing values of the geomembrane modulus, K_m, and the crack width, b. The stress in the geomembrane can be obtained by dividing the force per unit width by the thickness of the geomembrane.

3.3 Stresses in a Geotextile-geomembrane System

The situation to be analyzed is presented in Fig. 5. If the friction coefficient, f_{ts}, between the geotextile and the supporting concrete, and/or between the geomembrane and the geotextile, is very small, the geotextile acts as a lubricant (i.e., slip surface) and the geomembrane does not move. As a result, the opening of the crack does not induce stresses in the geomembrane. However, in most cases, it is expected that the smaller shear stress in the system will be between the geomembrane and the concrete cover for the reasons indicated in section 3.2. Therefore, the geomembrane moves as a result of geotextile movement caused by cracked concrete lining movement.

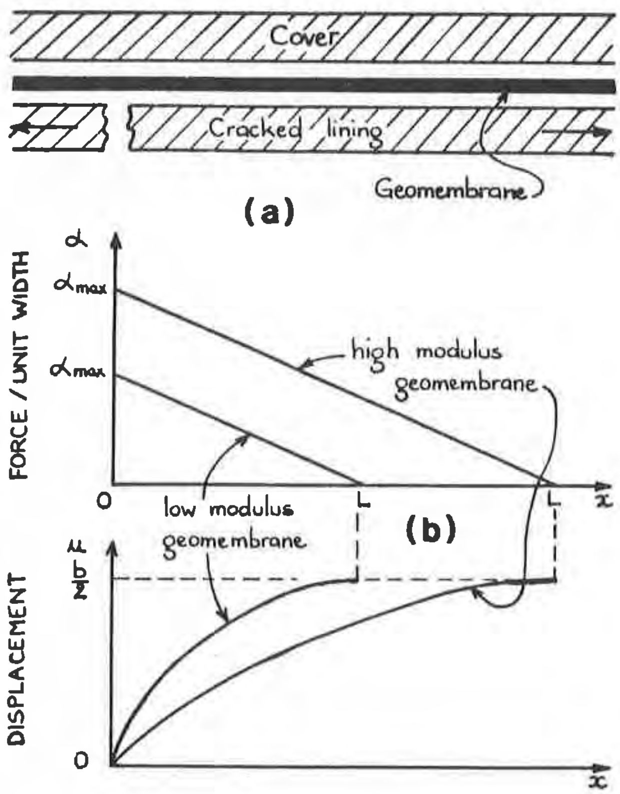
If the geotextile has a higher tensile modulus than the geomembrane, the length L (see Fig. 5) related to the geotextile tends to be larger than the length L related to the geomembrane. Consequently, the geotextile exerts on the geomembrane shear stresses which tend to stretch the geomembrane and equate the length of the moving part of the geomembrane to the length of the moving part of the geotextile. Stress transfer between the portions of length L of the geotextile and the geomembrane exhibiting the same elongation is governed by the adhesion coefficient, a_{mt}. The calculations presented in Fig. 5 are valid only if the adhesion coefficient, a_{mt}, is smaller than the friction coefficient, f_{mt}, between the geotextile and the geomembrane. If not, the geotextile and the geomembrane move with respect to each other, and the geomembrane is subjected to tensile stresses smaller than the stresses calculated below.

3.4 Design Example

The analyses presented in the two previous sections were developed for the design of a geomembrane liner placed over a cracked concrete lining. The maximum amplitude of crack movement was estimated as 20 mm. The geomembrane was an unreinforced butyl rubber with a thickness of 0.75 mm and a tensile modulus of 2 kN/m. The geotextile was a needlepunched nonwoven with a modulus of 55 kN/m. Friction coefficients measured in shear box tests were as follows: between geotextile or geomembrane and concrete, $f_{ms} = f_{ts} = 0.5$; and between geotextile and geomembrane, $f_{mt} = 0.7$ (this latter value also applies to geomembrane/concrete cover since the concrete cover was cast on a needlepunched geotextile).

The water depth was 4m and the thickness of the concrete cover was 0.1m. Consequently, the water pressure was 39240 N/m², the normal effective stress on the geomembrane was q' = 1373 N/m² and the total normal stress under the geomembrane was q = 40613 N/m².

Using Eqs. 10 and 11 it appears that the maximum force per unit width in the geomembrane is α = 880 N/m when there is no geotextile (Fig. 4), and α = 165 N/m when the geotextile is used (Fig. 5). The presence of the geotextile decreases the force per unit width (and, consequently, the stress) in the geomembrane by a factor of 5. Using Eqs. 9 and 13 shows that the length of geomembrane that elongates is 45 mm when there is no geotextile and 243 mm when there is a geotextile. In other words, the geomembrane elongation is five times smaller when there is a geotextile because the 20 mm displacement resulting from crack opening is distributed over a fivefold length. As a result, the force per unit width is five times smaller.



(d)

$$d\alpha = (q'f_{cm} - qf_{ms})\,dx$$

$$\alpha = (qf_{ms} - q'f_{cm})(L - x)$$

$$\alpha_{max} = (qf_{ms} - q'f_{cm})\,L$$

$$u = \int_0^x \varepsilon\,dx = \int_0^x (\alpha/K_m)\,dx$$

$$u = (qf_{ms} - q'f_{cm})(L - x)^2/2K_m$$

$$b = 2u_{max} = (qf_{ms} - q'f_{cm})\,L^2/K_m$$

$$L = \sqrt{bK_m/(qf_{ms} - q'f_{cm})} \qquad (9)$$

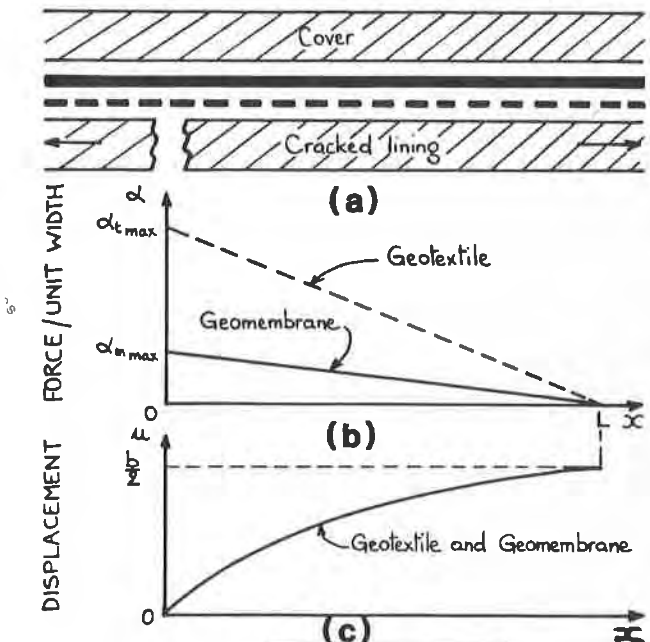
$$\alpha_{max} = \sqrt{bK_m(qf_{ms} - q'f_{cm})} \qquad (10)$$

Fig. 4 Geomembrane between a concrete cover and a cracked concrete lining: (a) Cross section; (b) Force per unit width in the geomembrane; (c) Displacement of the geomembrane; and (d) Summary of calculations, equations where: α = force per unit width in the geomembrane; q = normal stress below the geomembrane (N/m^2); q' = effective stress above the geomembrane (N/m^2); f_{cm} = cover/geomembrane friction coefficient; f_{ms} = geomembrane/supporting concrete friction coefficient; L = length beyond which the geomembrane does not move and is not stressed (m); K_m = tensile modulus of the geomembrane (N/m); and b = width of the crack(m).

3.5 Conclusion

Geotextiles can alleviate stresses in geomembranes resulting from movement of concrete linings in contact. Several cases can be considered: (i) geotextile with a very low friction coefficient and any tensile modulus and strength; (ii) geotextile with a high tensile modulus and strength and any friction coefficient; and (iii) a geotextile with a high tensile modulus and strength, glued to or otherwise associated with the geomembrane. An alternate solution to the latter case consists of using a reinforced geomembrane (i.e., reinforced with a scrim inside the geomembrane). The geotextile has another advantage: it protects the geomembrane from abrasion, scratching and snagging by the cracked, rough concrete lining.

The study presented above is devoted to cases where a geotextile has a high friction coefficient and a high tensile modulus and strength, i.e., cases (ii) and (iii).



(d)

$$\alpha_{m_{max}} = \sqrt{bK_m(qa_{mt} - q'f_{cm})} \qquad (11)$$

$$\alpha_{t_{max}} = \sqrt{bK_t\,q(f_{ts} - a_{mt})} \qquad (12)$$

$$L = \sqrt{bK_m/(qa_{mt} - q'f_{cm})} = \sqrt{bK_t/(q(f_{ts} - a_{mt}))} \qquad (13)$$

$$a_{mt} = (q'f_{cm}K_t + qf_{ts}K_m)/(q(K_m + K_t)) \qquad (14)$$

Fig. 5 Geomembrane and geotextile between a concrete cover and a cracked concrete lining: (a) Cross section; (b) Force per unit width in the geotextile and the geomembrane; (c) Equal displacement of the geotextile and the geomembrane, and (d) Equations obtained from calculations similar to those presented in Fig. 4, where: a_{mt} = geomembrane/geotextile adhesion coefficient; f_{ts} = geotextile/supporting concrete friction coefficient; and K_t = tensile modulus of the geotextile (kN/m).

ACKNOWLEDGEMENTS

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The earth stability analysis was done by the senior author when he was Director of the Geotextiles and Geomembranes Group of Woodward-Clyde Consultants, of which the junior author was a member.

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Selection Criteria for the Use of Geomembranes in Dams and Dykes in Northern Climate

This paper presents the results of a research project initiated in 1982 to establish selection criteria for synthetic and bituminous membranes used as impermeable barrier in dams and dykes to be constructed in northern regions. Due to lack of suitable construction materials at the Robertson Lake Project, northern Quebec, such an alternative was considered. It is proposed to install a geomembrane sandwiched between two geotextiles on the upstream face of the rockfill dam. The selection of the geomembrane, the type of glue and the fusion techniques used to provide sealed joints between geomembrane rolls and the need to glue geotextiles to the geomembranes under extreme temperatures have been studied. Specially manufactured specimens were tested under very cold temperatures ranging from 23°C to -35°C and results of tensile, bursting, puncture, flexibility and frictional tests are presented and discussed as a function of temperature. Finally test results from synthetic and bituminous-elastomer mixtures are compared and selection criteria are proposed.

INTRODUCTION

A program has been initiated to replace costly electrical power plants burning Diesel fuel and located in isolated localities of northern regions. The selected alternative is to replace them with small hydro-electric plants. The first choosen site is on the Ha-Ha river and the project is known as the Robertson Lake Project. The proposed dam about 40 meters in height and approximately 150 meters in length must be impermeabilised at its upstream face to contain water from the river. The reservoir water level is to fluctuate seasonally with an estimated maximum range of 9 meters during the winter months. Design considerations and detailed site conditions are to be found in a companion paper given in the session on dams and embankments.

Impermeabilisation of a rock fill dam is commonly provided by a clayey core compacted in thin layers (8). Due to lack of suitable construction material close to the site, the alternative of using a synthetic or bituminous membrane sandwiched between geotextiles and installed on the upstream face of the rock fill dam has been considered. To protect and support the impermeable liner against puncture from floating debris and ice action, a 1.5 meters thick layer of rip-rap must be placed upstream of the sandwich liner. Also, prior to the installation of the geotextile/membrane/geotextile system, a 1.5 meters thick layer of finer transition material must be set on both sides of the sandwich to protect it against the action of sharp edges of the rocks.

The impervious membrane will be submitted to very large temperature fluctuations in the upper section of the dam (temperature will vary from +23 to -35°C) for the following reasons: a) extremely cold temperatures during winter seasons; b) freeze and thaw cycles during spring and autumn seasons; c) the lowering of the reservoir level during winter months. Since the properties of geomembranes are affected by the temperature, their hydraulic and mechanical behaviors will also fluctuate with time. Furthermore, to cover the large upstream surface of the dam, many strips or rolls of membrane will be used such that an overlapping technique will be used to seal the joints. These glued or fusion type joints will also be affected by the temperature fluctuations.

Under extreme cold temperature, most of the available commercial synthetic membranes will be affected as they become brittle and stiff. If their glass transition temperature is higher than -35°C, then breaking of the membrane occurs. On the other hand, the present technology enables manufacturers to alter membrane composition by incorporating in the mixture, elastomers and binder agents to lower their glass transition temperature and therefore keep their elastic properties under colder temperatures. One must be aware that the life expectancy of any polymeric/elastomer and bituminous/elastomer membranes is lowered as the quantity and quality of incorporated agents are altered with time. A compromise between membrane life and elastic behavior under cold conditions must be reached. It should also be pointed out that all geomembranes and sealed joints cannot be considered inert materials and therefore a curing period is needed following the membrane production and joint installation. Moreover, these materials are altered in time from UV attack, solvent migration, oxydation reactions, and other extreme climatic conditions such as hot and cold temperatures.

Methods to install and anchor membranes have been developped (5) but great care should be given to those membranes which will be affected by cold temperatures. Cracking at potential locations such as anchor bends, folds formed during installation may occur at cold temperatures. Design of the upstream face of the dam and installation procedure must be planned to minimize the non-uniformity and bend angles of a thin geomembrane.

Membranes can be altered by chemical attacks such that an exact composition of the retained liquid must be known. Because of the remote location of the works and the contaminant dilution in the reservoir retained water, the danger of a chemical attack is almost nonexistent.

A literature survey has shown that many data are available with regards to membrane properties at temperatures above freezing point but very few data are available for subzero temperatures for most of the market known geomembranes. Because of the northern location of the proposed dam site, the behavior of these membranes under cold temperature conditions cannot be mispredicted and laboratory tests must be performed on samples at

+23 as well as -35°C. The existing tests to measure properties of polymeric, rubberized, bituminous or textile products are quite different and do not sollicit membrane specimens in a way as to simulate loads transmitted to a membrane installed in a dam. Test apparatus and procedures must then be developped to measure pertinent properties that could be used to select membranes and to compare advantages and disadvantages of each type of membranes to be installed in a dam located in a northern climate.

In this presentation, we will discuss properties related to dam application and present their behavior as a function of temperature. Also apparatus used in the testing will be described shortly and results will be discussed. Selection criteria of membranes to be used in a northern climate will be presented.

MEMBRANE PROPERTIES

A geotextile and a geomembrane installed in a dam will be sollicitated in such a manner that pertinent properties must be known prior to their selection. These properties have been defined as: a) brittleness under cold conditions; b) stress-strain properties; c) resistance to bursting; d) life expectancy and the sealed joints behavior under cold temperatures; e) permeability and f) structural changes under freeze and thaw cycles. In this paper, the first four properties will be discussed.

A) Brittleness and Stiffness

A literature survey (1) disclosed that thermoplastic materials are becoming less flexible and more brittle as the temperature is lowered. Under cold temperatures and under freeze and thaw cycles, these properties will control the membrane behavior. Similarly, bituminous products will become brittle and less flexible under extreme cold conditions. On the other hand, elastomers will keep their flexibility under a wide range of encountered temperatures. Since most of the available membranes are produced from a mixture of polymemer or asphaltic chains linked with an elastomer and others compounds to stabilize the product, the resulting composition can be fixed to obtain the desired behavior under cold temperatures. As the temperature at which the membrane will be submitted, is lowered, changes occur within its structure until a critical temperature is reached at which drastic and irreversible changes can occur. This is usually known as the glass transition temperature and the membrane becomes very brittle if submitted at temperatures lower than this value.

B) Stress-Strain Properties

A membrane under traction elongates in the direction of the applied load. At low strain, its resistance is usually proportional to the deformation and the resulting elastic modulus, E, can be measured. As the displacement is increased, a yield point is reached showing a change in its structure and usually a strain level at which permanent deformation appears. As the load is increased the membrane elongates until its breaking point. The temperature influences greatly the elastic behavior of a membrane such that its elastic modulus increases with the lowering of temperature so the yield point and the breaking point appears sooner (at lower strain).

C) Resistance to Puncture and to Bursting

The main purpose of membranes is to be impermeable to flow when a hydraulic gradient is applied such that any rupture or puncture of this impervious layer is unacceptable. At high temperatures, the puncturing danger and the bursting risks are a function of the elastic properties of the membrane (4). At very cold temperatures, their resistance to puncture and bursting is increased as long as the glass transition temperature is not exceeded (lower temperature) in which case breaking can occur from changes of structures.

D) Sealed Overlapping Joints

Membranes are available in rolls or strips of small width because of the limited process equipments and also their mass per unit area. As a result, rolls of membrane must be overlapped on the dam face and sealed joints performed using a glueing method or a fusion technique. Quality of the joints is a function of the material type to be sealed, the selected sealing material, the procedure and the expertise of the installation team. Also very little is known on the influence of subzero temperatures as well as the freeze and thaw cycles on the life expectancy of sealed joints. However, it is believed that cristallisation of the sealing material can occur under cold conditions.

GEOTEXTILE/GEOMEMBRANE/GEOTEXTILE SYSTEM

Non-woven geotextiles are not affected by cold temperatures due to their very large void ratio, very low moisture retention and relatively low modulus of elasticity. These geotextiles are used to sandwich an impervious membrane to protect it against puncture. As of a consequence, interfaces are created between the membrane and the textile as well as between the textile and adjacent soil layers. These interfaces offer a weak resistance to slipping, thereby increasing the risk of a downward movement of the rip rap. A suitable membrane must then be choosen and the slope of the upstream face of the dam selected to insure its stability. The adherence between all components must be sufficient to overcome the downward gravity loads resulting from the weight of the rip rap, from the ice action as the reservoir water level decreases and from any differential dam movement. The friction angle between soils, textiles and membranes must be sufficient to insure mechanical stability. Temperature may also play a role because, as it is lowered, the stiffness of the membrane can be increased resulting in a variation of the friction angles.

The adherence between the geomembrane and the protecting geotextile can be increased if both materials are glued together and/or surface-treated. Assessment must be made to determine the glueing material and the technique to be used, to determine the quantity of glue to apply and the extent of variation in the system properties as a result of the additional layer of glue.

The presence of the textile layers glued to the membrane will impose additional resistances that must be measured. At high temperatures, it is believed that the geotextile properties will control the mechanical behavior of the sandwich system while the geomembrane is expected to do so at cold temperatures since the mechanical properties of geotextiles does not appear to be adversely affected by subfreezing temperature (2). Another important property of the sandwiched material is its flexibility which can influence the easeness of installation on the dam slope.

LABORATORY INVESTIGATION UNDER COLD TEMPERATURES

A research program was initiated to establish membranes as well as geotextiles/geomembrane system behavior under cold temperatures. Twenty-one different synthetic and bituminous membranes were selected and tested under temperatures of +23°C, -5°C, -15°C, -25°C and -35°C. Their strength characteristics were measured and compared to assess their behavior under these temperatures. Detailed testing procedures and test results are given in reference (7).

Due to the lack of uniform tests to perform on elastomer, thermoplastic, textile and bituminous products and also the need to simulate load transmission to membranes as expected in a dam, laboratory apparatus were designed and build. These test apparatus were to stand at cold temperatures and were to permit measurement of properties at the five choosen temperatures. Four apparatus were built to measure the following membrane properties: tensile strength, bursting strength, puncture resistance and friction resistance. The membrane flexibility was measured using the ASTM D-1388-64 test procedure.

A) Tensile Strength

Membrane specimens, as shown on Figures 1 and 2, were placed between two steel grips of 250 mm width; 10 steel bolts were used to secure the membrane within the plates of each grip to insure that no slippage would occur. The distance between the grips is approximately 100 mm and the travelling distance of the bottom grip is 150 mm corresponding to a maximum straining of the specimen equal to 150%. The applied displacement and the resulting strain in the specimen were recorded continuously and the resulting curve of stress versus strain could be plotted. The rate of strain has been set to 3.8 mm per min.

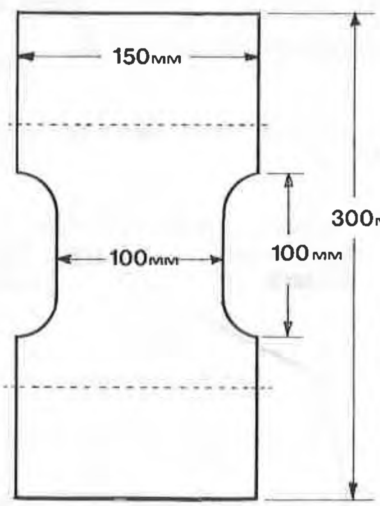

Figure 1: Membrane Specimen Dimensions

Each sample was cut into many specimens of dimensions presented on Figures 1 and 2 to insure that the breaking of the membrane would occur mid-distance between the grips avoiding end and grip effects. The width of the specimen, 100 mm at its center section, is sufficient enough to avoid excessive striction of the specimen, that could have affected the measurements.

Both the apparatus and the installed membrane specimen were placed in a controlled temperature room for a 24 hours period prior to the beginning of the test in order to ensure that the specimen would be thoroughly at the selected temperature and also to minimize handling that would have damaged the specimen at very low temperatures.

The calibration of the apparatus was obtained from trial tests performed at all selected temperatures and the reproductibility of the results was checked with many specimens of the same membrane. Twenty-one single piece specimens were tested and specimens with joints were also tested at the five different temperatures. As shown on Figure 2, the overlapping length of the joints is 50 mm such that the specimens are of 350 mm in length. This overlap was found to be large enough for all materials except the bituminous materials tested at a temperature of +23°C where slipping occured.

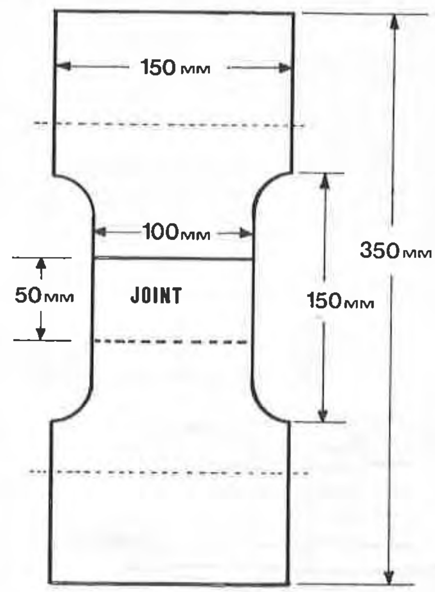

Figure 2: Membrane Specimen Dimensions with Joint

B) Bursting Strength

As shown on Figure 3, a circular membrane specimen greater than 150 mm in diameter is installed between the flanges of the top and bottom sections of a steel cylinder. The top plate is bolted to the lower section as the specimen is secured in place. On the top section, an air line is connected, a pressure gauge and regulating valve are installed on that line. A drain is connected at the bottom section of the cylinder and used to evacuate air when needed. The controlled air pressure acts on the specimen to deform it in a spheric shape.

Similarly to the procedure followed with the tensile strength test, the apparatus and the specimen were allowed to reach the selected temperature in a controlled temperature room for a 24 hours period. Air pressure of 50 kPa was then applied and gauge readings taken.
The obtained deformation was measured by a marker that touches the membrane at its center. By step of 50 kPa, the pressure was increased until bursting occurred as noted by a sharp decrease in the air pressure in the system and by measuring air flow in the bottom drain.

C) Puncture Resistance

The apparatus used to measure bursting strength was slightly modified to measure the puncture resistance. Sub-angular material, gravel with diameter between 19 and 25 mm, filled up the bottom section of the cylinder and resin was used to fix in place the top gravel layer to insure that all specimens would be submitted to the same number of edges. The membrane specimen was installed between the top and bottom sections of the apparatus and care was taken that it is laid on the granular material. Similarly, pressure on the specimen was gradually increased by step of 350 kPa at every 5 minutes until puncture occurred as noted by a sudden drop in the air pressure or up to the capacity of the apparatus (1050 kPa).

D) Friction

The adherence between different materials of the suggested system was measured with the apparatus described on Figure 4. The geotextiles and the membrane specimen were installed between two boxes filled up with sub-angular aggregate material corresponding to the one that will be used in the dam. A load was applied normally on the top box while a mechanism applied a horizontal shear force on the bottom box. This horizontal load was slowly increased as the friction developped through the interface so to reach its maximum value. Similarly to the others tests, the apparatus and the specimens were placed in a controlled temperature room prior to the test.

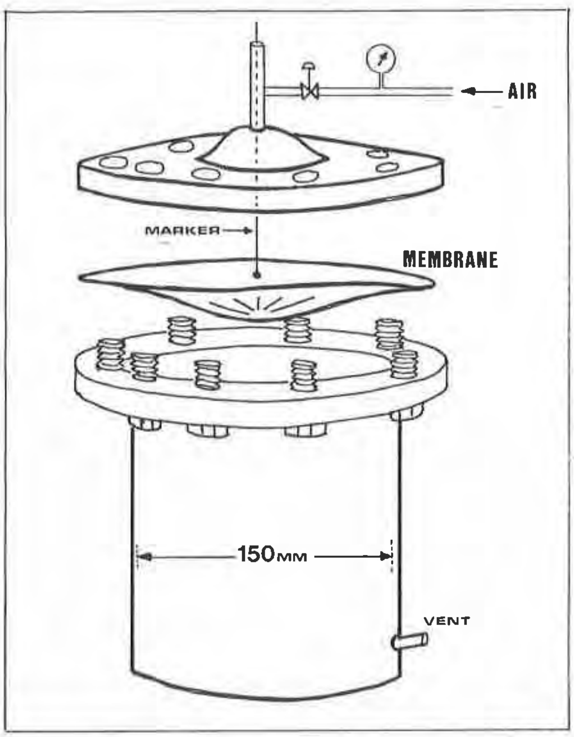

Figure 3: Bursting Strength Apparatus

RESULTS

In the research program, a total of 21 different geomembranes were selected and tested to characterize their behavior.

In this paper, few data are presented to stress the temperature influence on membrane behavior. They are presented on Figures 5 to 10 in a way such as to compare difference in behavior between thermoplastic, elastomer and bituminous materials.

A) Tensile Strength

The experimental curves presented on Figure 5, indicate that the membrane tensile strength is increasing with decreasing temperature. The membrane resistance increases linearly with strain until a yield point is reached and then the rate of the resistance growth is reduced until breaking occurs. Also it can be observed that the product becomes more brittle as the temperature is lowered. Below a temperature of -20°C a very different behavior was found for most of the membranes.

To compare the different behaviors of a thermoplastic, a rubberized and a bituminous membrane, data are presented on Figure 6. For both materials, curves obtained at +23°C and at -35°C, are plotted on the same Figure. One can observe that the elastic behavior of the elastomer (E) at a very low temperature (-35°C) compares to the large resistance of the thermoplastic (T) and of the bituminous-elastomer (B).

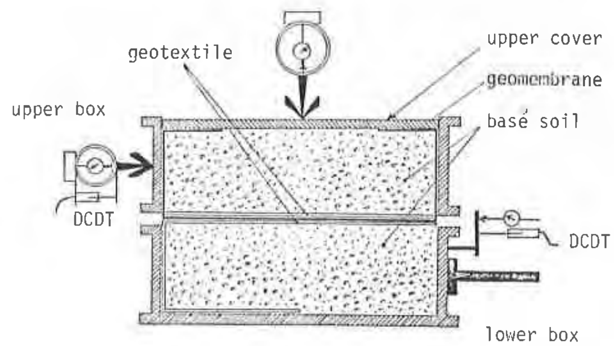

Figure 4: Apparatus used to Determine Adherence Between Materials

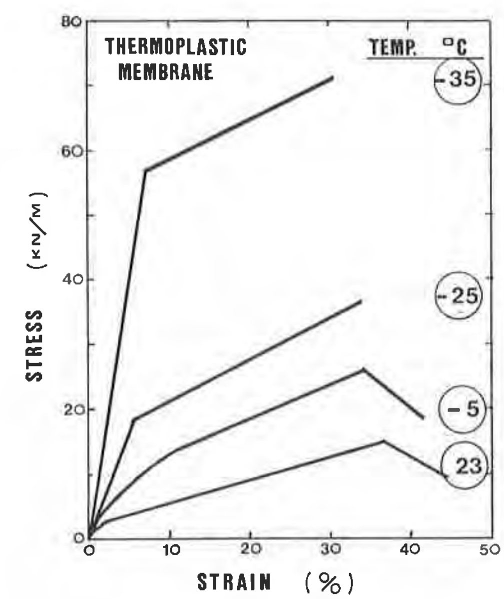

Figure 5: Temperature Influence on Tensile Strength

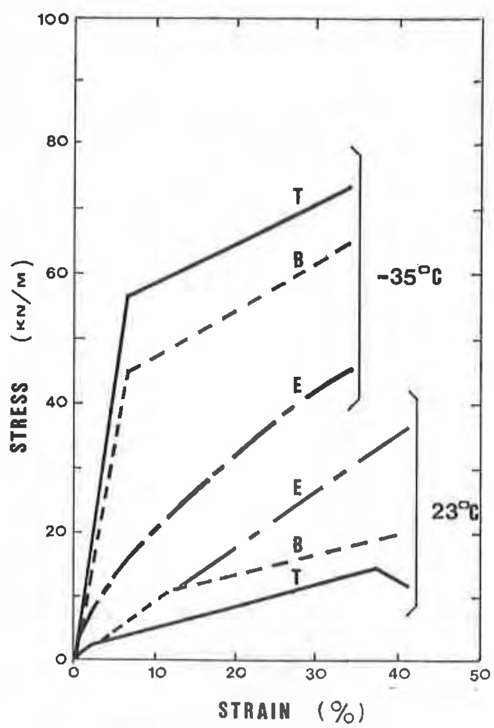

Figure 6: Stress-Strain Curves of Various Types of Geomembrane

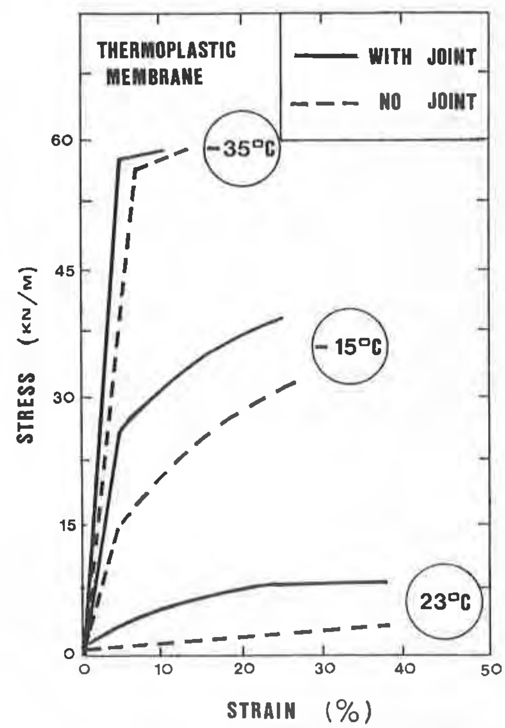

Figure 7: Tensile Strength of Overlapping Joints

B) Joint Quality

The resistance of the sealed overlapping joints has been measured to determine their quality as a function of temperature. Membrane specimens with a 50 mm joint located at mid distance between the grips position, as shown on Figure 2, were tested in the tensile strength apparatus. For each temperature , the stress-strain curves of the jointed specimen were obtained and compared to the single piece sample as shown for a thermoplastic material on Figure 7. It was found that joint resistance was greater than the resistance of the single-piece membrane and that it increases linearly with strain until a yield point was reached. For all thermoplastic and elastomer materials tested, the sealed joints behave well under the temperature range of +23°C to -35°C. On the other hand, it was found that an overlapping length of 50 mm was not sufficient for bituminous products at a temperature of +23°C since slipping has been observed.

C) Bursting Strength

The bursting strength of membranes were measured as a function of temperature. As shown on Figure 8, most of the specimens bursted during testing at pressures ranging from 100 to 1050 kPa with lower values for elastomer and thermoplastic materials at higher temperatures.

D) Puncture Strength

The results obtained from the puncture tests indicated that all membrane specimens were not punctured under an applied pressure of 1000 kPa for 5 minutes at +23°C and -35°C. The angular edges of the gravel particles of the granular layer adjacent to the impervious material cannot then be considered as a potential risk to punctu-

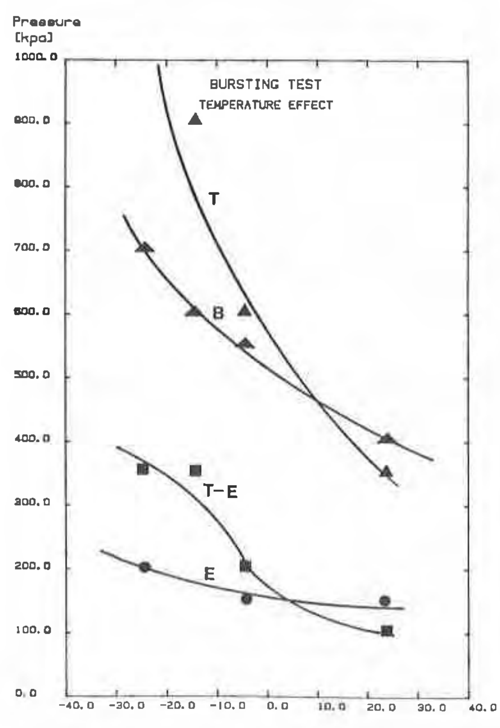

Figure 8: Bursting pressure -vs- Temperature

re the membrane when resulting pressure from the rip rap and the water is considered.

E) Friction Angle

The friction angles between both membrane-soil and sandwiched material-soil were measured as a function of the particles size to assess the adherence between the different materials of the upstream face of the dam and to design the dam slope to insure its stability. As shown on Figure 9 for example, it was found that the friction angle of the unglued sandwiched material (20°-26°) in contact with the soil was lower than that of the membrane vs soil (30°-40°) and than that of the non-woven geotextile vs soil (28°-34°). These latter values corresponds to the results of Ingold (6). These results suggest that the adherence is minimum between the membrane and the geotextiles and therefore glueing of the textiles on the membrane is needed to increase the rip rap stability against sliding. Finally no significant influence of the temperatures was noted.

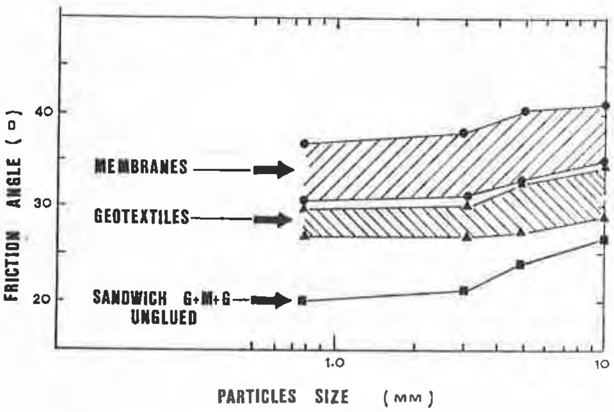

Figure 9: Adherence of Membranes and Geotextiles on Soils

F) Flexibility

As shown on Figure 10, the stiffness moduli of membranes are plotted as a function of temperature for four different types of geomembrane. The membrane flexibility decreases exponentially with decreasing temperature with higher stiffness moduli obtained with thermoplastic materials and lower values obtained with elastomer materials. Intermediate values were found for the bituminous-elastomer mixture materials.

SELECTION CRITERIA

Many criteria must be met by a geomembrane in order to be installed safety in a dam. These criteria are well known and have been reported in the literature (3,5). In this paper, we are presenting only the additional criteria that are important when a membrane is installed in a northern climate with the possibility of reaching a temperature of -35°C.

A) Deformability

The selected membrane must exhibit a low module of elasticity under a wide range of temperatures (from +23 to -35°C) to allow a sufficient strain before reaching its yield point or breaking point. But one must be aware not to choose a too weak membrane that will offer not enough resistance to applied loads. Under stresses the membrane must be impermeable and must participate in the slope stability by resisting the applied loads. A compromise between polymeric or bituminous compound and the mixed elastomer must be reached without affecting the life expectancy of the membrane.

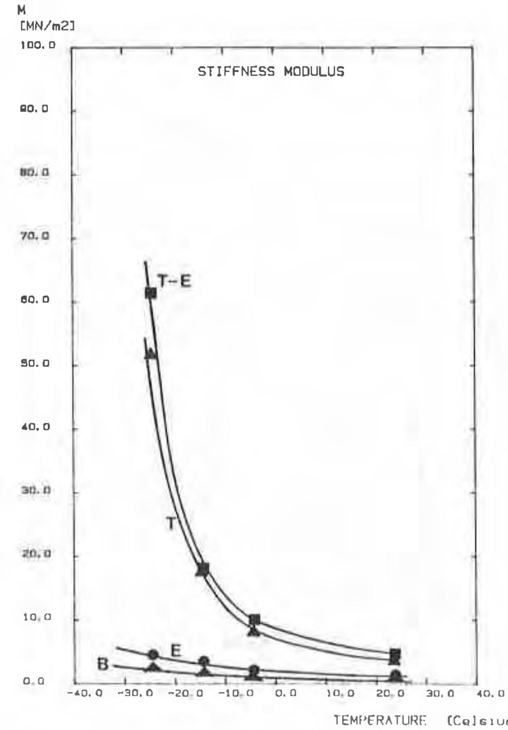

Figure 10: Stiffness Modulus -vs- Temperature

B) Brittleness

The selected membranes should not be affected by the temperature fluctuations such as to break under expected loads especially at edges, joints and folds. Under cold conditions, one should avoid to use a membrane that becomes brittle enough that any load transmitted from the ice action on the rip rap might damage the membrane. Similarly the types of glue used to seal overlapping joints and to increase adherence between the textiles and the membrane should not be affected by cold temperature. Any product with a glass transition temperature lower than -35°C is acceptable.

C) Anchoring Technique

The membrane to be laid down on the upstream face of a dam should be anchored at the top section of the dam. Care should be taken to select a membrane with a high flexibility and to avoid sharp bends that will represent sites for stresses to build up when the membrane will be submitted to loads under cold temperatures. The risk of breaking the membrane at anchor location under those conditions can be very high.

D) Overlapping of Rolls

The overlapping width of rolls should be determined to forecast the slipping risk. Tests must then be performed at the expected higher temperatures at which the risk of slipping is greater because, as believed, most joints will behave better at colder temperatures unless the seal material becomes brittle. Fusion techniques as well as sealing materials should be thoroughly checked under cold temperatures prior to the selection of the geomembrane.

CONCLUSION

Geomembranes are not inert materials. They are affected by UV attack, migration of solvents and among other factors extreme temperatures conditions. As found in this study, mechanical properties of thermoplastic and bituminous membranes were drastically changed under sub-zero temperatures. To insure elastic behavior under cold conditions, elastomer can be added but one should be aware that the stability of the material can be affected. Moreover, it should be pointed out that not only the right material should be selected but also compatible techniques should be used in the following areas: overlapping of membrane strips, installation procedure, thermal isolation of the upper part of the dam, slope angle, and others.

ACKNOWLEDGEMENTS

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Design of Fill Dams Including A Geomembrane

Although a geomembrane easily fits to complicated shapes, it was noted that dams including a geomembrane have simple structures : constant slope, no berm, rectilinear axis. The membrane supporting structure fullfils different mechanical and hydraulic functions : as far as rockfills are concerned the transition is firstly a granulometric one. However, the supporting layer is stabilized through impregnated gravel, bituminous concrete...etc... A special emphasis is laid on both the local (under the geomembrane) and general (through the fill) drainage systems. The general principles of mechanics are applied to the design of connections with both the foundations and the crest. But, some examples drawn from the available dams show that the selected designs are sophisticated.

Laboratory tests were carried out to analyze the lining stability together with its protection.

Other uses but less widespread-of geomembranes in fill dams are described.

When new technologies in the field of dam construction are put forward, the first reaction is obviously one of concern for public safety. This question is rarely left to independent judgement, but is defined by texts which usually suggest an administrative procedure. Generally speaking, the three following categories can be distinguished in France :

a) Dams affecting public safety (application of the government circular dated 14 August 1970) ; the dams in categories b and c are automatically included in this group.

b) Dams whose design projects are examined by the Comité Technique Permanent des Barrages (compulsory examination for all dams higher than 20 metres, the measurements being taken from the crest to the natural soil straight below it).

c) Dams requiring the installation of a warning system. This only concerns dams over 20 metres high with a storage capacity of more than 15 million cubic metres.

The French first used geomembranes as liners on small dams which did not come into any of the categories mentioned above, and they have only been building category b structures for the last five years (Ospédale and Codole).

The design of these dams is based on about fifteen years experience ; moreover, there are now more than twenty dams lined with a geomembrane and higher than 10 metres. At 28 metres, the Codole dam is the highest of them all. The ROUCHAIN dam must be considered separately. This is a rockfill dam rising to a height of 60 metres above its foundations, and whose upstream lining, comprised of cement concrete, was repaired in 1983 by the laying of a geomembrane.

In the rest of this report, the details given apply to dams over 10 metres high. The following questions will be successively dealt with : the general design, support, connections, protection and less conventional uses of geomembranes.

I. General design.

As soon as one considers using a thin membrane in a fill dam, problems affecting the overall design are posed.

It is advisable to adopt a general rectilinear axis and to avoid berms. This provides an even surface for laying the membrane, without any angles formed by changes in gradient. In effect, any discontinuous line on the installation surface can cause complications with respect to the topography, and the execution of the work, particularly in the laying of the geomembrane without its being stressed, or developing folds or permanent deformations.

Only the Mas d'Armand dam (1) has a berm and, even if certain materials make it possible to use slip joints, the designers decided to keep a uniform gradient. This was the case for the lining of the La Coche dam (6), which was laid over à 60 m length of slope.

The watertightness of the foundations and the fill must be designed to form a continuous line. But geomembranes are adapted for use in carrying out the most difficult connection work between the fill and the foundations of the basin. One can cite the Revin dam (2), for which sheets of butyl were used in order to ensure connection between the upstream lining in bituminious concrete and the clay layer of the basin.

Generally speaking, the site of the dam must be chosen so that the foundations or the basin can be dealt with in perfect hydraulic, mechanical (loads, stresses) and geometrical (displacements, deformations) continuity.

The fill should not differ in any way from dams where other forms of lining are used. The choice of slope depends on the techniques of soil mechanics. However, it can be seen that in several rockfill dams, the slope is not as steep as it might have been and that the batter is between 1 in 1.6 and 1 in 2. The use of relatively gentle side slopes for rockfill dams enables the contractor to reduce the difficulties he will have to resolve with respect to the positioning of the support, the laying and protection of the membrane, and the execution and control of the joints.

With respect to the zoning of the fill, apart from the lining support, the designer's attention must be drawn to the increased security that a zoning of the fill can bring. Zoning as it was carried out on the Vallon d'Ol dam (2) involves the use of different materials or granulometric treatments in order to obtain contrasts in fill stability in the event of a large leakage. The supporting layer often constitutes the least permeable element in this zoning where permeability greatly increases from the upstream to the downstream part. The advantage of this system has been verified in the Codole dam, when in the course of the construction work the basin was partially filled before the lining had been laid ; this incident did not damage the fill in any way. The granulometric stability of the materials comprising the fill is evidently fundamental in case of leakage.

II. The geomembrane support.

Given the very different gradients adapted for fills depending on whether they are comprised of rock fill or fine soils, the two categories of dam must be dealt with separately.

2.1. Rock-fill dams.

As the typical profiles of Ospédale and Codole show, the support is comprised of multiple discontinuous layers with varying grain sizes, moduli of elasticity, deformations and permeabilities.

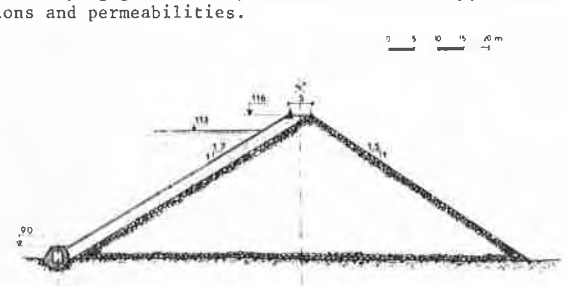


Fig. 1. Codole profile

The base layer is generally laid in successive horizontal layers ; its role is that of granulometric transition layer and it is re-profiled by compaction on the slope. The supporting layer is comprised of a finer, stabilized material : it might be sand stabilized with bitumen or cement, lean concrete, bituminous concrete or gravel impregnated with bitumen. One notices that, more and more frequently, the immediate support is comprised of a geotextile.

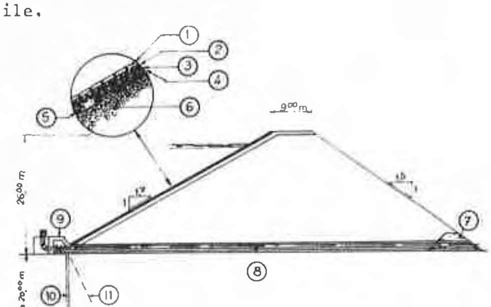


Fig. 2. Ospédale profile

1. Concrete blocks
2. Geotextile
3. Geomembrane
4. Geotextile
5. Gold bituminous concrete
6. 25/50 gravel
7. Toe weight
8. Ballast layer
9. Valve chamber
10. Grout curtain
11. Drainage curtain

2.2. Loose dams.

It is first of all advisable to obtain the final fill profile by the removal of excess of earth ; in any case, if profiling is carried out by bringing material onto the slope, it must be followed by effective compaction. Here also, the geotextile constitutes an immediate support which reduces the risk of puncturing.

2.3. Drainage.

A priori, one must distinguish between the general drainage of the fill and local drainage beneath the membrane.

General drainage is not indispensable in the case of rock-fill dams but two ideas would merit further study : fill zoning with increasing permeability towards the downstream part and division of the contact zone with the foundations. This latter procedure may consist of positioning watertight bands on the scraped earth of the foundations and also by placing a collector whose flow can be measured between two watertight bands.

This collector can be given a larger capacity by using a porous concrete tube, for example. The sizing must follow certain principles : increasing permeabilities, increasing drainage capacities. At all events, the areas collecting the drainage water must have a cross section in the order of 1 to 5 times the height of the fill.

In the case of the loose soil fill, general drainage of the fill is necessary and it must be in accordange with the sizing rules for fill dams. Generally speaking it is easy to ensure that its evacuation capacity corresponds to the following : leakage through the foundations, leakage through the fill, assuming the latter to be saturated insofar as this is not too disadvantageous, maximum water evacuation capacity arising from local geomembrane drainage. It must conform to the filtering principle, whether it be comprised of natural materials or of geotextiles. Taking into account general experience, the capacity may be in the order of the following value :
$q = \alpha H L$
where q is the drainage capacity (m^3/s)
H is the height of the dam in metres
L is the length of the dam in metres, and
α a coefficient varying between 10^{-6} and 10^{-5} m/s.

The local drainage beneath the lining should be related to the risk of leakage through the membrane.

III. Connection of the lining to the foundations and the crest.

3.1. General principles.

Like all dams with thin linings, the design of the peripheral anchorage of the geomembrane requires particular care. It is imperative for the waterproof barrier itself to be continuous and this must be reconciled with the mechanical discontinuity of the support, which is expressed by differential movements at the concrete/fill, fill/foundation contacts :

- irreversible movements linked to deferred settlements.
- reversible movements of an elastic nature, linked to the thermal and hydrostatic stresses.

The principles which must be observed are as follows :

- creation of transition zones so that particularities are not concentrated at certain points : no sharp edges, use of supple materials which can withstand large deformations.
- doubling and/or local reinforcement of the membrane.

- careful preparation of the lining support ; compressible materials should be well-compacted and concrete smoothed.

On the other hand, it is pointless to leave folds in the membrane as a way of dealing with large elongations, since they might well become blocked by hydrostatic pressure.

3.2. Connection with the waterproofing of the foundation.

• Clay-filled anchor trenches.

This technique is employed for low structures and is recommended if the foundations are compressible. It may be useful to move the anchor trench upstream of the toe of the dam in order to :

- avoid the foundation's zone of maximum shearing stress,
- to reduce the hydraulic gradients should their watertightness prove insufficient.

• Rigid structures.

Concrete cut off trenches, cut off walls or galleries usually ensure connection between the upstream lining of the dam and the waterproofing of the foundations (grouting, diaphragm ...).

The design of the toe structure must be in accordance with the principles outlined above (fig. 3), which involve :

- a concrete/membrane contact which is a smooth as possible, with a wide curving connection.
- a concrete/fill contact with a gentle enough slope to distribute differential settlements over a certain horizontal distance.
- construction joints sufficiently close together to avoid differential movements being too large between the sections.

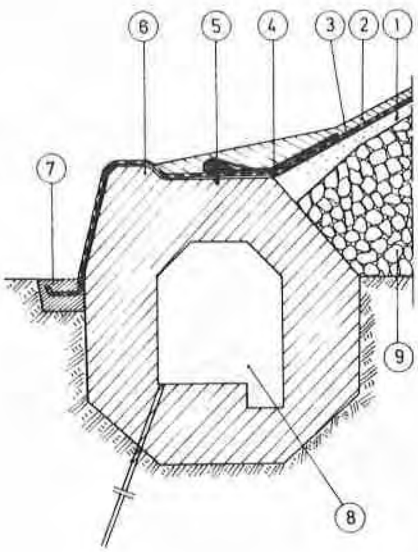


Fig. 3. Connection with a gallery.

1. Supporting layer.
2. Protection concrete slabs.
3. Geomembranes and geotextiles.
4. Secondary geomembrane.
5. Anchorage.
6. Concrete bench.
7. Clay-filled trench.
8. Gallery.
9. Rockfill.

The toe of the dam must, moreover, provide the abutment for the protective layer of the membrane, by means of a bench.

With respect to a gallery, the membrane can be passed across its upstream face down the foundation, in order to reduce the risk of leakage through the concrete.

The fixing of the membrane and protective geotextiles onto the concrete can be ensured, as in the case of the OSPEDALE and CODOLE (Corsica) dams, by a system of plates secured with bolts. Polyethylene bands which permit an overall tightening of the membrane, and are chemically inert, are an improvement on the metal plates.

It is advisable to double the membrane locally by a second (eventually reinforced) layer with acts as a joint cover in case of tearing of the first membrane.

Finally, the intersections between the construction joints of the toe structure and the concrete/membrane joint must be executed with great care.

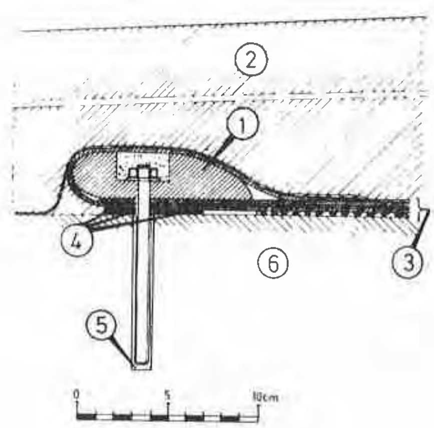


Fig. 4. Codole - Detail of anchorage.

1. H.D.P.E. band.
2. Concrete protection.
3. Five-layer diaphragm.
4. Mastic and 2 butyl bands.
5. Bolts.
6. Concrete of the toe gallery

3.3. Anchorage of the membrane to the crest of the dam.

The membrane must always cover the upstream face as far as the crest and connect with any eventual lining of the latter.

With the exception of very permeable rockfill dams, the design of the upper anchorage must prevent rainwater entering downstream of the lining, as this could create underpressure capable of lifting it off. The crest of the dam must be practically watertifht and inclined towards downstream.

On small earth dams, the membranes can be anchored by means of a simple clay-filled trench.

But generally speaking, dams with smooth upstream faces are fitted with a deflecting wave wall, a rigid structure whose lining connections are carried out in the same way as for the toe structure.

However, the hydrostatic head to be withstood is zero and therefore a simpler system of jointing is permissible.

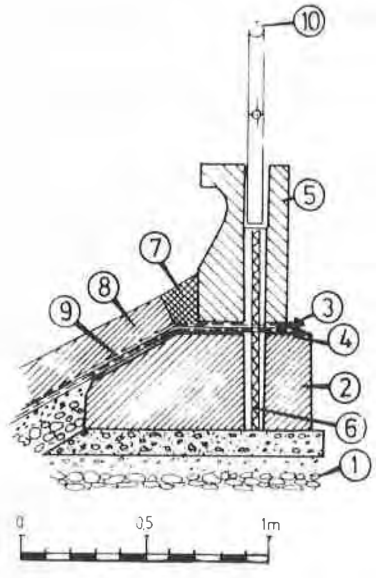


Fig. 5. Codole - Top anchorage.

1. Rockfill.
2. Concrete wall.
3. Steel plate.
4. Bolts (6 for 1 metre).
5. Wave-protection wall.
6. Iron anchorage.
7. Cement and sand.
8. Concrete.
9. Watertight facing.
10. Protection for visitors.

3.4. Other particularities.

Whenever possible all particularities in the upstream face (berms, changes in gradient) and toe structures must be avoided : the connection cutt off wall/gallery, the design of the upstream slide valve chamber must ensure the continuity of the connections.

IV. Protection of the membrane.

4.1. Is protection of the membrane necessary ?

On dams more than 10 m high, the mechanical characteristics of the prefabricated thin membranes currently in use are note adequate for them to be subjected, unprotected, to external aggressions :

- climatic aggressions : thermal shocks, wind on parts which are note submerged.
- biological aggressions : roots, algae, bacteria, burrowing animals.
- chemical aggressions : ultraviolet rays.
- vandalism or sabotage.
- swell, impact from floating bodies (wood, ice ...).

However, the experience of the 110 m high PARADELA dam in PORTUGAL, which was lined with an unprotected membrane constructed in situ in 1980, indicates a trend which could develop given the high cost of systems of protection (half the price of the whole lining) and inherent advantages for ease of inspection and repair of the membrane.

4.2. Determination of the design wave.

This question has been broached by numerous writers, in particular TAYLOR (4). The effect of swell on a slope is a complex phenomenon involving not only the height of the waves, but also their frequency, angle in relation to the facing and the slope and roughness of the latter. Swell can be assimilated to a series of waves whose amplitudes are a random variable. Empirical formulae suggest an estimation of the significant wave (mean value the upper third of the amplitudes) from the duration and speed of the wind and the fetch of the basin in the direction under consideration. The design wave is calculated on the basis of this significant wave by a multiplying factor which takes into account the vulnerability of the dam with respect to the overtopping.

Finally, in the vicinity of the facing, reflection phenomena bring about an increase in amplitude, to be considered using another multiplying factor which depends on the gradient of the slope and on the frequency.

4.3. Analysis of lining stability.

Schematically, the action of swell on a facing successively brings about an impact or overpressure effect and a blackflow or underpressure effect.

In view of these stresses whose direction fluctuates in space and time, stability can be attempted :

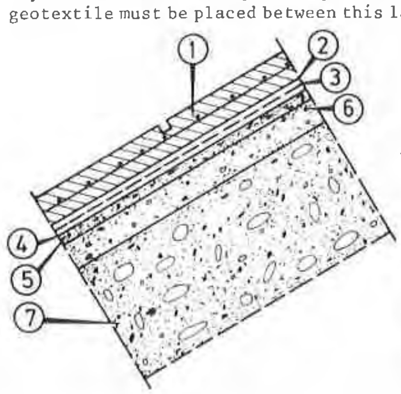
- either by the massiveness of the elements (rock-fill protections) ;
- or by their monolithism, and/or imbrication of the neighbouring elements, which makes it possible to integrate the stresses over a large enough surface for them to offset each other (protection by concrete slabs cast in situ, or paving comprised of prefabricated units).

For the design of the CODOLE dam (Corsica), a theoretical approach and tests on models were carried out at the same time in order to obtain the dimensions of the concrete protective cover. The amplitude of the crest to hollow of the facing whose batter is 1 in 1.7. In the complete absence of drainage under the protection, a monolithic slab, 10 m long in the direction of the slope, is self-stabilizing with respect to underpressures as soon as it is more than 0.14 m thick. Tests on a small-scale model carried out at the CEMAGREF confirmed this stability in respect of 1.75 m high waves.

On the other hand, it was ascertained under the same conditions that paving with self-interlocking, prefabricated blocks 0.50 x 0.20 m and 0.08 m thick, was unstable.

4.4. Protective techniques used.

Rock-fill protections necessitate a sand and rolled gravel layer underneath the to prevent puncturing of the membrane. A geotextile must be placed between this layer and the membrane.



1. Reinforced concrete (14 cm)
2. Geotextile
3. PVC geomembrane (2 mm).
4. Geotextile supporting layer.
5. Cold bituminous concrete
6. Impregnated 25/50 gravel.
7. 25/120 ballast.

Fig. 6. Diaphragm - Detail - Codole dam.

Another possibility is to construct protective layers in open-graded bitumen coated aggregates.

Concrete protections can only be used on fills with limited settlements, which is generally the case with rock-fill dams resting on rock and less than 50 m in height. They must be well-buttressed at the toe. They need a very even support. There is :

- paving with so-called self-locking prefabricated blocks, imbricated by a system of tenons or shoulders. These coverings are very susceptible to irregular laying ;
- protection using reinforced concrete slabs cast in situ. The joints must be designed so as to allow underpressure drainage.

V. Other uses of geomembranes in fill dams.

5.1. Repairs.

One can quote four examples which illustrate this type of use :

- the Alesani dam where the utilization of a reinforced bituminous membrane made it possible to reduce leakages between the bituminous concrete lining and the cement concrete cut off wall ;
- compensation basin (5) JUNKER described the technique of repairing cracks in the bulk of a bituminous concrete upstream lining using bands of polymerous bitumen ;
- the Paradela and Rouchain dams have upstream linings in cement concrete. The linings of these two dams have been completely re-covered in a membrane so that the leakages would be reduced to acceptable amounts.

5.2. Separation.

In the La Coche dam (6) a P.V.C. membrane constitues the secondary lining. In the Verney dam (7) the geomembrane is horizontal and linked to the connecting zone of the diaphragm (foundation) and of the bituminous concrete lining (fill). the membrane cuts off the drainage system of the lining from the foundations.

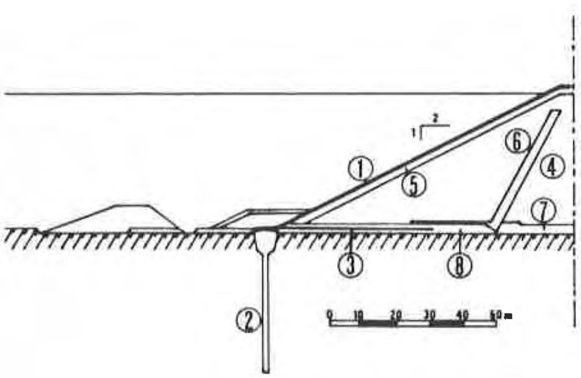


Fig. 7 Verney dam - Typical section.

1. Bituminous concrete.
2. Wall cut off.
3. Geomembrane.
4. Alluvium fill.
5. Processed alluvium.
6. Chimney drain.
7. Downstream drain.
8. Drainage blanket.

5.3. Continuity.

At the Revin basin, butyl was used to complete the continuity of the lining between the clay layer of the basin and the bituminous concrete upstream lining.

Conclusion.

The principles embodied in this report can be considered to be theoretical methods of approach based on the mechanics of a continuous medium. In effect, knowledge of the laws of behaviour of the materials utilized, properties at the interfaces and a good schematization of the overall behaviour of fill dams are essential elements in this method of approach.

However, the proposals made have resulted in their being applied to structures which have been built, making it possible to test their reliability. Finally, one can only hope that laboratory research and observation of existing structures will progress simultaneously.

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Geomembranes Used to Control Expansive Soil

Geomembranes were tested under field conditons to determine their ability to minimize damages caused to highways by expansive soils. These impermeable engineering fabrics were used on three projects in San Antonio, Texas by the State Department of Highways and Public Transportation. One test section used the fabric horizontally on the subgrade, two used it as a deep vertical moisture barrier. In all cases, tests for riding quality, pavement cracking and system strength on the horizontal section, the geomembranes were elements in maintaining a better riding highway sustaining less soil damages. Geomembranes performance merit further testing in controlling expansive soil's destructiveness.

INTRODUCTION

Geomembranes waterproofed engineering fabrics may help control the expansive soils problem. Expansive soils probably caused $10 billion damages in the United States in 1983. Their damages were estimated to be $2 billion a year in 1970 (1) and $7-9 billion in 1980 (2). Half the damages occur on highways, streets and roads. In the United States expansive soils extend from the Mexican to the Canadian border, from the Atlantic to the Pacific coast. They are a world wide problem. Expansive soils have been the subject of extensive studies by federal, state and local agencies, universities, research institutes and international conferences. (3,4,5)

Texas has, for several decades, looked for solutions. A variety of potential remedies have been used, including soil replacement, ponding, deep sand backfilled under-drains, chemical treatments, pressure injection, and geomembranes.The Texas State Department of Highways and Public Transportation is using geomembranes in San Antonio, a rapidly growing city in the south central part of Texas. Three projects involving geomembranes in an effort to control expansive soils in San Antonio area have a history of testing. The geomembranes are used in an effort to minimize pavement distortion by minimizing moisture change in the expansive clay subgrade.

All three sites have swelling soils of the Houston Black Terrace Association, deep calcareous clays that crack when dry and swell when wet. Surface drainage is slow when dry, rapid when saturated. Internally drainage varies from slow to none.

GENERAL McMULLEN DRIVE - A HORIZONTAL PLACEMENT

At the first test site a geomembrane was placed horizontally on a swelling soil subgrade in building an urban arterial street, General McMullen Drive. The 183m (600 ft) test section was installed in 1976. The pavement was severely distorted, cracked, uneven and dangerous to drive. The curbs and sidewalks reflected the irregular movements. Test holes at three locations provided samples that indicated plasticity indeces from 11 to 63, liquid limits from 26 to 72. The McDowell method potential vertical rise (PVR) estimates were 6.3cm (2½ in.) at station 194, 11.1cm (4½ in.) at station 224+50, and 8.1cm (3¼ in.) at station 238. The reconstructed section had 0.15m (6 in.) of flexible base, 0.28m (11 in.) of black base and a 0.04m (1½ in.) finish course. The 5.15km (3.2 mi.) contract, low bid at $1,698,425, began in January 1976 and completed in May 1977. This project, probably the first in Texas where a geomembrane was placed on an expansive soil subgrade to evaluate its effectiveness in controlling swelling clays, used DuPONT's coated Typar spunbonded polypropylene Styles 3353 and 3153 given by the manufacturer. It was delivered in 3m (10 ft.) wide rolls and used on a 183m (600 ft.) section 27.4m (90 ft.) wide from subgrade crown to crown and placed over the subgrade with the highest PVR, from station 226 to 232. Each roll lapped 0.30m (1 ft.) over the one laid previously, and laps were tacked with an emulsion. The rolls were placed by a six man crew. Trucks unloaded base which was then spread by a maintainer on the coated fabric. None of the operations tore the geomembrane though truck turning tended to pull it.

Adjacent 183m (600 ft.) roadway sections were used as controls for comparison. Roadway profilometer testing computer reduced to serviceability indeces (S.I.) were used in all three test projects to measure riding quality. An S.I. of 5 is the smoothest ride with descending values indicating a rougher surface. Readings were taken in July and December 1981, June 1982 and February 1983. In July 1981, the geomembrane sections had higher S.I.'s than the control sections in 5 of the 6 lanes. Their indeces varied from 3.86 to 2.64 while the control sections ranged from 4.14 to 2.17 (Table 1). This pattern continued through the tests to February 1983. Again 5 of of 6 highest values were maintained on the fabric coated lanes, indicating a smoother riding surface. The inside lanes generally indicated higher S.I.'s than the outside ones that would have the greater moisture variations.

Photologging measures the cracking of the pavement sur-

TABLE 1 - McMULLEN - SERVICEABILITY INDECES

	Outside			Center			Inside	
7-81	6-82	2-83	7-81	6-82	2-83	7-81	6-82	2-83
Northbound Lanes								
South Control								
2.75	1.95	2.00	3.67	3.25	3.14	3.85	4.13	4.02
Fabric								
3.63	3.58	3.58	3.86	3.59	3.52	3.41	3.33	3.26
North Control								
2.17	2.92	2.79	3.69	3.67	3.42	4.14	3.87	3.65
Southbound Lanes								
South Control								
2.32	2.16	2.11	3.09	2.94	2.87	3.41	3.31	3.27
Fabric								
2.64	2.61	3.83	3.83	3.84	3.81	3.86	3.75	3.81
North Control								
-	2.60	2.53	3.68	3.64	3.51	3.61	3.56	3.50

face. Openings in the pavement surface permits the intrusion of water which may contribute to subgrade moisture variation leading to pavement distortion. Photologging involves taking pictures in each lane of the pavement, with a camera mounted 2.44m (8 ft.) high on a moving vehicle. Each slide, showing a roadway section one lane wide and 2.44m (8 ft.) long, was projected onto a grid screen divided into 100 squares. The number of squares containing cracks were counted and calculated as a percent of the total. The first photologging took place in January 1981. Results ranged from 0 to 1.05% with the maximum cracking occurring in the non fabric protected sections (Table 2). The highest was 1.05% and 0.98% both on the non fabric protected section. January 1983 reflected the pattern with indicated maximum cracked areas in the north control of 2.71%. The south control pavement has a maximum lane cracking of 2.14%. The geomembrane sections had a maximum value of 0.95%. Both controls had more cracking than the coated fabric section.

TABLE 2 - McMULLEN - PHOTOLOGGING (% CRACKED - Abridged)

	Outside			Center			Inside	
1/81	1/82	1/83	1/81	1/82	1/83	1/81	1/82	1/83
Northbound Lane								
South Control:								
0.98	3.93	2.06	0	0	0.64	0	0	0
Fabric:								
0.36	0.53	0.95	0	0	0.17	0	0	0
North Control:								
0.54	0.40	0.77	0	0	0	0	0	0.06
Southbound Lane								
South Control:								
0.54	1.24	1.82	0.97	1.65	2.14	0.51	0	0.02
Fabric:								
0.26	1.49	0.95	0	0	0.03	0	0	0
North Control:								
0	1.32	0.72	1.05	0	2.71	0.69	0	0.14

Generally cracking of the pavement increased with the passing years. Also the inside lanes reflected less cracking than the outside lanes.

Elevations were taken on the finished pavement through 1983. The average movement, from the first reading, indicated little variation. The south control from stations 220 to 226 averaged from 0.9cm (3/8 in.) to 1.8cm (3/4 in.). Between 233 and 238, averages were 0.9cm (3/8 in.) to 1.5cm (5/8 in.) and on the fabric protected pavement 0.3cm (1/8 in.) to 0.9cm (3/8in.). These are minimal, a credit to the design procedure used. The maximum elevation variation occurred in the sections without the fabric, -3.6cm and +2.4cm (Table 3). The PVR calculations predicted movements have not occurred.

TABLE 3 - McMULLEN
DIFFERENCES IN ELEVATIONS 1978-1983 (in cm's)

STATION	SOUTHBOUND		NORTHBOUND	
	Gutter Outer Edge	Median Crown Point	Median Crown Point	Gutter Outer Edge
South Control AVERAGES:	1.8	1.2	0.9	0.9
Fabric AVERAGES:	0.9	0.6	0.3	0.3
North Control AVERAGES:	1.5	0.9	0.9	0.9

MAXIMUM CHANGES:

South Control	+2.4	-2.7
Fabric	+1.8	-1.5
North Control	+0.9	-3.6

Dynaflect testing was conducted in August 1973 before construction, in January 1981 and March 1982. A static tire loading test device is used. Five geophone readings are taken, a curve plotted, stiffness coefficients, spreadability and maximum deflections calculated. The stiffness coefficient separates subgrade values from the pavement's total section with the higher number having the greater strength. The preconstruction subgrade values show a uniformity. In March 1982 the geomembrane section stiffness was slightly lower than the adjacent ones. Pavement values for the fabric protected section are slightly higher (Table 4). This may be related to its PVR, its greater expansive capabilities, or the holding of moisture under the material.

TABLE 4 - McMULLEN - STIFFNESS COEFFICIENTS

	Subgrade		Pavement	
Date:	8-73*	3-82	8-73*	3-82
North Control	0.20	0.20	0.60	0.95
Fabric	0.20	0.18	0.72	1.09
South Control	0.21	0.20	0.65	0.98

* Prior to Reconstruction

Spreadability is determined by summing the five geophone readings and dividing it by five times the geophone number one reading. The higher the value the stiffer the pavement structure. In the January 1981 and March 1982 tests, the fabric sections in all cases showed the greatest strength (Table 5).

TABLE 5 - McMULLEN DRIVE - SPREADABILITY INDECES

Dates:	1-81	3-82	1-81	3-82	1-81	3-82
	OUTSIDE		CENTER		INSIDE	
Southbound Lanes						
Control	81	74	79	79	76	80
Fabric	86	78	83	83	83	83
Control	81	75	79	79	74	80
Northbound Lanes						
Control	80	76	79	80	79	79
Fabric	84	77	84	82	84	82
Control	80	73	82	81	80	81

Maximum deflection is calculated by multiplying geophone number one values by 22.5 to simulate the movement produced by an 18 kip load and reflects subgrade value properties. These tests in January 1981 and March 1982 indicated the subgrade under the geomembrane had greater deflection in five of six cases and was possibly weaker (Table 6).

TABLE 6 - McMULLEN - MAXIMUM DEFLECTIONS

Date:	1-81	3-82	1-81	3-82	1-81	3-82
	OUTSIDE		CENTER		INSIDE	
Southbound Lanes						
Control	.015	.023	.017	.017	.022	.016
Fabric	.018	.022	.015	.017	.023	.019
Control	.012	.016	.013	.014	.016	.013
Northbound Lanes						
Control	.018	.020	015	.015	013	.015
Fabric	.025	.026	019	.020	.019	.021
Control	.020	.023	015	.017	.017	.017

The outside pavement lanes have developed a weaker pavement structure than the interior ones. This may be expected since traffic is considered to be concentrated on the outside lanes, and it is also more susceptible to moisture change.

The higher average stiffness coefficients on the total geomembrane section's pavement structure and its higher spreadability indeces are significant since it was placed on the most active, and weaker, subgrade section.

This tends to substantiate the previously reported effectiveness of fabrics as a strengthener of total pavement structure. (6,7,8)

LOOP 410 - DEEP VERTICAL FABRIC MOISTURE BARRIERS

The first test of the deep vertical fabric as a moisture barrier was located on Interstate Highway Loop 410 in the Valley Hi Drive interchange area in southwestern San Antonio. Atterberg limits for these Houston clays indicated liquid limits from 50 to 79, plasticity indeces from 28 to 48 and moisture contents from 7.7% to 34.1%.

This 410 section, built in 1960, is a four lane divided highway with 0.41m (16 in.) of foundation course, 0.20m (8 in.)of flexible base, 0.08m (3 in.) of Type A and 0.05m (2 in.) of Type C hot mix asphaltic concrete (HMAC) pavement. Both the north and southbound main lanes have two 3.66m (12 ft) driving lanes, 3.05m (10 ft.) outside and 1.22m (4 ft.) inside shoulders separated by 13.41m (44 ft.) grassed median. Access roads paralled the main lanes. Since its construction swelling soils activity has been reflected in this area with repeated asphaltic concrete level ups, followed by more pavement distortions, more level ups, "heater planner" work, irregularity in the curb profiles and an attempt with pressure injection of lime.

The rehabilitation project, low bid at $3.8 million in June 1978, was 24.5km (15.2 mi.) long. It included an asphalt seal coat 0.03m (1½ in.) of Type C hot mix asphaltic concrete, as a level up, to be followed by 0.02m (3/4 in.) of Type D HMAC. The level up actually placed varied from 0.03m (1 in.) to 0.31m (12 in.). A 0.8km (½ mi.) test section for a fabric moisture barrier was included. A spun bonded polypropylene, ethyl vinyl acetate coated, 15.5 mils thick, Dupont Typar Coated EVA T063 fabric was placed along the northbound lane. It was placed 2.44m (8 ft.) deep at the edges of the outside and inside shoulders. The depth was chosen to protect the clays zone of activity determined by earlier testing (9).

Installation included tacking 0.61m (2 ft.) of the fabric with asphalt emulsion to the paved shoulder. The Typar rolls were 3m (117 in.) wide, a 0.27m (10 3/4 in.) diameter and a 0.08m (3 in.) core. A sand backfill was selected as being easily placed and compacted in a narrow trench. The fabric was placed between February and March 1979. A backhoe with a 0.61m (2 ft.) bucket was used for excavation. A front end loader loaded the excavated material on dump trucks for a waste haul and placed the sand backfill. A sliding steel shoring using 0.001m (1/4 in.) plate which held the fabric roll vertically, pulled by the backhoe, solved the problem of occasional trench slides. Placement averaged 106.7m to 121.9m (350 to 400 ft.) per day; Bid price was $65/m ($20/ft.).

Moisture sensors were placed following calibration at Texas Transportation Institute. Readings were taken for several years. Calibration problems resulting in feelings of unreliability in determining moisture contents and failure of the instruments have caused cessation of readings.

In the first profilometer readings, the northbound lane with the geomembrane had the highest serviceability indeces, 4.16 and 4.11, for the outside and inside lanes compared to southbound lanes, 4.13 and 4.02. This pattern has continued through current testing where northbound geomembrane lanes had 3.57 and 3.55 for the outside and inside lanes while the southbound S.I.'s were 3.28 and 2.99 (Table 7). The protected northbound lane had a "smoother" ride. When the area received a level up in 1981, the southbound lane received twice the tonnage as the fabric protected northbound section, 181,436kg (200 tons) compared to 90,718kg (100 tons).

TABLE 7 - LOOP 410 - SERVICEABILITY INDICES

	6-79	8-80	12-80	3-81[1]	7-81[2]	12-81	2-83
NORTHBOUND LANE (Fabric Section)							
Outside	4.16	3.83	3.55	3.72	3.74	3.76	3.57
Inside	4.11	3.83	3.57	3.76	3.61	3.66	3.55
SOUTHBOUND LANE (Control Section)							
Outside	4.13	3.39	3.28	3.38	3.43	3.57	3.28
Inside	4.02	3.30	3.15	3.17	3.29	3.56	2.99

1 Rotomilled 4-81
2 Project leveled up 8-81.

Photologging began in August 1980 and has continued annually. Reflecting the surface cracking, the southbound lane has consistantly had the higher percentage indicating more openings. In August 1980 it was 0.24 and 0.28 compared to the northbound lanes 0.07 and 0.01. This pattern continued through 1981 and 1982. The fabric protected northbound lane always had less cracking than the southbound lane. In 1983 it was 0.19 and 0.37 to 0.45 and 0.75 respectively (Table 8).

TABLE 8 - LOOP 410 - PHOTOLOGGING

	8-80	12-80	6-81	1-82	8-82	1-83
NORTHBOUND LANES						
Outside	0.07	0.08	-0-	0.04	-0-	0.19
Inside	0.01	0.24	0.08	0.34	0.03	0.37
SOUTHBOUND LANES						
Outside	0.28	0.62	0.11	0.87	0.29	0.75
Inside	0.24	1.01	0.10	0.78	0.51	0.45

INTERSTATE 37 - ANOTHER DEEP VERTICAL PLACEMENT

The Loop 410 results created a guarded optimism and a more extensive deep vertical moisture barrier placement was completed on San Antonio's Interstate Highway 37. The project area has been experiencing a substantial swelling clay movement since its construction in 1968. The movements have been reflected in severe distortions in the riding surface, in the median guardrail, the curbs and some slope slides. Also in the Houston Black series, these clays had plasticity indeces of 54; liquid limits 86, and are very active. I.H. 37 is an 8 lane divided highway, with a sodded median from 8.54m to 10.97m (28 to 36 ft.) wide with a 0.91m (3 ft.) median ditch and steel barrier guardrail. Each travelway lane is 3.66m (12 ft.) wide with a 3.05m (10 ft.) outside and a 1.83m (6 ft.) inside shoulder. The main lanes were constructed on 0.15m (6 in.) of lime stabilized subgrade, 0.20m (8 in.) of cement stabilized base and 0.20m (8 in.) of concrete pavement. The main lanes vertical alignment varies from a 6.06m (20 ft.) embankment to a 6.71m (22 ft.) excavation below natural ground. The active clays have caused an estimated $50,000 a year in maintenance expenditures for asphalt level ups, heater planner work and construction of retaining walls in this 3.22km (2.1 mi.) area. The present rehabilitation contract includes a rubberized asphalt seal coat, asphaltic concrete level ups, asphaltic concrete finish course, reconstruction of the median to provide positive drainage from a built up section, and a deep vertical fabric moisture barrier, Dupont Typar T063 EVA geomembrane placed 2.4m (8 ft.) deep in trenches outside the shoulder of the north and southbound lanes. A 0.15m (6 in.) perforated underdrain pipe is placed in some areas in the bottom in the ditch line, outside the fabric. The fabric trench is backfilled with gravel and the top 0.91m (3 ft.) cement stabilized. The project contract called for 19,715sqm, (23,753 sq.yd.) of fabric placement. With the low bidder's price of $69 per linear m. this compared not unfavorably with the bid for this work on Loop 410.

The same subcontractor from the 410 fabric test used a trenching machine with a conveyor attachment discharging the excavated material into a dump truck. The trenching machine's boom was fitted with a sliding shoring to hold the Typar roll vertically in the excavation and unrolled it as the machine progressed. A portable paving machine batched the one sack cement stabilized base that topped the gravel backfill in the fabric's trench. The vertical barrier was placed between February 4 and May 21, 1980. The best day of placing fabric was 147.8m (485 ft.); their averages,106.7m (350 ft.) a day.

Serviceability indeces show prior to the rehabilitation, the adjacent control sections had the higher values, the smoother ride. Following the fabric placement and repaving in 1980 through 1983, the protected lanes had the higher S.I.'s with little deterioration in values. In 1980, the fabric S.I.'s averaged 3.68 and 3.81, in 1983, 3.61 and 3.68 compared to the control values of 3.09,3.30, 2.97, 3.18, 3.19, 3.09, 3.29 and 3.01 respectively (Table 9).

TABLE 9 - IH 37 - SERVICEABILITY INDECES

Southbound Lane	11-79	12-80	7-81	12-81	6-82	2-83
Control Section	3.22	2.97	2.79	2.99	3.04	3.09
Fabric Section	2.79	3.68	3.72	3.67	3.69	3.61
Control Section	3.33	3.18	3.20	3.22	3.22	3.19
Northbound Lane						
North Control	3.15	3.09	3.02	2.93	3.12	3.01
Fabric Section	2.92	3.81	3.84	3.75	3.80	3.68
South Control	3.24	3.30	3.30	3.28	3.33	3.29

November 1979 testing prior to rehabilitation work. December 1980 and later tests after work completed.

Photologging revealed little surface cracking. A maximum reading of 0.15% occurred in the fabric section, in June 1981. Subsequent readings in 1982 and 1983 showed no cracks appearing. A 0.07% cracking in the control non fabric protected section was the only sign of pavement distress in 1983 tests (Table 10).

Moisture sensor readings following calibration at Texas Transportation Institute showed greater reliability than previous tests. Measurements of matrix potential show consistant readings inside the fabric protected subgrade while extreme changes have taken place outside the geomembranes. These matrix potential changes are considered indicative of moisture changes. While four of the original ten sensors are not operational, the others continue to function. (10)

TABLE 10 - I.H. 37 - PHOTOLOGGING

Northbound	6/81	4/82	8/82	1/83
Outside Lane				
Control	0.04%	0.00%	0.00%	0.07%
Fabric #1	0.03%	0.00%	0.06%	0.00%
Fabric #2	0.00%	0.03%	0.00%	0.00%
Inside Lane				
Control	0.00%	0.00%	0.00%	0.00%
Fabric #1	0.01%	0.00%	0.00%	0.00%
Fabric #2	0.15%	0.00%	0.00%	0.00%

Fabric sections 1750 ft. long sampled. Sections with 0% cracking not shown.

CONCLUSION

On all three projects the geomembrane protected sections indicated a smoother ride with generally less surface pavement cracking. On General McMullen the horizontal fabric section provided the smoother ride, with less pavement cracking, and demonstrated a greater pavement system strength than the unprotected adjacent control sections. On Loop 410 a good comparative section since adjacent roadways were used, the fabric protected northbound lane had the higher serviceability indeces, less surface cracking and used one half the asphalt for surface repair than the adjacent unprotected southbound lane. The I.H. 37 rehabilitation project indicated a similar pattern. Little loss of serviceability indeces, little surface cracking, reduced moisture variations on the protected lanes that previously required $50,000 annually for repairs. Positive drainage from the median and the rubberized asphalt seal are also significant factors. The geomembranes seem to have contributed to the reduction in damages due to expansive soils. Two additional recently awarded contracts include deep vertical geomembrane barriers, polypropylenes with polypropylene coatings by another manufacturer. In future work combining horizontal and deep vertical barriers of geomembranes should be considered in expansive subgrade areas.

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INTERNATIONAL CONFERENCE ON GEOMEMBRANES

LIST OF SYMBOLS

1. GENERAL SYMBOLS

1.1 Dimension symbols

Symbols used for the dimensions are:

L:	length
M:	mass
T:	time
–:	dimensionless

1.2 Unit Symbols

m	meter
m^2	square meter
m^3	cubic meter
mm	millimeter
µm	micron
g	gram
mg	milligram
kg	kilogram
s	second
N	newton
kN	kilonewton
Pa	pascal
kPa	kilopascal
MPa	megapascal
J	joule
tex	tex
L	liter
°	degree
%	percent
–	pure number

The following relationships exist:

1 Pa = 1 N/m^2
1 tex = 1 mg/m
1 J = 1 mN

1.3 Geometry and kinematics

L	L	(m)	length
B	L	(m)	breadth
H	L	(m)	height, thickness
D	L	(m)	depth
z	L	(m)	vertical coordinate
d	L	(m)	diameter
A	L^2	(m^2)	area
V	L^3	(m^3)	volume
t	T	(s)	time
v	$L\,T^{-1}$	(m/s)	velocity
g	$L\,T^{-2}$	(m/s^2)	acceleration due to gravity ($g = 9.81\ m/s^2$)

1.4 Hydraulic parameters and properties of FLUIDS

h	L	(m)	hydraulic head or potential
			sum of pressure height (u/γ_w) and geometrical height (z) above a given reference level
q	L^3T^{-1}	(m^3/s)	rate of discharge
			volume of water seeping through a given area per unit of time
v	$L\,T^{-1}$	(m/s)	discharge velocity
			rate of discharge per total unit area perpendicular to direction of flow
i	–	(–)	hydraulic gradient
			loss of hydraulic head per unit length in direction of flow
j	$ML^{-2}T^{-2}$	(kN/m^3)	seepage force per unit volume
			force per unit volume of a porous medium generated by action of the seeping fluid upon the solid elements of the porous medium ($j = i\gamma_w$)
ρ_w	ML^{-3}	(kg/m^3)	density of water
γ_w	$ML^{-2}T^{-2}$	(kN/m^3)	unit weight of water
η_w	$ML^{-1}T^{-1}$	(kg/ms)	dynamic viscosity of water

NOTE: Instead of w, use any other appropriate subscript of other fluids (e.g.: η_a, dynamic viscosity of air)

2. Hydraulic properties of a net, geogrid, or geotextile associated with a geomembrane

k_n	LT^{-1}	(m/s)	coefficient of normal permeability of a geotextile
k_p	LT^{-1}	(m/s)	coefficient of permeability in the plane of a geotextile
ψ	T^{-1}	(s^{-1})	permittivity of a geotextile ($\psi = K_n/H_g$)
θ	L^2T^{-1}	(m^2/s)	transmissivity of a geotextile ($\theta = k_p/H_g$)

3. Physical/mechanical properties of a geomembrane

μ		(kg/m^2;g/m^2)	mass per unit area
α_ε	MT^{-2}	(kN/m)	force per unit width of the geomembrane
α_f	MT^{-2}	(kN/m)	force per unit width of the geomembrane at failure
α_y		(kN/m)	force per unit width at yield
ε		(%)	strain (also called elongation)
ε_f		(%)	strain at failure (elongation at failure)
ε_y		(%)	strain at yield (elongation at yield)
J	MT^{-2}	(kN/m)	"modulus" of the geomembrane

NOTE: The "modulus" of a geomembrane is defined as a force per unit width while modulus is usually defined as a force per unit area.

J_i	MT^{-2}	(kN/m)	initial (tangent) "modulus" of the geomembrane
$J_{t\varepsilon}$	MT^{-2}	(kN/m)	tangent "modulus" of the geomembrane at elongation (e.g. J_{t30} is the tangent "modulus" of the geomembrane at 30% elongation)
$J_{sec\ \varepsilon}$	MT^{-2}	(kN/M)	secant "modulus" of the geomembrane between 0 and elongation (e.g. J_{sec30} is the secant "modulus" of geomembrane between 0 and 30% elongation)
F_G	MLT^{-2}	(N,kN)	breaking force of geomembrane as measured in grab test
F_p	MLT^{-2}	(N,kN)	breaking force of geomembrane in a puncture test (to be defined)
F_T	MLT^{-2}	(N,kN)	breaking force of geomembrane in a tear test (to be defined)
P_B	$ML^{-1}T^{-2}$	(kPa,MPa)	bursting pressure of a geomembrane

International Conference on Geomembranes

Definition of Terms

Geomembrane:

Synthetic membranes, polymeric membranes, flexible membrane liners, plastic liners, and impervious sheets are a few examples of the many names given to these relatively new materials. Although many users of these materials often prefer to use trade names, this practice is deemed inappropriate because it creates considerable confusion.

Geomembrane is the generic term proposed to identify these liner and barrier materials. Geomembranes are impermeable membrane liners and barriers used in civil engineering for geotechnical projects. They can be either sprayed on a surface or prefabricated and transported to the construction site. Sprayed-on geomembranes are composed predominantly of asphalt. They are either sprayed directly on a surface (earth, concrete, etc.) or onto a geotextile. Prefabricated geomembranes are usually composed of synthetic polymers, elastomers (rubbers), or plastomers (plastics); some are reinforced with a fabric. There are also prefabricated asphaltic geomembranes.

Classification of Geomembranes[1]

Geomembranes can be classified according to production process and reinforcement:

1. **Made in situ, non-reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material directly onto the surface to be lined (earth, concrete, etc.). The non-reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) non-reinforced geomembranes." Typical materials used are based on asphalt, asphalt-elastomer compound, or polymers such as polyurethane. Due to the spray application, the final thickness of such geomembranes is not easy to control and may vary significantly from one location to another. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

2. **Made in situ, reinforced geomembranes** are made by spraying or otherwise placing a hot or cold viscous material onto a fabric. The reinforced geomembranes made by spraying are called "sprayed-on (or spray-applied, or sprayed in situ) reinforced geomembranes." Typical materials used are the same as for the made in situ non-reinforced geomembranes described above. Typical fabrics used are the needle-punched nonwoven geotextiles because they can absorb viscous materials. As discussed above, the final thickness of such geomembranes is not easy to control. Typically, required thicknesses range between 3 and 7.5 mm (120 and 300 mils).

3. **Manufactured, non-reinforced geomembranes** are made in a plant by extrusion or calendering of a polymeric compound, without any fabric reinforcement, or by spreadying a polymer on a sheet of paper removed at the end of the manufacturing process. Typical thicknesses range from 0.25 to 4 mm (10 to 160 mils) for geomembranes made by extrusion and 0.25 to 2 mm (10 to 80 mils) for geomembranes made by calendering. Typical roll width for geomembranes made by extrusion is 5 to 10 m (16 to 33 ft), although some are narrower. Typical roll width for geomembranes made by calendering is 1.5 m (5 ft), with some manufacturers producing 1.8 to 2.4 m (6 to 8 ft) wide rolls.

4. **Manufactured, reinforced geomembranes** are made in a plant, usually by spread coating or calendering. In spread-coated geomembranes, the reinforcing fabric (woven or nonwoven) is impregnated and coated on one or both sides with the compound, either polymeric or asphaltic. In calendered reinforced geomembranes, the reinforcing fabric is usually a scrim. Calendered geomembranes are always made with polymeric compounds and are usually made up of three plies: compound/scrim/compound. Sometimes they are made of five plies: compound/scrim/compound/scrim/compound. Geomembranes with additional plies can be made on a custom basis. Typical thicknesses of asphaltic spread-coated geomembranes are 3 to 10 mm (⅛ to ⅜ inch). Typical thicknesses for polymeric spread-coated and three-ply calendered geomembranes are 0.75 to 1.5 mm (30 to 60 mils). Typical thicknesses for five-ply calendered geomembranes are 1 to 1.5 mm (40 to 60 mils).

5. **Manufactured, reinforced geomembranes laminated with a fabric** are made by calendering a manufactured geomembrane (usually a non-reinforced

geomembrane previously made by calendering or extrusion) with a fabric (usually a nonwoven) which remains apparent on one face of the final product.

Classification of Geomembrane Polymers (National Sanitation Foundation):

1. *Thermoplastics:* Polyvinyl Chloride (PVC); Oil Resistant PVC (PVC-OR); Thermoplastic Nitrile-PVC (TN-PVC); Ethylene Interpolymer Alloy (EIA);

2. *Cristalline Thermoplastics:* Low Density Polyethylene (LDPE); High Density Polyethylene (HDPE); High Density Polyethylene-Alloy (HDPE-A); Polypropylene; Elasticized Polyolefin;

3. *Thermoplastic Elastomers:* Chlorinated Polyethylen (CPE); Chlorinated Polyethylene-Alloy (CPE-A); Chlorosulfonated Polyethylene (CSPE), also commonly referred to as ''Hypalon;'' Thermoplastic Ethylene-Propylene Diene Monomer (T-EPDM);

4. *Elastomers:* Isoprene—Isobutylene Rubber (IIR), also commonly referred to as Butyl Rubber; Ethylene-Propylene Diene Monomer (EPDM); Polychloroprene (CR), also commonly referred to as ''Neoprene;'' Epichlorohydrin Rubber (CO).

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