Geosynthetics '99

"Specifying Geosynthetics and Developing Design Details"

GEOSYNTHETICS 99 CONFERENCE
Boston, Massachusetts USA

Conference Proceedings

April 28-30, 1999

Organized by:

IFAI
Industrial Fabrics Association International

NAGS
North American Geosynthetics Society

GMA
Geosynthetic Materials Association

under the auspices of
IGS
International Geosynthetics Society

Table of Contents

LESSONS LEARNED FROM FAILURES ASSOCIATED WITH GEOSYNTHETICS

J.P. GIROUD
CHAIRMAN EMERITUS, GEOSYNTEC CONSULTANTS
PAST PRESIDENT, INTERNATIONAL GEOSYNTHETICS SOCIETY

ABSTRACT

Based on a review of approximately 100 case histories, this paper analyzes systematically the modes of failure of structures incorporating geosynthetics and reviews the design and field situations that lead to failures. Numerous examples and case histories are presented, and lessons learned from the failures are summarized.

1 INTRODUCTION

Geotechnical engineers who do not learn from mistakes made by others will learn from their own mistakes. This should encourage geotechnical engineers to read this paper.

1.1 Learning From Failures

Geotechnical engineering is an art as much as a science, as many like to say. This statement is incorrect and it may mislead those who are learning about geotechnical engineering. Geotechnical engineering is a science, it is not an art. It is a science because all phenomena of geotechnical engineering can be explained rationally. It is not an art because, in geotechnical engineering, there is no room for personal emotions and abstract imagination. One may object that the word "art", being used in expressions such as "the art of building", does apply to geotechnical engineering. In these expressions, the word "art" designates *methods and skills*, generally *derived from practice*. Geotechnical engineering, being an applied science, certainly includes activities that require "methods and skills derived from practice" (e.g. the art of conducting field investigations or, even, the art of writing papers). However, geotechnical engineering itself is a science, an applied science, not an art.

As a science, geotechnical engineering requires reliable tools. Common sense, which is often invoked by geotechnical engineers, is not a reliable tool, because it is a random collection of beliefs, many of them being bad habits only justified by tradition. As the origin of the beliefs packaged under the label "common sense" is usually unknown, there is no way to distinguish between the good and the bad. Therefore, common sense cannot be used as a basis for rational decisions in a scientific discipline.

The fact that, in a scientific discipline, all phenomena can be explained rationally on the basis of first principles does not mean that all knowledge must result from logical deduction. In fact, a large fraction of the present scientific knowledge — and this applies to all disciplines — was generated from experience, often by chance. Rational explanations of the phenomena were developed eventually. This is particularly true for geotechnical engineering, a discipline where the complexity of materials, mechanisms and boundary conditions makes it difficult to predict phenomena only by pure logical deduction. This is also true for geosynthetics, because, in addition to the constraints inherent to geotechnical engineering, there is the fact that the use of geosynthetics is relatively new. As a result, the body of rational knowledge is still under development while the variety of uses and users creates a wealth of experience from which additional knowledge can be tapped.

It is clear from the above discussion that no opportunity should be missed to learn from experience. However, there is a great difference between experience and learning from experience. The only way to learn from experience is to analyze available data and to incorporate the results of the analyses into an organized body of knowledge. This is particularly true for learning from failures, which constitute the ultimate level of experience. In this paper, numerous case histories are used to show how lessons can be learned from a rational analysis of failures.

Based on the above discussion, it is important to know what a failure is.

1.2 Definition of Failure

Asking the question, "What is a failure?", regarding geotechnical structures, often attracts a confusing answer based on "common sense", the magic phrase used every time it appears difficult to develop a rational approach. Indeed, it is not easy to rationally define what a failure is, as seen below.

A failure in a geotechnical structure can affect the entire structure (e.g. a road embankment), a system (e.g. a cover system on a landfill), or a component (e.g. a geosynthetic). Several definitions can be considered for a failure.

A first tentative definition, which is often mentioned, would be:

> *A structure, system, or component fails if it does not perform its intended function.*

Certainly, a structure, system, or component must perform its intended function, but a definition that only contains this requirement is not complete. This definition may be too lax or may be excessive depending on the interpretation of the word "function". For example, according to the above definition, a retaining structure that exhibits a very large deformation, but still retains the soil, is not considered a failure regardless of the consequences of the large deformation if the function of the retaining structure is only understood to be "to retain the soil". Also, according to the above definition, a geomembrane liner with a very small defect causing an inconsequential leak would be considered a failure if it is understood that the intended function of a liner is to act as a fluid barrier. At this point, it is important to note that there is a difference between the function of a structure and the function of the geosynthetic in the structure. Thus, while the function of the geomembrane liner in a pond is to act as a fluid barrier, the function of the pond is to contain a liquid. This distinction is illustrated by the case of a small pond where the geomembrane liner is entirely uplifted by gas (a real case); in this case, the pond fails to perform its function of containing liquid while the geomembrane liner does perform its function of acting as a fluid barrier. The function of the structure should not only be clearly defined, it should also be quantified; for example, in the case of a geomembrane-lined pond, the volume of liquid to be contained should be specified. However, this may still not be sufficient, because a single geomembrane bubble in the case of a large pond may not significantly affect the volume of liquid contained, but may affect the long-term performance of the geomembrane (by exposing the geomembrane to sunlight, wind, etc.) and may hamper the operation of the pond. Clearly a definition of failure based on the function of the structure is too vague to be adequate. Based on a comment made above, the definition of failure should include a number of quantified requirements, i.e. performance criteria. This leads to the second tentative definition:

> *A structure, system, or component fails if it does not meet its performance criteria.*

This definition is better than the first definition because it includes performance criteria, but it is flawed because it implies that performance criteria were set for the considered structure, system, or component, which is not always the case. (In other words, this definition opens the door to the absurd situation where there cannot be a failure because no criteria were set.) Also, this definition implies that the performance criteria, if any, are complete and adequate. For example, simplistic performance criteria such as "no settlement" or "zero leakage" are not adequate because they cannot be met and, therefore, any behavior is a failure with respect to such criteria.

Clearly, a better definition is needed. Combining the two above tentative definitions in a phrase such as "*if it does not perform its intended function and/or does not meet its performance criteria*" does not solve the problems illustrated by the examples presented above.

Finally, the proposed definition could be:

> *A structure, system, or component fails if it does not meet complete and adequate performance criteria.*

This definition is technically correct because "*complete and adequate performance criteria*" can be expected to define and quantify, completely and adequately, the intended function of the structure, system or component. A potential drawback of the above definition is that the adjectives "*complete and adequate*" may be subject to interpretation and discussions. However, it should be possible to develop guidance regarding what "*complete and adequate performance criteria*" are. Tentatively, the following guidance is proposed. To be complete, the criteria should address the three following potential modes of failure: failure to perform the function of the structure, system, or component; disruption of, or nuisance to, operation or use of the structure, system, or component; and threat to the future performance of the structure, system, or component. These three potential modes of failure are discussed below:

- *Failure to Perform the Function of the Structure, System, or Component.* As stated after the first tentative definition, it is clear that a structure, system, or component must perform its intended function. Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the ability of the structure, system, or component to perform its intended function. In the case of a pond, examples of such requirements are: the volume of liquid that the pond must contain and the maximum allowable leakage rate. Examples of failures to meet these requirements would be: a geomembrane liner uplifted by gas to the extent that the required volume of liquid cannot be contained; a leak that exceeds the maximum allowable leakage rate; and a very large leak that both exceeds the maximum allowable leakage rate and prevents the pond from containing the the required volume of liquid.

- *Disruption of, or Nuisance to, Operation or Use of the Structure, System, or Component.* Every structure, system, or component is operated or used. Therefore, there are disruptions of, or nuisances to, the operation or use of the structure, system, or component that cannot be tolerated by the operator or user. This kind of failure is often referred to as "serviceability failure". Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the disruptions of, or nuisances to, the operation or use of the structure, system, or component that cannot be tolerated. In the case of a pond, an example of such requirements is that boats can navigate in all parts of the pond, which means that the localized uplift of the geomembrane liner by gas cannot be tolerated even if it does not affect the ability of the pond to perform its function which is to contain a certain volume of liquid.

- *Threat to the Future Peformance of the Structure, System, or Component.* A structure, system, or component must perform its function and be operated or used for a certain period of time. Therefore, to be *complete and adequate*, performance criteria should include qualitative and quantitative requirements describing the ability of the structure, system, or component to perform its function and be operated or used during a certain period of time usually referred to as the design life. Criteria can even include trends (such as change in some geomembrane characteristic that indicates degradation, or monitoring of the inclination of a reinforced-soil wall facing) or symptoms (such as water seeping through the downstream face of a dam) that may help predict future failure, or even imminent failure (if the trends and/or symptoms indicate rapid or even accelerating material and/or structure degradation). Failures that result from not meeting the requirements related to the future performance of the structure, system, or component can be referred to as "durability failures". In the case of a pond, a localized uplift of the geomembrane liner forming a "bubble" may not prevent the pond from performing its function (which is to contain a certain volume of liquid) and may not hamper the operation and use of the pond. Therefore, the first two types of criteria are met. However, the bubble exposes the geomembrane to sunlight and vandals, which may decrease the ability of the geomembrane liner to perform its function during the entire design life of the pond. Therefore, the performance criteria should include some language treating the development of a geomembrane bubble as a symptom that is not acceptable and requires immediate action because it indicates the beginning of a mechanism leading to failure.

The boundaries between the three modes of failures are not totally rigid and some criteria may be at the limit between two modes. For example, the deformation of a reinforced soil wall may be only a nuisance to the user if it affects the appearance of the wall face or it is a failure to perform the function if, due to the deformation of the wall, a foundation that was to be built on the retained soil cannot be built. (The limit may even evolve with time: a wall with a face tilting forward may only be a "nuisance to the use of the structure", however, as the tilting continues to increase, it may become a warning of imminent collapse.) However, the above guidance makes it possible to establish a list of criteria that is *complete*, which is essential. In addition to be *complete*, the criteria should be *adequate*. Adequate criteria are criteria that are rationally quantified in a way that reflects the performance of the structure and the needs of its operators and users.

It should be noted that the three modes of failure mentioned above are different from the two modes often mentioned, structural failure and serviceability failure. The terminology "*structural failure and* *serviceability failure*" comes from the designers of structures that may collapse when poorly designed and/or constructed (e.g. reinforced-soil structures with a vertical face). This terminology is not applicable to the many types of structures that do not collapse. Clearly, instead of referring to "*structural failure and serviceability failure*", it is more general and more correct to refer to "*functional failure, serviceability failure, and durability failure*", as explained above. An advantage of the proposed definition of failure is that it makes it possible to evaluate rationally designs and specifications and to identify those that are based on incomplete or inadequate performance criteria.

According to the proposed definition, a defect is not necessarily a failure. Whether a defect is a failure depends on its acceptability. A large hole in a geomembrane is clearly a defect but it may not cause a failure if the resulting leakage is acceptable and is properly handled by an adequate drainage system. In contrast, a small hole in a geomembrane is a failure if it causes unacceptable pollution of the soil or ground water. However, it may not be appropriate to attribute this failure to the geomembrane. It is likely that, in such a case, the unacceptable pollution is due to a design mistake because small defects are known to happen in geomembranes and a different design should have been considered (e.g. a double liner instead of a single liner).

The types of failures that are covered by the proposed definition generally occur when the structure is in service. However, certain types of failures may also occur during construction. For example, the rotational instability of a road embankment during construction is a failure because it seriously compromises the construction of the road. In contrast, construction problems which are solved during construction (e.g. a geomembrane torn during construction and repaired) are not failures because they do not prevent the structure, system, or component from meeting performance criteria and from being constructed in time.

The consequences of a failure depend on the function of the structure, system, or component and may vary widely. Thus, after a failure, a structure, system or component is repaired, reconstructed or abandoned. Regardless of the magnitude of the consequences, all failures result in an interruption of the operation or use of the structure; from this viewpoint, an imminent failure (i.e. a failure defined above as being characterized by trends and/or symptoms indicating rapid or even accelerating material and/or structure degradation) is equivalent to an actual failure.

1.3 Functions of Structures and Functions of Geosynthetics

Based on the above discussion on the definition of failure, it is necessary to have a good understanding of the functions performed by structures, systems, or components to understand and avoid failures. It is important to note that functions performed by structures are different from functions performed by geosynthetics. For example: the function of a geosynthetic-reinforced retaining structure is to retain the soil, whereas the function of the geosynthetic is to reinforce the soil; and the function of a drainage system is to drain the soil whereas the function of the geotextile filter is to act as a filter for the drainage system. The function of the structure is defined in relation to the soil mass or the environment, whereas the function of the geosynthetic is defined in relation to the structure.

In any case, it is important to understand the functions of geosynthetics to avoid and investigate failures. It is also necessary to properly identify the properties required for a geosynthetic to perform a given function. Furthermore, it is necessary to understand the requirements for performing each function, such as the geosynthetic and soil movements associated with the performance of each function. Indeed, it will be seen in this paper that these geosynthetic and soil movements may have a significant impact on the performance of structures incorporating geosynthetics. Information on functions of geosynthetics can be found in several publications on geosynthetics (Giroud, 1980; Giroud et al., 1985; Holtz et al., 1997; Koerner, 1998).

1.4 Failures Associated with Geosynthetics

To prepare this paper, approximately 100 documented case histories were reviewed. This paper includes three main sections: Section 2, *Modes of Failures Associated with Geosynthetics*, Section 3, *Situations Leading to Failures* and Section 4, *Learning Lessons and Lessons Learned*. Throughout the paper, special emphasis is placed on lessons learned from case histories.

2 MODES OF FAILURES ASSOCIATED WITH GEOSYNTHETICS

2.1 Overview of Modes of Failure

2.1.1 Failure Modes that Are Already Well Known in Geotechnical Engineering

Some of the functions performed by geosynthetics are identical to functions performed by soil materials. For example, a geotextile filter and a sand filter perform the same function. Geosynthetics and soils that perform the same function are likely to fail according to similar modes if they do not perform the required function. For example, geotextile filters may fail by clogging or by allowing piping, as sand filters do.

Even if a geosynthetic does not perform a function that could be performed by a soil (e.g. if the geosynthetic performs the reinforcement function), the mode of failure of a structure incorporating geosynthetics may be similar to that of a soil structure. This is the case when the failure mode is mostly governed by soil behavior. Thus, a number of failure modes that are well known in geotechnical engineering are applicable to geosynthetic engineering.

2.1.2 Failure Modes that Are Specific to Geosynthetics

In addition to the failure modes that are well known in geotechnical engineering and are applicable to geosynthetic engineering, there are new failure modes that are specific to geosynthetics. Many of these new failure modes are related to the fact that most geosynthetics have two characteristics that make them different from soils: they are two-dimensional and they are polymeric. It should be noted that this characterization of geosynthetics does not include geofoam blocks (because they are three-dimensional), but does include geosynthetics made with natural fibers (because these fibers are typically made with natural polymers).

Also, geosynthetics are in contact with adjacent materials (soil, waste, other geosynthetics, etc.); as a result, some of the failure modes that are specific to geosynthetics are linked to interaction mechanisms. In other words, geosynthetics are incorporated in a three-dimensional structure and many of the failure modes of structures incorporating geosynthetics result from the interaction between the (generally) two-dimensional geosynthetic and the three-dimensional structure that surrounds it.

2.1.3 Organization of Section 2

The failure modes that are well known in geotechnical engineering, and are applicable to geosynthetic engineering, are discussed in Section 2.2, and the failure modes that are specific to geosynthetics are discussed in Sections 2.3, 2.4 and 2.5. A conclusion on the failure modes associated with geosynthetics is presented in Section 2.6.

2.2 Failure Modes Common to Soils and Structures Incorporating Geosynthetics

2.2.1 Overview of Failure Modes Common to Soils and Structures Incorporating Geosynthetics

As indicated in Section 2.1.1, there are failure modes that are well known in geotechnical engineering and are applicable to geosynthetic engineering. This happens because: (i) some of the functions performed by geosynthetics are identical to functions performed by soil materials; and (ii) even if a geosynthetic does not perform a function that could be performed by a soil (e.g. if the geosynthetic performs the reinforcement function), the mode of failure of a structure incorporating geosynthetics may be similar to that of a soil structure. Examples are presented below.

2.2.2 Soil Particle Migration

There are unstable soils in which fine particles migrate when water flows through the soil. This is in particular the case for gap-graded soils. As indicated by Giroud (1982b, 1996) and Giroud et al. (1998), a gap-graded soil where the fine fraction is less than approximately 30% by weight can be unstable (i.e. fine particles can migrate) if it is subjected to a flow of water. If a filter is used in such a soil and if this filter has the appropriate opening size to retain the particles, clogging is likely to occur. This is true whether the filter is a sand or a geotextile. This is illustrated by the following case histories.

Case History — Road Edge Drain. This case history has been presented by *Bieth and Faure.* An edge drain along a road was placed 5 to 8 cm away from the edge of the pavement structure instead of being next to the pavement structure. The 5 to 8 cm thick space located between the pavement structure and the geotextile filter of the drain consisted of a soil that was internally unstable. As a result, fine particles of this soil migrated when water flowed from the pavement structure to the drain, thereby causing accumulation of particles in front of the filter. The resulting clogging caused the failure of the road drainage system and the formation of wet areas on the pavement surface.

Case Histories — Erosion Control Systems. Several failures of geotextile filters used with gap-graded silty sands have been analyzed by Fluet and Luettich (1993). In all those cases, the geotextile function — filtration — was properly identified, but the way the geosynthetic performs the function was not clearly understood by the designers of the projects. The soils were gap-graded; they contained fine particles that migrated, accumulated on the filter, and clogged it. These failures could have been avoided by using a geotextile filter with large openings so the migrating particles could pass. As shown by Fluet and Luettich (1993) and Gourc and Faure (1990), clogging of geotextile filters can result in spectacular uplifts because of the geotextile tensile strength, a property that sand filters do not have. Such spectacular failures — some of which occurred a long time ago — have led some engineers to reject geotextile filters. In reality, geotextile filters that are properly used function satisfactorily. For example, the first geotextile filter installed in a dam has been working with no problem for 28 years (Delmas et al., 1992, 1993; Giroud, 1984e; Giroud et al., 1977a, 1977b, 1984; Giroud and Gross, 1993).

Lessons Learned from the Above Case Histories. The above case histories show that the phenomenon of migration of fine particles in some soils cannot be ignored, and can result in the clogging of a geotextile filter (as it would clog a granular filter).

2.2.3 Instability of Slopes, Soil Masses, Waste Masses and Ore Heaps

The forces (e.g. gravity — possibly increased by seismic forces — and water pressure) that cause instability of soil, ore or waste slopes and masses that do not incorporate geosynthetics also cause instability of soil, ore or waste slopes and masses that do incorporate geosynthetics. Examples follow.

Examples — Waste Slides. Several examples of waste slides in landfills equipped with a geosynthetic liner system have been reported (*Chang et al., Ouvry et al., Stark et al.).* The forces involved in these slides (gravity, hence waste density, and pore water pressure) are the forces typically involved in landslides studied in geotechnical engineering. Therefore, landfill designers who do not have an in-depth knowledge of geotechnical engineering run a high risk. A case in point is leachate recirculation. This technique has been promoted by environmental scientists to accelerate leachate decomposition, which has merits from the viewpoint of waste management. However, many of those who have promoted leachate recirculation did not consider the geotechnical consequences of this technique. These consequences can be very serious because leachate recirculation significantly increases the moisture content of the waste, thereby promoting pore pressure buildup in the waste, which decreases the stability of the waste mass. Indeed, waste slides have been attributed to leachate recirculation (*Gross et al.*). In addition, as indicated in Sections 2.4.4, 3.4 and 3.5, the presence of geosynthetics in a slope, soil mass or waste mass often increases the risk of instability.

Examples — Ore Heap Slides. Several slides have been observed in the case of ore heaps constructed on pads equipped with geosynthetic liner systems, during or at the end of construction of the first lift of ore (typically 5 to 10 m high) (*Smith, Breitenbach*). This common type of failure of ore heaps is due to the fact that the most critical phase of the construction of an ore heap is often the first lift. The reason is the following. The ore is typically placed at the angle of repose. Then a horizontal bench is used between each lift to decrease the average slope to a value determined at the design stage in order to ensure an acceptable factor of safety. As a result, the factor of safety at the end of the construction of the first lift may be less than the factor of safety of the entire heap (and it is definitely less if the ore is a cohesionless material, which is generally the case, at least approximately). The design engineer who calculates the factor of safety for the entire heap may neglect to calculate the factor of safety for the first lift based on the belief that the factor of safety of a slope decreases as the height of the slope increases, forgetting that, in this case, the two slopes (i.e. the slope of the first lift and the average slope of the entire heap) do not have the same slope angle. Such a basic cause of failure exists regardless of the presence of geosynthetics. As a general rule, design engineers should evaluate the stability of any structure at all stages of construction.

Example — Surficial Instability of Reinforced Soil Slopes. The surficial instability of a reinforced soil slope, i.e. the localized instability that occurs at the face of the reinforced soil slope between geosynthetic reinforcement layers (*Collin*), is a reminder that layers of reinforcement act discretely. As a result, there may be locations in a reinforced soil structure where the geosynthetics do not have an impact on the soil behavior. In those locations, the behavior of the soil will be that of a soil without geosynthetic reinforcement. Therefore, in those locations, the modes of failure of a non-reinforced soil apply.

Examples — Rapid Drawdown. Two case histories (presented in Sections 2.4.2 and 3.6) illustrate failures of ponds with geosynthetics in case of rapid drawdown. The first case is that of a bank protection system incorporating a geotextile (Section 2.4.2), and the second case is that of a geomembrane liner placed on top of an existing liner (Section 3.6). Rapid drawdown is a situation typically considered by geotechnical engineers.

Lessons Learned from the Above Examples and Case History. The above examples and case history show that an engineer using geosynthetics should not forget the forces, well known in geotechnical engineering, that impact the stability of slopes and masses. These forces should be considered at all times (i.e. the design engineer must check that the structure is stable at all stages of construction) and all locations (i.e. the design engineer must check that all parts of the considered structure have an appropriate factor of safety).

2.2.4 Differential Settlements

One of the main causes of problems and failures in geotechnical engineering, differential settlement, also exists in the case of structures incorporating geosynthetics. Several examples are given below.

Case History — Geotextile-Reinforced Soil. The failures of the facings of two geotextile reinforced retaining wall bridge abutments built on soft soil have been reported by *Palmeira and Fahel*. The pavements of the highways supported by the bridge abutments were also damaged. The observed problems were mainly caused by differential settlements affecting the structure and by erosion due to a river flood. The investigation showed that, in spite of rather large differential settlements, the geosynthetic reinforced structures have been flexible enough to accommodate the differential settlements, requiring rather light repairs to the wall faces and pavement of the highway. Geotechnical engineers must design transition zones between geosynthetic-reinforced soil masses and adjacent non-reinforced fills to alleviate the effects of differential settlement.

Case History — Geotextile-Reinforced Slope. In the summer of 1982, an existing road, built partly on an embankment, was widened using a 3.2 m high geotextile-reinforced slope with a face batter of 1.5V:1H (*Ingold*). Construction was completed successfully, but by the spring of 1984 the pavement of the road extension supported by the slope was showing severe longitudinal cracking. The slope had suffered a serviceability failure with total collapse being imminent. The investigation revealed that an outlet pipe for surface water drainage, buried in the fill, had fractured, leading to development of a high internal water table in both the original fill and the reinforced fill, which in turn caused the failure. The pipe fracture was attributed to differential movement between the original embankment fill and the reinforced slope extension. An adequate degree of stability was restored by the use of remedial drainage.

Examples — Connections Between Geosynthetics and Structures. Differential settlement affects connections of geosynthetics to structures (such as connections of geomembrane liners to appurtenances, e.g. concrete structures), or elements of structure (such as connections of geosynthetic reinforcement to wall facing) (*Richardson*). In a number of cases, geomembrane liners have failed because of differential settlements between the soil supporting the geomembrane and the concrete structure to which the geomembrane was connected (Giroud, 1977a, 1977b, 1983a, 1983b, 1984a) and Giroud and Soderman (1995). Geotechnical engineers must review connections very carefully because they are sometimes designed by individuals who have an insufficient knowledge of geotechnical engineering and who, therefore, do not realize that even a small amount of differential settlement may cause the connection to fail.

Examples — Damage to Geotextile Filter. Differential settlement of dikes (including geotextile filters and traditional gabions) constructed across rivers have caused: (i) tension of the geotextile filter, resulting in tearing and puncturing of the geotextile at the contacts between the geotextile and gabions; and (ii) separation of overlaps between adjacent sheets of geotextile (*Vertematti*). Due to the high hydraulic gradient, these discontinuities in the geotextile filter resulted in significant piping and partial destruction of several dikes.

Lessons Learned from the Above Examples and Case Histories. The above examples and case histories confirm that differential settlements are a major cause of problems in geotechnical structures, whether they incorporate geosynthetics or not. Furthermore, structures incorporating geosynthetics provide new opportunities for differential settlements to cause problems, for example: differential settlement between reinforced and non-reinforced fill, and effects of differential settlements on geosynthetic-structure connections.

2.2.5 Soil Deformations

Unless they perform the reinforcement function, geosynthetics have very little impact on deformations of adjacent soils. A geosynthetic may be damaged by excessive tension due to the deformation of the adjacent soil, especially in the case of localized subsidence of the soil underlying the geosynthetic (see Section 2.3.6), or in the case of bank deformation by wave action, even though the waves are on one side of the geosynthetic and the bank is on the other side. Thus, there are many examples of geomembrane liners that have been affected by deformations of the supporting soil (*Fourie, Giroud*). Clearly, engineers designing with geosynthetics should consider all soil deformations that are possible (assuming that, in many cases, the geosynthetic will not have any significant influence on these deformations) and should evaluate the impact of soil deformation on the geosynthetics.

2.2.6 Soil Erosion

Soil erosion may affect structures incorporating geosynthetics as it affects traditional geotechnical structures (i.e. structures that do not incorporate geosynthetics). Erosion contributed to the failure of a canal liner protective soil layer reported by *Well*; several failures of landfill cover systems to which soil erosion contributed have been reported (*Gross et al.*); and the collapse of a modular-faced bridge abutment has been caused by erosion of the soil supporting the facing (*Gourc et al.*). Erosion was also a factor in the bridge abutment problems reported by *Palmeira and Fahel* (see Section 2.2.4). Also, geotextile filter clogging due to soil erosion has been reported by *Vertematti* (see the example below).

Case History — Clogging of Geotextile Filter. Dikes (including geotextile filters and traditional gabions) were constructed across rivers (*Vertematti*). Due to the rigidity of the gabions and the fact that the excavation was not smooth, the geotextile and the soil were not in intimate contact. As a result, water flowed between the geotextile and soil, causing soil erosion. Fine soil particles carried by water caused clogging of the geotextile (see Sections 2.2.2 and 2.4.2).

2.2.7 Improper Drainage

Improper drainage — or, more generally, improper control of water likely to reach a structure — which is a major cause of failure of traditional geotechnical structures (i.e. structures that do not incorporate geosynthetics) is also a major cause of failure of structures incorporating geosynthetics.

Example — Failure due to Insufficient Drainage. Lack of drainage occurred in the failure of a dewatering system reported by *Christopher* (see Section 3.7) due to insufficient pumping.

Examples — Failures due to Excessive Water Supply to a Structure. Failures of geosynthetic-reinforced soil structures have been caused, at least partly, by an excessive amount of water reaching the structure (*Bernardi, Frost et al., Giroud and Beech*) (see case histories in Sections 3.3). In a case reported by *Well*, the failure of a canal liner protective soil layer was caused in part by water infiltrating under the liner due to inadequate surface drainage and the lack of anchor trench on top of the geomembrane liner. In two landfills, water that had penetrated into the anchor trench at the top of the slope reached the geonet leakage collection and detection system, thereby causing false leakage detection (*Gross et al.*). In another case, the failure of a bank protection system incorporating a geotextile was caused by wastewater seeping from a wastewater pond (*Davis et al.*) (see the first case history in Section 3.3).

Lessons Learned from the Above Examples. The above examples confirm that water is a major cause of failures of geotechnical structures, whether they incorporate geosynthetics or not.

It is possible, also, that the water supply that causes the failure be conveyed by a geosynthetic having a high hydraulic transmissivity. This mechanism is illustrated by the failure due to excessive leakage of a dam, where water was conveyed from the reservoir to the dam by a geotextile, as described in the first case history below. Another example of the same mechanism is the failure of a pavement due to blisters generated by humid air conveyed by woven geogrids used to reinforce the pavement, as described in the second case history below.

Case History — Leakage through a Dam due to Liner Bypassed by Transmissive Geotextile. The geomembrane liner for the upstream face of a small earth dam was underlain by a needle-punched nonwoven geotextile acting as a cushion (*Levillain*). Both the geomembrane and the geotextile were anchored in the same anchor trench at the toe of the dam, with the geotextile slightly longer than the geomembrane. As a result, water from the dam reservoir could infiltrate into the geotextile. This water flowed along the geotextile and then through the dam, resulting in a considerable amount of leakage. This failure would not have occurred if the extremity of the geotextile had been properly sealed in the anchor trench.

Case History — Blisters in an Asphalt Pavement Reinforced Using Woven Geogrids. One of the most intriguing cases reviewed by the author of this paper has been reported by *Wilmers*. Blisters up to 0.6 m in diameter and 4 cm in height appeared in an asphalt overlay less than a year after construction of the overlay. Prior to construction of the asphalt overlay, a woven geogrid layer had been placed on the existing pavement to prevent cracking of the overlay. It took several night-day cycles for the blisters to grow to their final size, during a period where the days were warm and the nights were cool. The unique development of these blisters was explained and quantified by a pumping mechanism (*discussion by Giroud of the paper by Wilmers*): during the day, as a result of high temperature, the blister grows by vaporization of humidity contained in the entrapped air; and, during the night, as a result of low temperature, the blister does not decrease because the asphalt stiffens and the pressure in the blister decreases, thereby sucking into the blister some additional humid air. The air was able to flow toward the blister because it was conveyed in the ribs of the woven geogrids, which were not impregnated with asphalt during construction. The reported failure would not have occurred if the geogrid had been impregnated or coated with asphalt.

Lessons Learned from the Above Case Histories. The above case histories show that a needle-punched nonwoven geotextile and even a woven geogrid can convey enough fluid (water in the first example, and humid air in the second example) to cause a failure. Such cases should be identified by the design engineer and appropriate measures should be taken.

2.2.8 Frost Action

Geosynthetics, which are thin (with the exception of geofoam panels), are unable to prevent frost from penetrating into the underlying soil. Therefore, failure mechanisms involving frozen soil are to be considered. For example, a canal liner system incorporating a geomembrane failed by sliding on ice lenses, when the canal was empty during the winter (*Frobel and Comer*). A similar failure occurred on a landfill cover (*discussion by Luettich et al. of Frobel and Comer's paper*); in a case mentioned by *Luettich et al.*, the formation of ice was due to high water content of the soil resulting from heavy rainfalls prior to the placement of the geomembrane (see Section 2.4.4).

Geofoam panels are thick and have a low coefficient of thermal conductivity. As a result, they are used to delay, minimize or prevent frost penetration (*Horvath*), especially under pavements. However, the phenomenon of differential icing reported by *Horvath* results from an unexpected disadvantage of the low coefficient of thermal conductivity of geofoam: under special and rare weather conditions, a thin sheet of ice has developed on a pavement section underlain by geofoam, while ice did not develop on the adjacent pavement section that did not incorporate geofoam, thereby creating dangerous driving conditions. This may be considered as one of the most subtle cases of serviceability failure.

2.2.9 Conclusions Regarding Failure Modes Common to Soils and Geosynthetics

The above examples and case histories show that engineers designing with geosynthetics must have a broad knowledge of geotechnical engineering, and, in particular, a comprehensive knowledge of the failure modes of traditional geotechnical engineering structures (i.e. structures not incorporating geosynthetics). They should review all the classical failure modes relevant to the considered structure and evaluate if these failure modes are impeded by the presence of the geosynthetic.

It is interesting to note that, while geotechnical engineering should not be ignored by those designing with geosynthetics, knowledge of geotechnical engineering has been increased thanks to well designed geosynthetic projects or research conducted on geosynthetics. For example, knowledge of granular filters has benefited from research on geotextile filters.

2.3 Failure Modes Associated with the Two-Dimensional Nature of Geosynthetics

2.3.1 Overview of Failure Modes Associated with the Two-Dimensional Nature of Geosynthetics

Geosynthetics are the only two-dimensional materials used in geotechnical engineering. A two-dimensional material is thin (hence light) and continuous. As a result of these characteristics, geosynthetics may exhibit the following modes of failure: (i) failure modes due to lack of geosynthetic continuity at the time of construction; (ii) failure modes due to damage to the geosynthetics caused by tensile and compressive stresses, in particular concentrated stresses; (iii) failure modes due to "out of plane" stresses, i.e. stresses applied in a direction approximately normal to the geosynthetic, thereby causing geosynthetic deformation perpendicular to its plane (i.e. uplift and localized subsidence), which results in tensile stresses in the geosynthetic and, if the stresses are excessive, in tensile rupture; and (iv) failure modes resulting from thermal expansion and contraction, which are related to the two-dimensional nature of geosynthetic, as temperature affects differently the two-dimensional geosynthetic and the three-dimensional adjacent soil.

2.3.2 Lack of Continuity

Since a basic characteristic of a two-dimensional material is continuity, a basic failure mode of a geosynthetic is lack of continuity. This failure mode may result from mechanical damage, as discussed in Sections 2.3.3 and 2.3.4, or because overlaps separated as a result of differential settlement (see Section 2.2.4) or localized subsidence (see Section 2.3.6). This failure mode may also occur because a geosynthetic was not continuous when it was installed, as shown in the examples given below. The lack of continuity of a geosynthetic may result in a failure that is particularly severe if the hydraulic gradient is large, as illustrated by dike failures (*Vertematti*) due to lack of continuity of geotextile filters (with the geotextile discontinuity resulting from overlaps that were insufficient or should have been sewn, and from tears and punctures), and the failure of an earth dam reported by *Sembenelli* and attributed to lack of continuity of a geomembrane liner (see the case history below).

Examples — Lack of Continuity of Geotextile Filters. Failures due to lack of continuity of geotextile filters may result from overlaps that are insufficient, which occurred in a case history of a dewatering system reported by *Christopher* (see Section 3.7), and may have occurred in the case of a sewer pipe excavation reported by *Rowe and Seychuk.* The dike failures mentioned above (*Vertematti*) resulted from lack of continuity of geotextile filters due to differential settlement (see Section 2.2.4).

Examples — Lack of Continuity of Geomembrane Liners. Failures due to lack of continuity of geomembrane liners occur when geomembrane seams are inadequate (numerous examples are provided by *Laine and Darilek*, *Rollin and Jacquelin*, and *Nosko and Ganier*), or connections between geomembrane liners and appurtenances are not waterproof (see, in Section 3.2, the case history of a geomembrane liner on a karstic soil).

Case History — Dam Failure Due to Lack of Continuity of Geomembrane Liner. A geomembrane liner was placed over the entire area of the reservoir of a dam. The dam was a 18 m high embankment dam with an upstream clay zone. The geomembrane liner did not extend on the upstream face of the dam; rather, the geomembrane lining the reservoir floor was stopped at the upstream toe of the dam and buried in the clay with a flat end. There were no seams between adjacent geomembrane sheets; rather, all geomembrane connections were made by folding the ends of adjacent sheets over twice. After five years of operation, breaching of the dam occurred within a few hours. First, seepage was noticed in the immediate vicinity of the dam; then the rate of seepage increased rapidly and the seeping water started carrying solids, until the dam was breached. The breach was 40 m wide and the 1,500,000 m^3 of water stored in the reservoir escaped through the breach. The forensic analysis (*Sembenelli*) showed that the failure could be explained by piping due to the combination of high hydraulic gradient and lack of continuity of the geomembrane liner: (i) the folded connections between adjacent geomembrane sheets were not sufficiently watertight; and (ii) the connection between the geomembrane and clay at the toe of the dam created preferential paths for the water.

Lessons Learned from the Above Examples and Case History. The above examples and case history show that lack of continuity of geosynthetics can cause failures, in particular the geosynthetics that perform functions where high hydraulic gradients are involved, such as filters and geomembrane liners. Therefore, great precautions should be taken during placement of geotextile filters and geomembrane liners at locations where high hydraulic gradients are likely to exist.

2.3.3 Mechanical Damage Caused by Tensile Stresses

Due to their two-dimensional nature, geosynthetics are often subjected to tensile stresses. Therefore, the most typical mode of geosynthetic failure is tensile rupture, which occurs when the tensile stresses are excessive. Tensile rupture may happen when the excessive tensile stresses are applied or at a later time due to creep (since all polymeric materials exhibit creep). Tensile rupture typically occurs as the result of a failure mechanism of which many examples are given in this paper (e.g. uplift, subsidence, waste or ore slide, collapse of a structure). Since tensile rupture is the principal mode of failure of geosynthetics, design engineers should make every effort to avoid situations likely to lead to excessive tensile stresses in geosynthetics. In particular, they should try to avoid concentrated tensile stresses as discussed below.

Because geosynthetics are thin, any abrupt change in their thickness may cause concentration of tensile stresses likely to result in geosynthetic rupture. (Indeed, any change in thickness is a departure from the ideal two-dimensional conditions.) This is particularly the case for geomembrane seams. Examples are given below.

Examples. A number of geomembrane failures have been observed next to seams (Giroud, 1994a, 1994b; *Thomas and Kolbasuk*). Many of these failures result from stress concentration as shown and quantified by Giroud (1994b) and Giroud et al. (1995b). An interesting example is provided by *Sharma and Settepani*. They reported the case of a long tear in a geomembrane liner in a landfill that occurred near the top of a slope during an earthquake and started at the seam of a patch covering a hole in the geomembrane resulting from the removal of a geomembrane sample for destructive testing.

Lessons Learned from the Above Examples. The above examples show that the number of seams in a geomembrane liner should be minimized. In particular, the number of samples removed from an installed geomembrane liner for destructive testing should be limited. Also, these samples should not be taken in areas where tensile stresses are likely to be large, such as the tops of slopes. Finally it is suggested that research be conducted to develop seams that cause minimum stress concentration.

Regarding the recommendation for minimizing the number of seams, an interesting case has been discussed by Giroud et al. (1993). The specifications for the construction of a geomembrane liner for a landfill included criteria for destructive testing with extremely high values for the shear and peel strengths of the tested seams. These values were so high that many samples failed the tests. As a result, more tests were needed and additional samples had to be taken. The intent of the stringent criteria was to improve liner quality; the result was a liner weakened by an excessive number of patched holes.

2.3.4 Mechanical Damage Caused by Compressive Stresses

Some geosynthetics are compressible and their performance can be affected by compressive stresses such as overburden stresses. This is the case for geonets (Giroud, 1985) and needle-punched nonwoven geotextiles (Giroud, 1981, 1996). Although no failures have been reported that can be attributed to compressive stresses, some of the characteristics of certain geosynthetics are affected by compressive stresses. For example: (i) the opening size of needle-punched nonwoven geotextile filters is reduced, which can be quantified, as shown by Giroud (1996); and (ii) the hydraulic transmissivity of geonets and needle-punched nonwoven geotextiles is reduced. Design engineers should address this issue in their designs. A problem that is related to the effect of compressive stresses is the intrusion of a geosynthetic into the adjacent geosynthetic (see Section 2.4.3). The effect of compressive stresses on geocomposites may in fact result from the intrusion of the geotextile component of the geocomposite into the geonet component of the geocomposite. Design engineers should make every effort to avoid the detrimental effects of compressive stresses, in particular the concentrated stresses discussed below.

Because they are thin, geosynthetics may be damaged by concentrated stresses. The typical modes of rupture due to concentrated stresses are: (i) puncture and burst, which are two types of concentrated normal stresses; and (ii) tear and grab, which are two types of concentrated tensile stresses, but are initiated by puncture. Accordingly, *survivability tests*, which are intended to evaluate the ability of a geotextile to "survive" concentrated stresses in the field, include tear, grab, puncture and burst tests. This type of damage occurs during construction ("construction damage") or after if adequate precautions are not taken, and may result in a major failure if the geosynthetic performs a function for which continuity is critical (e.g. geotextile used for filtration and/or separation; and geomembrane or geosynthetic clay liner used as fluid barrier). Another type of failure linked to the thinness of geosynthetics is abrasion.

Examples. The following types of mechanical damage have been reported: (i) puncture, tear and abrasion of geotextiles (*Raymond*, for the case of railroad tracks); (ii) puncture, tear, abrasion and other mechanical damage of geomembranes (*Well*, *Datye and Gore*, *Laine and Darilek*, *Rollin and Jacquelin*, *Nosko and Ganier*), in particular during placement of soil on top of a liner system (*Nosko and Ganier*) and due to steel wire mesh used for the reinforcement of concrete slabs (*Datye and Gore*); (iii) puncture of the entire liner system of a landfill by a well driven through the waste (*Gross et al.*); (iv) geomembrane liner cut by vandals and by construction workers shooting nails intentionally (both cases observed by the author of this paper) (v) damage to geotextiles and geomembranes by small animals (*Datye and Gore*), which is extremely rare, whereas damage by large animals (e.g. deer, bears) is not infrequent, especially during construction, as pointed out by Thiel (1999); and (vi) puncture of geosynthetic clay liner and decrease of geosynthetic clay liner thickness due to squeezing of bentonite (*Peggs and Olsta*).

Example — Statistics Regarding Damage to Geomembrane Liners. Based, in particular, on results of electric leak detection and location surveys presented by *Nosko and Ganier*, *Gross et al.* have established the following approximate statistics for geomembrane defects in geomembrane liners that do not exhibit any failure mode other than localized defects:

- 25% of the detected leaks are due to installation problems (including 20% inadequate seams and 5% mechanical damage);

- 70% of the detected leaks are due to mechanical damage caused during placement of the overlying soil; and

- 5% of the detected leaks are due to problems that occurred during operations.

Since problems that occurred during operations probably result from mechanical damage, it appears that 80% of geomembrane defects are due to mechanical damage and that most occur during placement of the soil overlying the geomembrane.

Lessons Learned from the Above Examples. Several of the above examples show that great precautions should be taken during placement of geomembranes and geosynthetic clay liners. Construction quality assurance of geomembrane and geosynthetic clay liner installation is essential. It should include monitoring the placement of materials overlying the geomembranes and geosynthetic clay liners. It may also include an electric leak detection and location survey after placement of the materials overlying the geomembranes. Since the placement of overlying materials is a major cause of damage to geomembranes, design engineers should, whenever possible, select overlying materials that are least likely to damage the geomembrane: (i) when granular soil is used, the particles should be as small as possible; (ii) if necessary, a thick needle-punched nonwoven geotextile acting as a cushion should be used between the geomembrane and the granular soil; and (iii) when reinforced concrete is used to protect a geomembrane (as on the upstream face of dams), fiber-reinforced concrete should be preferred to the traditional concrete reinforced with steel bars (Tisserand et al., 1997).

Precautions must also be taken during the placement of other geosynthetics because construction damage may reduce the ability of a geotextile to perform its function. For example, holes may cause a geotextile filter to fail, and construction damage may reduce the ability of a geogrid to reinforce a soil.

2.3.5 Uplift

Because they are continuous (and especially when they have a low permeability such as geomembranes and geosynthetic clay liners), geosynthetics allow pressures to develop under them and are sensitive to negative pressure (suction) applied by the wind; and, because they are light, geosynthetics are relatively easily uplifted by these pressures. Regarding uplift by the wind, it should be noted that, when a geomembrane starts being uplifted, a suction develops under the geomembrane. This suction is negligible if the material underlying the geomembrane is permeable, but can be significant if the material underlying the geomembrane has a low permeability. As a result, geomembranes are less likely to be uplifted by the wind if the material underlying the geomembrane has a low permeability (assuming that the geomembrane is sealed at its periphery and is not torn or punctured).

Examples. Examples of uplift include: (i) uplift of geomembrane or geotextile by wind (Giroud et al., 1995a; Zornberg and Giroud, 1997; *Fairborn and McKelvey, Fayoux*); (ii) uplift of geomembrane liner or geosynthetic clay liner by liquid, which may occur when the liner blocks the natural drainage of ground water (*Bonaparte, Datye and Gore*); (iii) uplift by gas of geomembrane liner in a pond (*Fayoux, Frobel, Giroud*) or geomembrane cover in a landfill (*Richardson et al.*); (iv) uplift of geomembrane liner by air entrapped between the geomembrane and liquid that is present under the liner (*Fayoux, Giroud*) (the air generally accumulates under the geomembrane liner over high spots of a rather flat pond bottom during the first filling of the pond, and the liquid present under the geomembrane pond liner results either from leakage through the geomembrane or from an external source, such as a rising ground water) ; (v) uplift of geotextile filter, as mentioned in the case history on erosion control system presented in Section 2.2.2; and (vi) uplift of a geosynthetic used for erosion control (*Jacotot et al.*). Uplift may result in two types of failures: disruption in the operation of the structure or geosynthetic damage. For example, an uplifted geomembrane may be damaged, in the short term, by excessive tensile stresses or, in the long term, by creep, by exposure to wind and sunlight, and by being a tempting target for vandals and hunters.

Lessons Learned from the Above Examples. Engineers designing a geosynthetic application where the geosynthetic is not covered by a layer of soil, or is covered by only a very thin layer of soil or a layer of soil that could be eroded or otherwise removed, should identify all possible sources of fluids capable of uplifting the geosynthetic.

2.3.6 Localized Subsidence

Geosynthetics are often placed horizontally and normal stresses develop on top of them. As a result, in case of localized subsidence of the underlying medium, geosynthetics sag, stretch and may rupture. It should be noted that both uplift and localized subsidence result from out of plane stress, i.e. the geosynthetic, a two-dimensional material, is subjected to three-dimensional stresses.

Example — Localized Subsidence of a Sludge Pond Cover System. Badu-Tweneboah et al. give the example of a sludge pond cover system that exhibited localized subsidence during construction under the weight of equipment due to lack of bearing capacity of the sludge. The cover system of the sludge pond included a PVC geomembrane and a geosynthetic clay liner. The PVC geomembrane was able to elongate sufficiently to follow the deformed shape of the supporting material without rupturing, whereas the geosynthetic clay liner separated at a seam.

Case Histories. In the two case histories presented in Section 3.2, the geomembrane failure was caused by localized subsidence.

Lessons Learned from the Above Example and Case Histories. Excessive deformation followed by tensile rupture is a typical mode of failure of geosynthetics. Engineers designing a geosynthetic application should be knowledgeable in geotechnical engineering to be able to identify all possibilities of localized subsidence and to make a reasonable assumption of the areal extent and depth of the subsiding area. Then, quantification of the geosynthetic strain and tension is relatively easy thanks to available analytical methods (Giroud, 1982a; Giroud et al., 1990a; Giroud, 1995b).

2.3.7 Thermal Expansion and Contraction

All polymers have a large coefficient of thermal expansion-contraction (i.e. about ten times as large as that of usual construction materials such as concrete and steel). However, polymers that are drawn (i.e. stretched in a controlled manner to make fibers and geogrid ribs) do not expand and contract much (and may even shrink, but in an irreversible manner, when heated excessively). As a result, geonets and non-reinforced geomembranes significantly expand or contract when temperature increases or decreases, respectively; in contrast, geotextiles, geogrids and reinforced geomembranes do not exhibit significant thermal expansion and contraction.

Thermal expansion of geomembranes results in wrinkles. The drawbacks of geomembrane wrinkles are as follows: (i) they can be damaged by placement of soils over them; (ii) they prevent intimate contact between a geomembrane and the underlying clay in a composite liner, as indicated in Section 2.4.2; and (iii) in landfills, they impede the flow of leachate in leachate collection layers. The mechanism of geomembrane wrinkle formation, which has been demonstrated by Giroud and Morel (1992) and further discussed by Giroud (1994b, 1995a), can be summarized as follows: (i) a wrinkle develops a bending moment that is a function of the bending modulus of the geomembrane; (ii) this bending moment must be balanced by resisting forces in the plane of the geomembrane on each side of the wrinkle; (iii) the resisting forces are generated by interface shear strength (essentially friction) between the geomembrane and the underlying medium; (iv) therefore, the resisting forces are proportional to the distance between consecutive wrinkles; (v) as a result, the distance between wrinkles is small — and, consequently, *the wrinkles are small* for a given thermal expansion — when the resisting forces per unit length of geomembrane are large; and (vi) the resisting forces per unit length of geomembrane are large if the friction between the geomembrane and the underlying medium is high and the mass per unit area of the geomembrane is high. Therefore, geomembranes that are less likely than others to exhibit wrinkles are: (i) geomembranes with a white upper surface, which reduces thermal expansion by limiting the geomembrane temperature; (ii) geomembranes with a small coefficient of thermal expansion, such as reinforced geomembranes; (iii) geomembranes with a low bending modulus, i.e. flexible geomembranes; (iv) geomembranes with a textured lower surface, which generally results in a high interface shear strength between the geomembrane and the underlying medium; and/or (v) heavy geomembranes.

Thermal contraction results in geomembrane tension, which may cause tensile failure (in the case of geomembranes that become stiffer at low temperature) or stress cracking (Giroud, 1994a; *Giroud, Thomas and Kolbasuk*).

<u>2.4 Failure Modes due to Interaction Between Geosynthetics and Adjacent Materials</u>

2.4.1 Overview of Failure Modes due to Interaction

Interaction between geosynthetics and adjacent materials may involve mostly normal stresses (Sections 2.4.2 and 2.4.3), shear stresses (Section 2.4.4), and tensile stresses (Section 2.4.5).

2.4.2 Intimate Contact

There are a number of cases where geosynthetics need to be in intimate contact with the adjacent soil to perform their function (Section 1.3). A geosynthetic may not be in intimate contact with the adjacent soil if it lacks flexibility or if it has not been applied against the soil with sufficient pressure. Examples are given below.

Examples — Composite Liner. Wrinkles due to thermal expansion prevent geomembranes from being in intimate contact with the underlying low-permeability soil (compacted clay or geosynthetic clay liner), thereby preventing the formation of an effective composite liner. Further, wrinkles can promote the mechanism of desiccation that deteriorates the clay component of a composite liner, which is illustrated by the following case history.

Case Histories — Desiccation of Clay Component of Composite Liner. Observations made on several slopes lined with geomembranes that are not overlain with a protective cover (*Basnett and Bruner*) have shown that clay, placed immediately under the geomembrane, could undergo significant desiccation and cracking, whereas common sense indicates that the sheer presence of the geomembrane should prevent evaporation and help the clay keep its moisture content constant. According to *Basnett and Bruner*, the explanation is as follows:

- during the day, the geomembrane heated by the sun expands and becomes wrinkled;
- air entrapped in the wrinkles becomes saturated with water vapor that evaporates from the clay;
- during the night, the geomembrane cools and water vapor condenses against the lower face of the geomembrane;
- condensation water migrates along the slope toward the toe of the slope; and
- at each cycle, clay desiccates a little more, and the toe of the slope becomes more humid.

Lessons Learned from the Above Case Histories. The following lessons can be learned from the above case histories:

- Under certain circumstances, a geosynthetic may have a detrimental effect on the behavior of another material.
- Some detrimental effects of geosynthetics result from mechanisms that are not easy to predict, even if they can be explained after observations are made. Engineers must therefore keep themselves informed of field observations.
- Common sense, as often, can be misleading.

Examples — Filters. Ever since the first observations of the performance of Valcros dam, the first dam with a geotextile filter (Giroud et al., 1977a), the importance of intimate contact between a geotextile filter and the adjacent soil has been recognized. The concept has been discussed in detail by Giroud

(1989, 1996). Examples of failures due to lack of intimate contact between a geotextile filter and the adjacent soil have been reported by *Koerner and Koerner* and by *Vertematti*. In these two cases, the lack of intimate contact between the geotextile and soil was due to the fact that the geotextile filter was attached to a relatively rigid element: prefabricated edge drains (*Koerner and Koerner*) and gabions (*Vertematti*) (see the example in Section 2.2.6).

Examples — Asphalt Overlays. When a geotextile is used between an existing pavement and an asphalt overlay, it is essential to have intimate contact, between the geotextile and the underlying pavement and between the geotextile and the overlay, to ensure that the overlay will be entirely supported by the existing pavement, thereby preventing localized cracking of the overlay. A failure due to lack of contact between a geotextile and the underlying pavement has been reported by *Guram*. The failure was due to the fact that the geotextile, after impregnation with asphalt, was rolled with a steel roller. This roller, being rigid, could not force the geotextile to follow the shape of the existing pavement surface which was irregular. Instead, a pneumatic roller should be used.

Examples — Geomembrane Liner in an Underground Reservoir. In the case history presented in Section 3.8 (Giroud and Stone, 1984; Stone, 1984; *Giroud and Stone*), failure of the geomembrane liner occurred because the geomembrane was installed too far from the corners of the reservoir. As a result, the geomembrane underwent excessive strain during the first filling of the reservoir as it was forced to move toward the reservoir corners by water pressure.

Lessons Learned from the Above Examples. The following lessons can be learned from the above examples:

- Intimate contact appears to be a major consideration in a variety of applications of geosynthetics.

- Clear instructions must be given to contractors regarding the importance of proper placement of geosynthetics and adjacent materials to ensure intimate contact.

Examples — Bank Protection. When a geotextile is used between an armor system (e.g. rocks or concrete blocks) and a bank, intimate contact between the geotextile and the bank is essential. This has been discussed in detail by Giroud (1996). Examples of bank protection failures due to lack of intimate contact have been reported (*Giroud, Gustin, Heerten*).

Case History — Bank Protection. The following case has been reported by Giroud (1993) and *Giroud*. Two identical ponds, surrounded by earth embankments, had their banks protected with a layer of rounded blocks placed on a geotextile filter. Rapid drawdown of the water in one of the ponds caused the bank protection (i.e. blocks and geotextile) to slide. Investigation of the other pond showed that the geotextile had many wrinkles, all filled with fine soil particles. During the drawdown of the pond, the quasi-impermeable layer formed by these fine particles did not allow a fast enough dissipation of the pore water pressure resulting from the presence of water in the embankment. This pressure uplifted the fine particles and the bank protection, just enough to decrease the shear strength at the interface between the fine particles and the rest of the embankment, hence the slide.

The investigation showed that the geotextile had been installed without wrinkles and that the wrinkles resulted from the placement of the blocks. Because of the wrinkles, the geotextile was not in close contact with the soil; consequently it could not prevent the fine soil particles from being eroded by wave action and rain. The particles thus eroded, accumulated naturally in the space between the geotextile and the bank. It should be noted that nothing in the specifications indicated the importance of a close contact between the geotextile and the soil.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- Geosynthetics are construction materials like soil, and failure mechanisms (such as uplift of materials in case of rapid drawdown) which are known in geotechnical engineering can occur with geosynthetics.

- Some construction methods can have a detrimental effect on the performance of geosynthetics. For example, any construction method that does not ensure close contact between a geotextile filter and soil leads almost automatically to clogging of the filter (Giroud, 1989, 1996).

- The following recommendations can be made to ensure intimate contact between a geotextile filter and the adjacent soil. In the case of drainage trenches, this can be achieved by using a flexible geotextile and filling the trench with relatively small stones (Giroud, 1996). In the case of a relatively rigid edge drain, sand should be poured in the space between the edge drain filter and the walls of the trench (Giroud, 1996; *Koerner and Koerner*). In the case of bank protection, this can be achieved by using a flexible geotextile (*Heerten*) with a layer of relatively small stones between the geotextile and the rocks (Giroud, 1996).

- Specifications must be sufficiently complete to prevent the contractor from using construction methods that may have a detrimental effect on geosynthetic performance.

- Users of geosynthetics (i.e. designers and contractors) must be educated to prevent the use of construction methods that may have a detrimental effect on geosynthetics.

2.4.3 Intrusion

When two geosynthetics are in contact, one of them may intrude (i.e. partly penetrate) into the other. This is the case in particular for geotextiles penetrating into geonets, thereby reducing their hydraulic transmissivity, a problem that was reported first by Williams et al. (1984). No failure due to geotextile intrusion into geonet has been reported, but a potential failure was prevented at the last minute (see, in Section 3.4, the case history on flow capacity reduction). Another example of intrusion is the case of geonet strands indenting into geomembranes, thereby locally reducing the geomembrane thickness. However, this case is rare (*Laine and Darilek*, and *discussion by Giroud et al. of Laine and Darilek's paper.*).

Another type of intrusion is that of bentonite particles which may extrude from geosynthetic clay liners and, then, intrude into the adjacent material or interface. The influence on bentonite on interface shear strength is discussed in Section 2.4.4. The author of this paper has considered and evaluated, in the design of a landfill liner system, the possibility for bentonite from a geosynthetic clay liner to intrude into the adjacent geonet, thereby reducing the hydraulic transmissivity of the geonet or even clogging it. However, no failure of this type has been reported.

2.4.4 Interface Shear Strength

Geosynthetics are more or less smooth. At best, the shear strength at a geosynthetic-soil interface is equal to the shear strength of the soil. Therefore, in general, the presence of geosynthetics creates one or several planes of low shear strength. These planes increase the risk of instability. Instability of a soil, waste or ore slope or mass is one of the main causes of failures associated with geosynthetics. Also, asphalt overlays may fail by sliding due to insufficient interface shear strength. Examples follow.

Examples — Liner System on Slope. Numerous examples of failures show that liner systems incorporating geosynthetics can be unstable on slopes (*Richardson et al., Stark et al., Bonaparte et al., Vander Linde et al., Frobel and Comer, Kavazanjian).* As the liner system is relatively thin compared to the length of the slope, these cases are often referred to as "veneer stability problems" (see Section 2.2.3 for the instability of large masses). In all these slides, a key role was played by low interface shear strength. Important parameters that played a role in the reported failures were: water pore pressure (*Richardson et al., Vander Linde et al., discussion by Giroud et al. of Vander Linde et al.'s paper*); gas pore pressure (*Richardson et al.*); and bentonite hydration and extrusion (*Bonaparte et al.*) in the case of liner systems incorporating geosynthetic clay liners. In the case history described by *Frobel and Comer* (Section 2.2.8), frost developed in the soil supporting the geomembrane (in spite of the fact that the soil was covered with the geomembrane and a soil layer), which resulted in a stability problem because ice lenses developed in contact with the geomembrane. The placement of the soil layer on a liner system on a slope may have an influence on stability; the slide of a soil layer on a geomembrane on the 1V:2.5H upstream slope of a dam has been reported (Girard et al., 1990) where the slide was attributed in part to the braking of construction equipment working on the soil layer and to the vibration by the vibratory compactor used on the soil layer.

Examples — Waste Slides. Several examples of waste slides in landfills equipped with a geosynthetic liner system were mentioned in Section 2.2.3 (*Chang et al., Ouvry et al., Stark et al.).* In all these slides, a key role was played by low interface shear strength. An important parameter that played a role in some of the reported failures was the water content at the geosynthetic-soil interface (*Chang et al., Ouvry et al.*). In the latter case, the high water content was due to rainfalls prior to geomembrane placement (see also Section 2.2.8).

Case History — Waste Slide. In a hazardous waste disposal landfill, a waste slide occurred, with the slip surface along the geomembrane. The investigation (Giroud, 1993) showed that waste stability calculations had been conducted using circular slip surfaces entirely contained within the waste mass and that no slip surface along the geomembrane had been considered by the design engineer. The design engineer had learned from a waste slide which had occurred in another landfill (where no waste stability analyses had been conducted) that the possibility of a slide should be considered. However, this engineer had not learned how to select the critical slip surface, i.e. the engineer had not learned that the presence of a geosynthetic can create one or several planes of low shear strength along which a slide can occur.

Examples — Ore Heap Slides. Several slides have been observed in the case of ore heaps constructed on pads equipped with geosynthetic liner systems (*Smith, Breitenbach*) (see Section 2.2.3.). In all these slides, a key role was played by low interface shear strength associated with the geomembrane liner. An important construction recommendation has been made by *Smith* and discussed in detail by *Smith and Giroud*: when ore is placed on a pad with a slope (e.g. 5%), the ore should be placed in the up-slope direction, because if there is any departure from the designed slope (e.g. 6% slope) the factor of safety increases in case of up-slope placement and decreases in case of down-slope placement. This recommendation results from failures observed in the field (*Smith*) and has been theoretically demonstrated (*Smith and Giroud*) based on slip surfaces that run along the geomembrane lining the pad. Clearly, this recommendation stems from the influence of interface shear strength on stability.

Lessons Learned from the Above Examples and Case History. The following lessons can be learned from the above examples and case history:

- The presence of a geosynthetic can create one or several planes of low shear strength along which a slide can occur.

- The water content of the soil adjacent to a geomembrane can have a significant influence on the interface shear strength.

- The presence of a low-permeability geosynthetic (geomembrane or geosynthetic clay liner) promotes the buildup of water and gas pore pressure, which can have a detrimental impact on stability.

Examples — Asphalt Overlays. When a geotextile is used between an existing pavement and an asphalt overlay, it is essential to have high interface shear strength, between the geotextile and the underlying pavement and between the geotextile and the overlay, to ensure that the overlay will not fail by sliding along one of the two geotextile interfaces. In the case of asphalt overlays, the interface shear strength is essentially of the adhesive type. It is ensured by using the appropriate amount of asphalt to impregnate the geotextile and coat its two surfaces ("asphalt tack coat"). *Baker and Marienfield* have shown that the main cause of asphalt overlay failure is lack of geotextile-asphalt pavement or overlay-geotextile adhesion due to an insufficient amount of asphalt tack coat.

Failures of asphalt overlays have been reported by *Wilmers* in cases where geogrids were used between an existing pavement and an asphalt overlay. These failures were due to the fact that the presence of the geogrid prevented proper bonding between the overlay and the existing pavement, thereby decreasing the interface shear strength between the overlay and the existing pavement.

2.4.5 Soil-Geosynthetic Tensile Interaction

When geosynthetics are used for soil reinforcement, a certain geosynthetic strain is required to mobilize the geosynthetic tension. This strain requires a displacement of the soil, which may cause a deformation of the geosynthetic-reinforced soil structure. The deformation of the structure may affect its serviceability to the point that it may be considered that the structure failed (see below the case history reported by *Bathurst et al.*). Sometimes, the wrap-around facing of a geosynthetic-reinforced soil wall is hidden behind a non-structural vertical facing to protect the geosynthetic from exposure to sunlight and for aesthetic reasons. It is important to leave sufficient space between the wrap-around facing and the non-structural vertical facing to allow for lateral displacement of the wrap-around facing as the geosynthetic tension progressively increases. The failure of a geogrid-reinforced soil wall involving this mechanism has been reported by *Frost et al.* (see the case history in Section 3.3).

Pre-tensioning of the geosynthetic reinforcement may be a way to minimize the required soil and structure deformation. On the other hand, installing a reinforcing geotextile with wrinkles may result in the collapse of the reinforced-soil structure, because the soil may not be able to deform sufficiently to mobilize the geotextile strength that would be required to ensure the stability of the structure. An example of such a failure has been reported by *Gale.*

Case History — Reinforced Soil Wall. In a case reported by *Bathurst et al.*, a 14 m high modular faced vertical geogrid-reinforced soil wall was constructed to retain the soil on one side of an excavation for the construction of a multi-story building. The wall facing moved laterally by up to 150 mm. As a result, it was impossible to construct the building on the foundations that had been constructed at the bottom of the excavation, near the wall facing. It was, therefore, necessary to construct new foundations away from the wall facing, which caused significant additional expenses. While the geogrid-reinforced soil wall performed the function of retaining the soil, it was considered to have failed, i.e. it was not a structural failure but it was a serviceability failure.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history: a geosynthetic-reinforced soil wall always exhibits lateral deformation due to the progressive tensioning of the layers of geosynthetic reinforcement. This deformation can be evaluated (*Bathurst et al.*) and must be accounted for in designing structures to be constructed next to a geosynthetic-reinforced soil wall.

In the above case history, as the geosynthetic tension increases, the soil deformation is in the direction of the geosynthetic, i.e. perpendicular to the wall face. In contrast, in the case of a geosynthetic reinforcing an unpaved road, the required soil deformation is perpendicular to the geosynthetic. An interesting example of such deformation is provided by the case history on the geotextile used in the construction of an airport taxiway presented in Section 3.5 (Giroud, 1993). In that case history, the geotextile was reinforcing a temporary access road and the required deformation of the geotextile and soil was perpendicular to the geotextile. Furthermore, this is interesting because the geotextile function that caused the soil deformation was a temporary function performed by the geotextile during construction. This shows that the design engineer must not only consider the functions performed by the geosynthetic in the designed structure, but also the functions that the geosynthetic may temporarily perform during construction.

Another aspect of soil-geosynthetic mechanical interaction is the necessary compatibility between the displacements of all of the materials (such as geosynthetics, soil or waste) and interfaces involved in a deformation mechanism. In particular, two adjacent materials must have the same displacement, unless they slide with respect to each other (which generally, but not always, is undesirable). If two materials have displacements that are compatible (i.e. related through a certain mechanism) or equal, it is unlikely that they will both mobilize their maximum shear strength, because the strains that correspond to these displacements are unlikely to be precisely the strains that are required for the two materials to mobilize their maximum strength. Indeed, each material requires a certain strain to mobilize its own strength, and the strain required to mobilize the geosynthetic strength is generally different from the strain required to mobilize the strength of the adjacent material (such as soil or waste). This important point is illustrated by the following example.

Example — Landfill Slide. The analysis of a landfill failure (*Stark et al.*) showed that the waste mobilized its peak shear strength at a displacement 10 to 15 times larger than the displacement at which the critical interface (between the geosynthetic clay liner and the compacted soil liner) mobilized its peak shear strength.

2.5 Failure Modes Due to Geosynthetic Degradation

2.5.1 Geosynthetic Degradation

The failure modes discussed in Sections 2.2, 2.3 and 2.4 resulted from the application of stresses to the geosynthetics or the structures incorporating geosynthetics. These failure modes can be characterized as macro-failures. Section 2.5 is devoted to micro-failures, i.e. failures that result from material degradation. Virtually all of these failure modes result from the polymeric nature of most geosynthetics. Indeed, polymers are physically and chemically affected by fire, heat, ultraviolet radiation, radioactivity, and chemicals; also, some polymers may exhibit cracking; finally, some geosynthetics may exhibit delamination and blistering. These failure modes are discussed below.

2.5.2 Fire

Polymers burn more or less easily, but all can be easily damaged by the heat generated by a fire. A case of geomembrane liner, geonet and geotextile damaged by landfill fire has been reported (*Adams et al.*). Two cases of geofoam fire have been reported (*Horvath*); these fires were caused by welding steel members (unrelated to the geofoam blocks) too close to uncovered geofoam blocks (which were not made of flame-retardant geofoam). The author of this paper knows of a fire that occurred during the installation of a liner system; several rolls of drainage geocomposite (polyethylene geonet core with polyester geotextile filter) caught fire after being struck by lightning, and the fire damaged the installed high density polyethylene (HDPE) geomembrane liner on which the geocomposite rolls were resting prior to being installed.

2.5.3 Heat

Thermoplastic polymers (e.g. polyethylene, polypropylene, polyvinyl chloride) soften when heated and, as a result, deform. This may happen during seaming when "overheating" results in large geomembrane deformations next to seams, which promotes the development of stress concentration. Heat may also affect the degree of crystallinity of polyethylene. As a result, the behavior of polyethylene geomembranes in the vicinity of seams may be affected, which may lead to geomembrane failures next to seams (Giroud, 1994a, 1994b). Geomembranes made of polyvinyl chloride tend to lose plasticizers when heated. As a result, they shrink and may crack. A failure of this type has been reported (Giroud, 1984c, 1984e; *Giroud*). Also, heat promotes thermo-oxidation of some polymers such as polyolefins (polyethylene and polypropylene).

2.5.4 Ultraviolet Radiation

All polymers tend to deteriorate when they are exposed to ultraviolet radiation, i.e. to sunlight. Geosynthetics can be protected against ultraviolet radiation using chemical additives ("antioxidants") and carbon black. The additives, which are included in the polymeric compounds used to manufacture geosynthetics, tend to block the degradation process triggered by the presence of oxygen and the energy supplied by ultraviolet radiation. These additives are progressively consumed, whereas carbon black is inert and remains in the polymeric compound. However, carbon black alone is not sufficient to effectively protect a geosynthetic from ultraviolet radiation. Carbon black used in polymeric compounds is in the form of aggregates of small particles that are mixed with the polymer; these particles stop the ultraviolet radiation, but they have no chemical action. A small percentage of carbon black (e.g. 2 or 3%), used in addition to antioxidants, is sufficient to effectively protect polyethylene and polypropylene geomembranes, geonets and geogrids for many years. In contrast, fibers, because of their small diameter (typically from 20 to 50 μm), cannot be effectively protected by carbon black for the following reasons: (i) the depth of penetration of ultraviolet radiation before it is stopped by carbon black particles is not negligible compared to the fiber diameter (whereas, in the case of geomembranes, geonets and geogrids, the depth of penetration of ultraviolet radiation is small compared to the thickness of the geosynthetic); and (ii) the amount of carbon black in fibers must be limited because the presence of too many aggregates of carbon black particles creates discontinuities that may weaken the fiber. Furthermore, fibers are more exposed to radiation of all kinds than geomembranes, geonets and geogrids, because they have a larger specific surface area (i.e. surface/volume ratio); therefore, the amount of radiation per unit mass of material is greater. As a result, geotextiles cannot be effectively protected against long-term exposure to ultraviolet radiation. Their durability when exposed outdoors depends of the "additive package", which varies from one geotextile to another.

Examples — Holes in Geotextiles Exposed on Landfill Slopes. Geotextiles exposed on landfill slopes have undergone severe degradation to the point that large holes (10 cm in diameter or more) developed (*Artières et al.*, *Giroud et al.*) after being exposed for periods of the order of one month to one year. The extent of the degradation depended significantly on the type of geotextile.

Example — Degradation of "Wrap-Around" Geotextile Facing of a Reinforced Soil Slope. As reported by *Cazzuffi et al.*, a geotextile-reinforced soil slope had to be dismantled because of the degradation of the geotextile at the face of the structure due to exposure to sunlight. The reinforced soil slope was eventually reconstructed using a geogrid.

Example — Degradation of Geomembrane Liner. As reported by *Hsuan et al.*, an exposed flexible polypropylene geomembrane in a pond degraded because the antioxidant package used in this particular flexible polypropylene geomembrane was unable to provide the required protection at the geographical location of the pond, even though the carbon black content was appropriate.

Lessons Learned from the Above Examples. The following lessons can be learned from the above examples:

- Geotextiles should not be permanently exposed to sunlight, and temporary exposure should be limited to an extent that depends on the geotextile.

- Even though geomembranes are more resistant to outdoor exposure than geotextiles, they are all sensitive to some degree to ultraviolet radiation and to heat.

- For permanent exposure of all types of geosynthetics, the design engineer should review the antioxidant package with the geomembrane manufacturer.

2.5.5 Radioactivity

All polymers are sensitive to radioactivity. However, the levels of radioactivity typically encountered, even in the storage of radioactive materials such as low-level radioactive waste, are generally not sufficient to significantly affect geosynthetics (Badu-Tweneboah et al., 1999).

2.5.6 Chemicals

Most synthetic geosynthetics have a high resistance to most chemicals found in geotechnical applications. However, some chemicals found in industrial landfills and ponds may be harmful to some polymers. Also, water and high pH materials such as fresh concrete may cause hydrolysis reactions that can degrade some polymers (e.g. polyester). Also, water causes the bentonite in geosynthetic clay liners to swell and exhibit a marked decrease in shear strength. Examples are given below.

Examples — Degradation of Geomembrane Due to Hydrolysis. An example of degradation of a geomembrane in a pond due to hydrolysis is provided by *Hsuan et al.* Also, the author of this paper knows of a canal liner case where a polyester geotextile degraded as a result of hydrolysis caused by contact with fresh concrete.

Examples — Degradation of Geomembrane Due to Attack by Acid. An example of degradation of a polyvinyl chloride (PVC) geomembrane in a pond due to attack by acid is provided by *Giroud.*

Examples — Liner System Failure Due to Bentonite Hydration. An example of liner system failure by sliding on a slope due to bentonite hydration is provided by *Bonaparte et al.* The hydration of the bentonite significantly reduced the internal shear strength of the geosynthetic clay liner.

2.5.7 Cracking

High density polyethylene (HDPE) may exhibit stress cracking. A number of geomembrane liner failures occurred in the 1985-1995 period (Giroud, 1990; *Thomas and Kolbasuk, Giroud*). In the early 1990s, methods were developed to select the base polyethylene resins used to make HDPE geomembranes, and the frequency of occurrence of failures due to stress cracking has been greatly reduced.

Some geomembranes may exhibit surficial cracking (e.g. geomembranes made with chlorinated polyethylene (CPE) or chlorosulfonated polyethylene (CSPE), and bituminous geomembranes). A failure of this type has been reported by *Kamp and Giroud.*

2.5.8 Delamination and Blistering

Some geomembranes are composed of several plies (i.e. layers) and separation of these plies (called "delamination") may occur. An example is given by *Giroud*. A geomembrane that starts delaminating must be replaced because, if delamination progresses from one side of a geomembrane sheet to the other, it results in a major leak. Delamination occurs when there is insufficient "ply adhesion", i.e. insufficient adhesion between the layers that constitute the geomembrane.

A mode of geomembrane degradation linked to lack of ply adhesion is blistering, which is the development of blisters between two plies of a geomembrane. An example is given by *Kamp and Giroud.*

2.6 Conclusion on Failure Modes Associated with Geosynthetics

The review of modes of failure associated with geosynthetics presented in Section 2 shows that the forces involved are known and the mechanisms are easy to understand. Many of them are similar, or related, to the mechanisms of failure that are familiar to geotechnical engineers. After all, the most common causes of failures of structures incorporating geosynthetics are gravity and water pressure. Also, as a result of cooperation between geotechnical engineers and polymer scientists, the modes of failures that are associated with the polymeric nature of geosynthetics have been identified and geotechnical engineers are becoming aware of these mechanisms. Therefore, one may wonder why there are still failures of structures incorporating geosynthetics. This is addressed in Section 3.

3 SITUATIONS LEADING TO FAILURES

3.1 Overview of Situations Leading to Failures

When confronted with a relatively new technology such as geosynthetic engineering, some engineers are reluctant to use the new technology while others are overly enthusiastic and believe too easily (sometimes encouraged by overzealous salespersons) that the mere fact of using a geosynthetic will automatically solve all problems. The former miss opportunities for better designs, while the latter are potentially dangerous because they may be inclined to neglect consideration of failure modes when they design.

Examples of cases where expectations regarding the geosynthetics were too high and required design considerations were neglected are discussed in Section 3.

Excessive expectations can lead to various types of design mistakes including: (i) design is neglected based on the belief that the geosynthetic will solve the problem regardless of the design (see Section 3.2); (ii) basic design steps that are usual in geotechnical engineering are omitted as if there was no relationship between geosynthetic engineering and geotechnical engineering (see Section 3.3); (iii) the fact that geosynthetics may have a detrimental effect on the structure is ignored in design, whether the geosynthetic is useless (see Section 3.4) or useful (see Section 3.5); (iv) using two geosynthetics instead of one may have detrimental consequences (see Section 3.6); (v) all geosynthetics are considered equal (see Section 3.7), which is wrong, but consistent with the fact that they are all assumed to make miracles; and (vi) even when it has been shown that a failure may happen, those who are overly enthusiastic about geosynthetics may not take the failure potential seriously, or those who have been trained to believe only in precedents do not believe the failure may happen if the same type of failure has not happened before (see Section 3.8).

3.2 The Use of a Geosynthetic Does Not Replace an Adequate Design

As indicated in Section 3.1, some overly enthusiastic users of geosynthetics tend to act as if they were thinking that the mere fact of using a geosynthetic will automatically solve all problems. As a result, they may underestimate the need for design of geosynthetic applications. Examples and case histories follow.

Example — Insufficient Reinforcement in a Reinforced-Soil Wall. Failures of reinforced-soil walls have occurred due to insufficient amount and/or length of reinforcement (*Watn*, *Al Hussaini*).

Case History — Use of a Geocomposite Drain in a Drainage System that Was Undersized. Clearly, a geocomposite drain cannot work when used instead of a conventional granular drain in a drainage system that was destined to fail, regardless of the type of drain (conventional or geosynthetic), due to its inadequate overall design (*Sprague et al.*).

Case History — Insufficiently Strong Geosynthetics Used for Erosion Control in a Torrent Bed. Various erosion control systems including several types of geosynthetics (geotextiles, geotextile-geonet geocomposites, geotextile-geomat geocomposites, geocells) were used, instead of the traditional solution which consists of using large rocks or concrete blocks, to control erosion in several reaches of a mountain torrent bed. The erosion control systems and the geosynthetics were severely damaged by the flow of water. This type of failure is not surprising considering the magnitude of the drag forces developed by water flow during the short periods of time when the torrent is full (i.e. forces that can displace large rocks). Therefore, it was predictable that erosion control systems, which did not include heavy elements such as rocks or blocks, would not resist (*Jacotot et al.*).

Case History — Use of Geomembrane Tubes for Coastal Protection. A case has been reported where large sand-filled geotextile tubes were used for coastal protection and failed (*Richardson*). In this case again, the expectations regarding the geosynthetic were too high considering the magnitude of subgrade erosion resulting from the forces associated with ocean waves.

Case History — Geomembrane Liner on a Soil with a High Carbonate Content. A single geomembrane liner was installed in a large (206 × 98 m), 3-m deep pond to contain phosphoric acid. The pond was located in a desert. Eleven months after the first filling, the liner burst over a 1-m diameter solution cavity (Giroud and Fluet, 1986; Giroud et al., 1990a; Giroud, 1993). Acid leaking through a defect in the geomembrane had attacked the high carbonate content subgrade soil. The owner insisted that the soil cavities be repaired (there were several cavities, each caused by a leak in the geomembrane) and the geomembrane replaced by an identical geomembrane, so the pond would be reconstructed as initially designed. The owner's insistence resulted from his belief that the only problem was a deficiency in the geomembrane liner, based on the project specifications which indicated that the "pond will be lined with a flexible, impermeable membrane to prevent seepage".

The consulting engineer hired to investigate the problem (the author of this paper) had to convince the owner that the same accident would occur again because it is not possible to install a geomembrane over a large area without any defect. The consulting engineer indicated that the problem resulted essentially from a conceptual mistake: acid should not be impounded over a soil with a high carbonate content using a single liner. Finally, the owner agreed that the conceptual design be revised. Answering questions from the consulting engineer, the owner indicated that the pond had two functions: storage and evaporation. The consulting engineer then recommended that the two functions be separated. Accordingly, the large pond was replaced by three smaller ponds that occupied the same total footprint:

- one storage pond, with a small surface area, but rather deep (6 m), with a double geomembrane liner and a leakage detection/collection layer between the two liners; and

- two twin evaporation ponds, large and very shallow (0.5 m of liquid), with a single geomembrane liner.

The concept of the adopted solution is a follows:

- large, shallow ponds promote evaporation;

- considering that the rate of leakage through a geomembrane defect under a liquid depth of only 0.5 m is expected to be small and that leakage would not cause water contamination because there was no ground water under the ponds, the use of a double liner for the evaporation ponds was not justified;

- comparing liquid levels in the twin evaporation ponds provides a means to rapidly detect if one of the two ponds is leaking; this pond would then be repaired while the other is still used;

- a double liner was justified in the storage pond because a large rate of leakage through a geomembrane defect could be expected under a depth of liquid of 6 m, which would cause the development of a cavity in the soil; and

- the soil excavated to construct the storage pond was used as backfill under the evaporation ponds, which made it possible to repair the subgrade soil which had been damaged in many areas by solution cavities.

It is noteworthy that calculations performed during the investigation using the method developed by Giroud (1981, 1982a) showed an excellent correlation between the depth of liquid at the time of failure, the density of liquid, the diameter of the solution cavity, and the tensile strength of the geomembrane. This showed that the failure could be explained rationally.

Case History — Geomembrane Liner on Karstic Soil. The geomembrane used as a single liner for a reservoir burst as a result of a karstic collapse in the supporting soil (Giroud and Goldstein, 1982; Giroud, 1993). The investigation showed that the karstic collapse had been provoked by intrusion of water into the ground resulting from leakage at a defective connection between the geomembrane and a concrete water intake tower. The owner hastily concluded that a geomembrane liner could not provide a reliable solution and decided to reline the reservoir with bituminous concrete reinforced with steel strips.

This solution seemed to be based on common sense, but the consulting engineer hired to investigate the failure realized that the problem was due to a defective conceptual design: a simple geologic exploration at the design stage would have shown that the reservoir was founded on a karstic terrain and that any intrusion of water into the ground would cause the soil to collapse. The consulting engineer then convinced the owner that, because of the permeability of bituminous concrete, there would be some intrusion of water into the ground and a new karstic collapse would occur. Therefore, the consulting engineer recommended that a double liner be used. This was achieved by using the bituminous concrete with steel reinforcement as the secondary liner and a geomembrane as the primary liner, with an intermediate leakage detection/collection layer made of gravel stabilized with bitumen. It is interesting to note that the owner was comfortable with the use of steel-reinforced bituminous concrete (i.e. a "reinforced traditional solution") because of the prevailing belief that "stronger is better". In contrast, the designer was comfortable with the solution that addressed the problem, the double liner.

Lessons Learned from the Above Examples and Case Histories. The following lessons can be learned from the above examples and case histories:

- Geosynthetics do not make miracles and should not be expected to make miracles. A design that relies on unrealistic expectations about geosynthetic performance is flawed and could lead to failures.

- The geosynthetic should not be automatically be considered as the main culprit if a failure occurs. Furthermore, the geosynthetic should not be used as a scapegoat when the design is flawed.

- A failure may provide an opportunity to improve a flawed design. Even if entitled to do so under the contract, the owner should not demand that the facility be reconstructed as initially designed until the design is evaluated to determine if it is flawed. Also, it is inappropriate, when a structure incorporating geosynthetics fails, to reject geosynthetics systematically and to demand that the problem be solved using a traditional design solution. It is worth noting that failures involving geosynthetics are usually successfully repaired using geosynthetics.

- Common sense often leads one to believe that traditional solutions are safer than solutions involving new materials such as geosynthetics. Those who think so fail to recognize that there is no general answer to this question. Only a rational analysis on a case by case basis may provide a valid answer. There are cases where it is possible to show that a geosynthetic solution is safer than a traditional solution, and vice versa. Once more, common sense can be wrong and is not a substitute for good design.

- Observed failures can always be rationally explained if appropriate data have been collected during the investigation. Failures provide an opportunity to calibrate design methods. When such calibration is done, it appears generally that available methods for designing geosynthetics applications are satisfactory. Design engineers are encouraged, therefore, to believe the results of their analyses, especially when these results lead to predicting a failure.

- The worst situation for a design engineer, if a failure occurs, is finding that any expert could explain the failure rationally using a method that was available at the time the project was designed.

3.3 Design Steps that Are Usual in Geotechnical Engineering Should Not Be Omitted

Failures occur when designers focus exclusively on the design of the geosynthetic aspects of an application and omit the more usual aspects of the design, i.e. the design aspects that involve geotechnical engineering. This may happen, in particular, if a structure is redesigned to replace an original structure component that did not include geosynthetics by a component with geosynthetics. In such cases, it is possible that additional geotechnical design is required (whether the original design is deficient or not). Some examples and case histories are presented below.

Case History — Insufficient Site Investigation. The failure of a portion of riverbank protection system incorporating a geotextile filter was due to the biological clogging of the geotextile (*Davis et al.*). The clogging of the geotextile caused pore water pressure buildup under the geotextile, which resulted in localized instability of the bank protection system. The investigation showed that, in the portion of the bank protection system where the failure occurred, water with a high organic content was seeping out of the bank. This water originated in a wastewater pond that was located on top of the bank. The potential for the wastewater to infiltrate into the ground and to daylight along the riverbank had not been recognized due to an insufficient site investigation. In this case basic geotechnical engineering was neglected, and it is likely that a traditional sand filter would have failed as the geotextile filter did.

Case History — Insufficient Site Investigation. The failure of a modular-faced reinforced soil wall illustrates the detrimental impact of insufficient site investigation (*Bernardi*). The only section of the wall that collapsed was located next to a street with inadequate surface drainage. Furthermore, at that location, the soil profile was different from expected and, as a result, the earth pressure on the lowest portion of the wall was greater than expected. These two important features of the site, the surface drainage and the soil profile, would have been identified if a thorough site investigation had been performed.

Case History — Deep-Seated Failure of an Ore Heap. In an ore heap leach pad, a deep-seated slide occurred, involving movements of the ore and the foundation soil (Van Zyl, 1993). In that case, the design engineer had considered only slip surfaces within the ore, or along the geomembrane, or in the foundation soil at shallow depth. Because of inadequate sub-soil investigation, the design engineer had not realized that the critical slip surface could be deep-seated.

Case History — Foundation Failure of the Modular Facing of a Reinforced-Soil Wall. The collapse of a section of the modular facing of a reinforced-soil wall was caused by erosion of the soil at the toe of the facing (*Gourc et al.*). Clearly, the failure was a bearing capacity failure. Indeed, the reinforced-soil mass remained stable after the collapse of the facing.

Examples — Stability of Geosynthetic-Reinforced Soil Walls. A geosynthetic-reinforced soil wall can collapse as a result of several mechanisms: (i) internal instability of the reinforced soil mass; (ii) external instability of the soil mass acting as a block (which includes sliding, overturning, and bearing capacity failure); (iii) global (also called overall) instability which corresponds to an instability of the entire mass of (generally) natural soil where the geosynthetic-reinforced soil wall is located, i.e. instability along a failure surface (sometimes referred to as "deep-seated) that passes behind and beneath the reinforced soil mass; and (iv) compound instability which corresponds to an instability that involves partly the natural soil and partly the reinforced soil mass, i.e. instability along a failure surface that passes partly through the soil and partly through the reinforced soil mass. Factors of safety for all of these mechanisms must be calculated. Several cases of failures of geosynthetic-reinforced soil walls have been reported where the failure was due to global or compound instability, aspects of the design that had been omitted by the designer who focused on the internal and external stability of the reinforced soil wall (*Berg and Meyers, Valentine and Damm, Giroud and Beech*). In other words, the designer focused on the geosynthetic aspect of the design (i.e. the design aspects involving only the geosynthetic-reinforced soil) and not on the design aspects involving the soil (i.e. the geotechnical aspect of the design). As pointed out by *Berg and Meyers*, this may happen in particular if there are two designers for the same project, one in charge of the reinforced soil structure, the other in charge of geotechnical engineering, without good communication between the two designers and, especially, without a clear understanding of who is in charge of the compound stability, which is a hybrid case.

Lessons Learned from the Above Examples and Case Histories. The following lessons can be learned from the above examples and case histories:

- Usual design steps should not be omitted from the design of a structures incorporating geosynthetics. These usual design steps include, for example: site investigation for all types of structures, in particular to identify sources of water and soil profile; deep-seated failure analysis for embankments and ore heaps; and global stability and bearing capacity for a reinforced-soil structure.

- Communication between team members is essential.

Case History — Global Stability of Reinforced-Soil Wall. A quay wall constructed with geogrid-reinforced clay and faced with prefabricated full-height concrete panels exhibited excessive bulging after intensive rainfalls (*Giroud and Beech*). The investigation showed that the bulging had two causes, which were not directly related to the geogrid reinforcement: (i) global stability of the reinforced soil mass was decreased by pore water pressure resulting from water intrusion in deep desiccation cracks located behind the reinforced mass, a mechanism that was not properly accounted for in the stability analysis; and (ii) hydrostatic pressure was applied directly on the concrete panels by water that accumulated in a sand drain located behind the panels because the sand was not permeable enough and because the drain (which was only intended to collect water seeping from the clay fill) was saturated by runoff water due to inadequate surface drainage.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- At the design stage, all possible mechanisms of failure must be considered. The presence of a geosynthetic generally does not eliminate the need for evaluating traditional mechanisms of failure, such as global stability.

- Water is often a cause of instability in geotechnical engineering, and structures incorporating geosynthetics are affected by water like traditional geotechnical structures.

Case Histories — Geogrid-Reinforced Soil Walls. The collapse or near-collapse (one of the walls was demolished before it collapsed) of two geogrid-reinforced soil walls (9 and 6 m high) constructed with cohesive fill has been reported by *Frost et al.* The investigation shows that the failures were caused by the decrease in strength of the cohesive fill due to: (i) poor surface drainage that resulted in a large amount of water infiltrating into the fill; and (ii) the fact that the fill had been placed 4 percentage points dry of the optimum, which made it very sensitive to an increase in water content (resulting in settlement and loss of strength). Also, the lateral deformation of the facing had not been properly evaluated and the wrap around facing came in contact with the protective non-structural facing during construction (see Section 2.4.5). Furthermore, at one of the two walls: (i) the fill was poorly compacted, which further decreased the fill strength; (ii) the top geogrid was missing; (iii) the sloping bedrock at the bottom of the excavation was not as planned due to insufficient soil investigation and, as a result, the placement of the bottom layers of geogrid was inconsistent with the construction drawings; and (iv) a major slide of the excavated slope occurred during construction, which led to an accumulation of non-compacted debris at the bottom of the excavation contributed to the large deformation of the structure once water infiltrated the backfill. Clearly, what was constructed did not reflect what was designed and an engineer should have been present during construction.

Lessons Learned from the Above Case Histories. The following lessons can be learned from the above case histories:

- As pointed out by *Frost et al.*, a geosynthetic-reinforced soil structure should not be regarded as a pre-fabricated system that is simply inserted in the field without consideration of the site conditions.

- As in traditional geotechnical structures, the use of cohesive fill requires certain precautions during placement.

- The use of cohesive fill in a geosynthetic-reinforced soil structure requires careful consideration of the potentially detrimental effects of water and pore water pressure within the fill. Also, potential sources of water likely to reach the reinforced-soil mass should be identified.

- An evaluation of the constructibility of a structure is an important part of design, and measures must be taken at the design stage to ensure that construction will proceed smoothly. Also, at the design stage, an effort should be made to foresee potential construction problems and propose tentative solutions.

- Insufficient site investigation leads to construction problems.

- Construction quality control and quality assurance are important in the case of critical structures.

Case History — Geomembrane Liner in a Dam Reservoir. A large reservoir lined with a geomembrane was used for the water supply of a winter resort (Giroud, 1993). In the winter, the reservoir emptied and water had to be transported by trucks to the resort at great expense. Due to snow, field investigation was not possible until June. By that time, most people involved were convinced that failure of the geomembrane was responsible for the observed problem, as common sense indicated that, if water is leaking, the cause must be a defective liner. Accordingly, the general feeling was that the geomembrane supplier/installer was liable.

At the insistence of the geomembrane supplier/installer, a thorough investigation was conducted. The investigation showed that the geomembrane had defects, but also showed that the breach in the geomembrane that emptied the reservoir had been caused by a depression of the soil supporting the geomembrane due to the collapse of a drainage pipe.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- Usual design steps (e.g. checking the strength of drainage pipes) should not be omitted.
- Geosynthetics should not automatically be blamed for all problems in a project, even if they caused some of them.
- Common sense is afraid of novelty and, therefore, leads to using geosynthetics as scapegoats when there are problems.
- Failure investigations must be thorough in order to have a chance of finding all the causes.

3.4 Useless Geosynthetics May Have a Detrimental Impact on the Structure

Sometimes geosynthetics are used in structures where they are not needed. This happens when overzealous salespersons manage to convince designers or contractors to use a geosynthetic they do not need, or when an overly enthusiastic design engineer specifies a geosynthetic "as an extra precaution", assuming that a geosynthetic can only add to the performance of the structure. It is important to recognize that, even though a geosynthetic is useless, it may have a detrimental effect on the performance of a structure. Examples are given below.

Case History — Earth Slide Caused by Additional Geosynthetic. During the design of a waste disposal landfill, a simple, but correct, analysis had convinced the design engineer that a layer of sand, which was to be placed on the smooth geomembrane liner on the landfill side slope, would be unstable. To improve stability, the design engineer decided to place a needle-punched nonwoven geotextile between the geomembrane and the sand. Common sense indicated that, since high friction angles are generally obtained with needle-punched nonwoven geotextiles, the incorporation of such a geotextile in the system could only improve the stability of the slope.

In reality, the geomembrane-sand interface friction angle was less than the geotextile-sand interface friction angle (which seemed to support the above approach based on common sense), but was greater than the geotextile-geomembrane interface friction angle. The design engineer had failed to recognize that the critical slip surface would be located between the geomembrane and the additional geotextile. The slope was constructed and failed with a slip surface at the geotextile-geomembrane interface.

Clearly the addition of the geotextile should have been followed by a new design effort where the new conditions created by the addition of the geotextile would have been reviewed.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- The friction angle (and, more generally, the shear strength) at the interface between two materials depends on both materials. It is not an intrinsic property of one material.

- Adding a geosynthetic may be detrimental. In the design of all structures incorporating geosynthetics, potential detrimental effects of geosynthetics must be considered. If such effects appear likely, they must be quantified.

- Simple failure mechanisms can be predicted easily by a geotechnical engineer using rational analyses, not common sense.

Case History — Flow Capacity Reduction due to Additional Geotextile. The leachate collection system in a waste disposal landfill consisted of a gravel layer on the floor of the landfill and a geonet on the side slope. A needle-punched nonwoven geotextile cushion was specified between the gravel and the geomembrane liner, but was not needed between the geonet and the geomembrane liner. The geotextile was shown on construction drawings to extent "0.3 m minimum" beyond the gravel-geonet junction. In the area where the geotextile was to be in contact with the geonet (i.e. approximately 0.5 m beyond the gravel-geonet junction), a double layer of geonet was rightfully shown in the construction drawings. This double geonet layer was intended to maintain sufficient hydraulic transmissivity in spite of the fact that the geotextile would intrude into the geonet (see Section 2.4.3). The contractor had too much geotextile on the site and decided to use a geotextile that extended more than 1 m beyond the gravel-geonet junction. Certainly, this met the letter of the "0.3 m minimum" specification. Furthermore, the contractor, using common sense, deemed he had improved the leachate collection system by installing a geotextile larger than required. As a result, however, a portion of the geotextile was in contact with the geonet in an area where only a single layer of geonet was used. Intrusion of the geotextile into the geonet channels would have decreased the geonet's hydraulic transmissivity, hence decreasing the flow capacity of the leachate collection system. The problem was discovered, and solved, at the last minute by the design engineer as he visited the site (Giroud, 1993).

Case History — Potential Clogging due to Additional Geotextile. The following case of a potential (and quasi certain) failure has been reported by Giroud (1993). A perforated drainage pipe was to be placed in a drainage trench filled with gravel. An engineer (the author of this paper) visiting the site during construction realized that a drainage pipe wrapped with a geotextile had been placed in the trench. The contractor told the engineer that the perforated pipe had been delivered with the geotextile and that the presence of a geotextile "filter" could only enhance the functioning of the drain. Indeed, common sense dictates that a drain with a filter is better than a drain without a filter. The engineer explained that, in contrast, the geotextile was useless and even detrimental: (i) useless, because the gravel size was greater than the size of the pipe perforations and, therefore, it was not necessary to use a filter to prevent the gravel from entering the pipe; and (ii) detrimental, because the gravel was dirty (i.e. covered with fine particles) and the geotextile would get clogged rapidly when it stops fine particles carried by the water being drained. The engineer explained that the goal of a filter in geotechnical engineering is not to stop particles that are moving but to prevent particles from moving. He explained that any filter (sand, geotextile, or other) gets clogged if it is placed at a location where it has to stop particles (which is how an air filter works, but an air filter is periodically replaced, which is not possible for a filter buried in the soil). The engineer recommended that the drain be dismantled, the geotextile removed from the pipes, the gravel washed and replaced, and the pipe replaced without the geotextile, which was done.

Lessons Learned from the Above Case Histories. The following lessons can be learned from the above case histories:

- Common sense, which indicates that adding a geosynthetic can only improve the performance of a structure, is wrong. An additional geosynthetic can be detrimental. Similarly, extending a geosynthetic beyond the area intended by the design engineer may cause a serious problem, because a geosynthetic may be beneficial in one area (where it is needed and specified) and detrimental in another area.

- Common sense, which indicates that adding a filter can only improve a drain is wrong. There are cases where a filter can be detrimental.

- Specifications must be as precise as possible. Ideally, engineers who prepare specifications should address problems that would result from potential misinterpretation of the specifications during construction. For example, there are cases where a minimum and a maximum geosynthetic length, width, or overlap should be specified, not only a minimum. In some cases, it is recommended to include in the specifications a warning against certain specific uses of a geosynthetic if experience dictates that, in the considered applications, contractors are tempted to use an extra geosynthetic, not knowing that it can be detrimental.

- Installers should be instructed not to place extra geosynthetics as they may cause problems such as clogging of drainage systems and slip surfaces. Installers should be instructed not to add a geosynthetic or extend the area covered with a geosynthetic during construction without being authorized by the design engineer.

- Design engineers should visit construction sites to learn about typical mistakes made during construction. This helps them write better specifications.

- The presence of the design engineer at the construction site may help prevent mistakes that could lead to failures.

3.5 Useful Geosynthetics May Have a Detrimental Impact on the Structure

It is important to recognize that a geosynthetic, even when it is useful and performs its intended function, may have a detrimental effect on the performance of a structure. This may happen as part of the function performed by the geosynthetic (e.g. the geosynthetic may require some soil deformation to mobilize its strength and it happens that this soil deformation has some detrimental impact on the structure) or because the geosynthetic happens to perform an additional function which was not recognized by the design engineer. It is, therefore, important to identify all functions performed by the geosynthetic, and understand the requirements for the geosynthetic to perform these functions. Examples and a case history illustrate this point.

Examples — Waste Slides. Several examples of waste slides in landfills equipped with a geosynthetic liner system were mentioned in Section 2.2.3 (*Chang et al., Ouvry et al., Stark et al.).* In all these slides, a key role was played by low interface shear strength associated with the presence of geosynthetics. Clearly, the presence of geosynthetics in a waste mass (which is essential to the liner system) often increases the risk of instability.

Examples — Geosynthetic Blocking Ground Water Drainage. Geomembranes and geosynthetic clay liners are used as liners because they have a low permeability. However, due to this low permeability, they may block natural drainage paths. As a result, pore pressure may develop under the liner, causing the liner to be uplifted (*Datye and Gore*) or the slope to become globally unstable (*Bonaparte et al.*). In the same category belongs the classic case of a geotextile roadway separator that is not sufficiently permeable and retains water beneath it, leading to failure. Clearly, geotextile separators must be designed for performing the secondary function of filtration.

Examples — Failures Caused by Geosynthetic Transmissivity. The two case histories presented in Section 2.2.7 show that a failure can be caused by water or air conveyed by a geosynthetic while it performs the intended function (which is not related to fluid transmission). In other words, a geosynthetic, while it performs the function it is supposed to perform, may perform other functions which may be detrimental to the structure.

Lessons Learned from the Above Examples. The following lessons can be learned from the above examples:

- A geosynthetic, even though it performs a useful function, may have a detrimental impact on the performance of a structure: it can impair its stability, convey undesirable water, etc.

- It is important to understand both the characteristics of geosynthetics that can affect the performance of a structure (e.g. interface shear strength, low-permeability, tensile modulus, hydraulic transmissivity) and the geotechnical mechanisms that can be mobilized by the geosynthetics (e.g. pore pressure buildup, deformation, water flow).

Case History — Geotextile in the Construction of an Airport Taxiway. New taxiways were being constructed in an airport. According to the design, a geotextile had been placed between the soft subgrade and the aggregate base. A thick concrete slab would then be built on top of the aggregate base. Prior to placing the concrete slab, a certain area of the aggregate base was used as an access road for the trucks. Some rutting developed and aggregate was added several times to fill the ruts.

Prior to constructing the concrete slab, a grader was used to level the aggregate surface at the design grade. In the area that had been used as an access road, the grader had to remove excess aggregate. In doing so, the grader's blade cut the geotextile in places where it had moved up between the ruts. (The upward movement of the geotextile had remained unnoticed by the contractor, because the geotextile had remained covered with the added aggregate.) The contractor did not understand what was happening because the geotextile supplier had stated that the geotextile would "reinforce" the aggregate base.

The contractor was dissatisfied with the performance of the geotextile, but the consulting engineer in charge of the investigation (the author of this paper) told him he was lucky, because if the problem had not been discovered (thanks to the grader's blade) the aggregate base under the concrete slab would have had a non-uniform thickness, which could have caused cracking of the slab. Repair was simple: in all areas where the aggregate base had been used as an access road, the aggregate base and the geotextile were removed, the foundation soil was graded, and, finally, a new geotextile and a new aggregate base were placed.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- Unsupported claims on geosynthetic performance can lead to wrong expectations.

- It is important to identify the function of the geosynthetic, and to understand the mechanisms involved when the geosynthetic performs its function. If a geosynthetic must perform two functions, one during construction (e.g. reinforcement), the other in service (e.g. separation), this may cause problems that the designer engineer must foresee and solve at the design stage.

3.6 Geosynthetic Redundancy May Have a Detrimental Impact on the Structure

Those who are overly enthusiastic about geosynthetics may believe that adding a geosynthetic to an existing structure may only improve the structure. This is not always the case as illustrated by the following case history. This case history shows that two liners are not necessarily better than one, contrary to what common sense would dictate.

Case History — Geomembrane Uplift Due to Liner Redundancy. A reservoir waterproofed with a bituminous roofing membrane had contained brine for years (Giroud, 1993; *Giroud*). A limited amount of leakage occurred, causing ground water pollution. Instead of repairing the bituminous roofing membrane, it was proposed to reline the pond with a geomembrane placed directly over the existing bituminous roofing membrane. A consulting engineer (the author of this paper), working on another part of the project, warned the geomembrane installer in writing that the geomembrane could be uplifted in case of rapid drawdown of the reservoir, a mechanism that geotechnical engineers are accustomed to consider. The consulting engineer indicated that, to prevent geomembrane uplift, it was necessary either to place a drainage system between the two liners or to place a load on the geomembrane (e.g. soil layer, concrete). The installer considered that the consulting engineer was too pessimistic and preferred to listen to common sense, which dictated that two liners were certainly better than one. Therefore, he placed the geomembrane directly on top of the bituminous roofing membrane, and did not place a load on the geomembrane or a drainage layer between the geomembrane and the bituminous roofing membrane.

The reservoir was part of a pumping station and its level fluctuated every day. However, it was never completely empty and, as a result of the lack of transparency of the brine, it was not possible to observe the condition of the portion of the geomembrane located below the brine level. A few months after the geomembrane installation, a geomembrane bubble appeared at the brine surface. The consulting engineer was called to investigate the problem. He requested that the reservoir be emptied. When the reservoir was empty, it appeared that a considerable amount of brine had accumulated between the bituminous roofing membrane and the geomembrane. There was some gas on top of the entrapped brine, which explained the geomembrane bubble. The investigation showed that the geomembrane had defects, which had caused the brine to leak and accumulate between the geomembrane and the bituminous roofing membrane (which was impervious enough to retain most of the brine leaking through the geomembrane).

The presence of brine between the geomembrane and the bituminous roofing membrane was unacceptable for two reasons: (i) the entrapped brine was bound to progressively leak through the bituminous roofing membrane and pollute the ground water; and (ii) each time the level of brine in the reservoir fluctuated, the entrapped brine moved, thereby inducing tensile stresses in the geomembrane. Repair was done as follows: (i) the geomembrane was removed; (ii) a thick needle-punched nonwoven geotextile was placed on the bituminous roofing membrane to be used as a drain and was connected to an outlet; and (iii) a new geomembrane was placed on the geotextile.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- Adding a geosynthetic to an existing structure without reviewing the design may cause problems.
- Common sense, which indicates that two are always better than one, is wrong. Two liners may be better than one only if precautions have been taken to prevent uplift of the upper liner.
- Geosynthetics are construction materials like soil, and failure mechanisms (such as uplift of materials in case of rapid drawdown) which are known in geotechnical engineering can occur with geosynthetics.
- Geotechnical engineer who use rational analyses, not common sense, can relatively easily predict simple failure mechanisms, i.e. those that stem directly from principles of physics such as pressure balance.

3.7 Not All Geosynthetics Are Equal

Not all geosynthetics are equal. Geosynthetics of the same type are not necessarily equal (i.e. two geonets are not necessarily equal). Even when geosynthetics appear similar, they may have different properties. Considering that "all geosynthetics are equal" leads to two types of problems: (i) a geosynthetic that is not equivalent to the specified geosynthetic may be considered equivalent; and (ii) instead of measuring the properties of a considered geosynthetic, properties are "borrowed" from another geosynthetic considered equivalent, although it is not.

Often, a certain geosynthetic is specified for a project, and the specifications indicate that an equivalent geosynthetic can be used. The specifications should indicate that, if the "equivalent" geosynthetic does not have properties (obtained from tests performed by an accredited laboratory) that are *identical* to those of the specified geosynthetic (with a small tolerance), an equivalency demonstration (including laboratory tests) should be provided. Failures have been reported that are due to the fact that the "equivalent" geosynthetic that was selected was not actually equivalent.

Considering that all geosynthetics are equal leads to using published properties instead of conducting project specific tests. This approach is dangerous, because there may be significant differences between materials that look similar. For example, an error of only a few degrees on the interface friction angle may cause slope instability, which has happened in several cases. Also, assuming on the basis of common sense that the interface friction angle is always greater with a textured geomembrane than with a smooth geomembrane can be wrong. Rubbing the hand against a geomembrane gives some indication of the friction at the hand-geomembrane interface, but gives no indication of the friction at the interface between the geomembrane and the adjacent material in the field. This was confirmed by a full-scale test and related laboratory tests (Giroud et al., 1990b, 1990c) where a geonet had a greater interface shear strength with a smooth geomembrane than with a textured geomembrane.

Example — Biological Clogging of a Geotextile Filter in a Riverbank Protection System. The failure of a riverbank protection system incorporating a geotextile filter due to the biological clogging of the geotextile was described in a case history presented in Section 3.3 (*Davis et al.*). The specified geotextile filter had been replaced by an "equivalent" geotextile that was significantly more susceptible to biological clogging than the specified geotextile. Although a major mistake in that project was the insufficient site investigation as pointed out in Section 3.3, the replacement of the specified geotextile by another geotextile contributed to the failure.

Example — Haul Road where the Specified Geogrid was Replaced by a Geotextile. The failure of a haul road where the specified geogrid was replaced by a geotextile has been reported (*Bostian et al.*). In fact, in that case, the haul road would have failed with any geosynthetic approximately equivalent to the specified geogrid, because the design was inadequate.

Example — Waste Slide in a Landfill. A large waste slide occurred in a landfill where the interface shear strength was overestimated based on published values (*Ouvry et al.*).

Examples — Liner System Slide in a Landfill. During the design of several waste disposal landfills, the interface friction angle between adjacent geosynthetics was estimated on the basis of published test data or results of tests on similar geosynthetics conducted for earlier projects. In one of these landfills, a major slide of the liner system occurred near the end of construction (Giroud, 1993).

Lessons Learned from the Above Examples. The following lessons can be learned from the above examples:

- Geosynthetic properties should not be estimated, they should be measured. This is particularly true in the case of interface friction angles, because this property, which often plays a critical role in the performance of structures, is difficult to estimate. Furthermore, a slight error on the interface friction angle may lead to the belief that a slope is stable when it is not.

- As more and more geosynthetics are available on the market, the probability increases that there are significant differences in properties between geosynthetics that may appear to be identical.

Case History — Dewatering System where the Specified Nonwoven Geotextile was Replaced by a Woven Geotextile. Two different drains were needed in the dewatering system for the foundation of a building (*Christopher*). The design engineer had rightfully specified two different geotextile filters because the soils in contact with each drain were different. The specified geotextile filters were: a filter with relatively large openings (a monofilament woven geotextile) for the first drain (a drainage trench) that was in contact with a sand; and a filter with relatively small openings (a needle-punched nonwoven geotextile) for the second drain (a horizontal drainage layer) that was in contact with a sandy silt. The contractor had ordered too much woven geotextile for the first drain. When construction of the first drain was completed, the contractor decided to use the remaining woven geotextile for the second drain, although a nonwoven geotextile filter had been specified for that drain. The dewatering system progressively failed because the woven geotextile allowed fine soil particles to pass due to its excessively large opening size for this type of soil. Also, the contractor used one pump instead of the three specified. As a result of insufficient dewatering, the sandy silt became soft and deformed under the weight of construction equipment, which tended to separate some of the geotextile overlaps. Furthermore, some of the overlaps were separated by construction equipment pushing aggregate on top of the geotextile in the wrong direction. Due to the separation of some geotextile overlaps, more silt particles passed through the geotextile. The aggregate contaminated with silt particles and the woven geotextile were removed, and were replaced by the specified nonwoven geotextile and clean aggregate. After that (and adding the two missing pumps), the dewatering system performed satisfactorily.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- Properly selected geotextile filters are required for satisfactory performance of drainage systems.
- Contractors should not replace a specified geotextile by another geotextile and should not ignore instructions regarding geosynthetic installation.
- The design engineer should educate contractors on geosynthetic construction requirements which are most likely unfamiliar to them.

3.8 Disbelief in Potential Failure

It is hard to believe that a mode of failure that never happened in the past could happen, even if this mode of failure is predicted using a rational analysis. Indeed, geotechnical engineers are accustomed to learn from precedents. However, it should be noted that, while such an attitude may be justified in a relatively old discipline such as geotechnical engineering, it is not appropriate in a relatively new discipline such as geosynthetic engineering. This is illustrated by the following case history.

Case History — Geomembrane Liner in an Underground Reservoir. In 1980, the design engineer for a deep (20 m) rectangular reservoir (*Giroud and Stone*) concluded, from an analysis of the stress-strain curve of the selected geomembrane (a high density polyethylene geomembrane), that this geomembrane would fail if it were installed as planned. The rationale presented by the design engineer, orally and in writing, can be summarized as follows:

- The geomembrane stress-strain curve obtained in a tensile test has a yield peak at a rather small strain (of the order of 10%) compared to the strain at break, which is of the order of 1000%.
- As the reservoir is filled, tensile stresses in the geomembrane will increase.

- It can be predicted that the distribution of tensile stresses in the geomembrane will not be perfectly uniform, in particular because some irregularities of geomembrane thickness cannot be avoided and because of the curved shape of the geomembrane in the corners of the reservoir.

- As a result of the non-uniform stress distribution, the tensile stress will reach the peak value, at a certain Cross Section A of the geomembrane, while, in the rest of the geomembrane, the tensile stress will have a smaller value.

- As soon as the tensile stress at Cross Section A reaches the peak value, the strain at Cross Section A increases suddenly.

- It appears that a small difference between the tensile stresses at Cross Section A and at other cross sections in the geomembrane (which can be close to Cross Section A) causes a significant difference in strain: the strain at Cross Section A is of the order of 100 to 1000% while the strain in the rest of the geomembrane is less than 10%.

- As a result of its very large strain at Cross Section A, the geomembrane becomes very thin at that location and bursts under the liquid pressure. The geomembrane bursts only in a small area (because the geomembrane strain is small at cross sections other than Cross Section A), but this is sufficient to cause significant leakage.

The design engineer thus described a mode of failure he had never observed before. This mode of failure had not been observed either by the geomembrane manufacturer/installer, in spite of his experience. Furthermore, the geomembrane manufacturer/installer insisted that common sense dictated that, since the geomembrane breaks at a 1000% strain in a tensile test, it would not break in the reservoir where the strain had no chance to reach such a high value. Everyone in the project team, including the owner, disagreed with the design engineer. The geomembrane manufacturer/installer convinced the owner that the prediction by the design engineer was just an academic exercise that had nothing to do with reality. The design engineer insisted his analysis was rational, but he had to acknowledge that there were no facts to prove that the failure prediction derived from the analysis was correct since the predicted mode of failure had never been observed before. As a result, the design engineer could not convince the other members of the team (in part because he, too, respected experience based on precedents and, therefore, was not fully convinced himself by the prediction he had made).

The design engineer finally concluded that he was overly pessimistic, and the measures he had recommended were not taken. The geomembrane liner was installed in 1981 and failed during the first filling as predicted. The same mode of failure has occurred since then in other reservoirs.

A quantitative analysis of the mode of failure described above has been published in a paper (Giroud, 1984d, 1984e) which established and disseminated the concept that geomembranes with a yield peak on their stress-strain curve can only be used in situations where their strain is less than the yield strain, which is approximately 50 to 100 times less than the strain at break measured in a uniaxial tensile test.

Lessons Learned from the Above Case History. The following lessons can be learned from the above case history:

- In a relatively new discipline such as geosynthetics, the fact that a certain mode of failure did not occur before is of limited value. Those who claim to have experience in a new discipline often do not; those who have experience often did not have the time to analyze it and, therefore, did not learn. To be valuable, experience must be complemented by rational analyses.

- If a rational analysis based on adequate data shows that a failure can occur, then it is likely to occur. (The only reason that would prevent such a failure from occurring would be the presence of hidden factors of safety.) The engineer who predicts a failure through a rational analysis should believe the results of the analysis, regardless of past experience and common sense. The engineer should, therefore, take the risk of failure seriously and should convince the owner that the failure is likely to occur even if the predicted mode of failure has not been observed before.

- Common sense can be wrong, as well as it can be right. Since the basis for common sense is not known, it is not possible to distinguish between the good and bad aspects of common sense. As a result, common sense is not reliable.

- A rational analysis is the only reliable way to make sound decisions.

4 LEARNING LESSONS AND LESSONS LEARNED

4.1 Overview

The title of this paper includes two essential words: *lessons learned.* Accordingly, many case histories were presented, and lessons were learned from these case histories (see Sections 2 and 3). These lessons were learned from the viewpoint of the design engineer, which is natural because design plays a central role in failures, as indicated in Section 4.2.1. However, mistakes during construction also play an important role in the development of failures, and it is useful to summarize the lessons learned regarding construction, which is done in Section 4.4. In addition to learning lessons from failures, lessons were learned about failures; these lessons are summarized in Section 4.5. However, before these two summary sections, two discussions are presented.

The first discussion (Section 4.2) addresses the impact of design on failures associated with geosynthetics; in this discussion, it is indicated that failures often result from design flaws and reasons for design flaws are reviewed. Recommendations are made to improve designs; one of them is to learn from failures. Accordingly, the second discussion (Section 4.3) addresses the importance of learning lessons from failures and mentions general lessons that can be learned. Then, specific lessons are listed in the two summary sections mentioned above (Sections 4.4 and 4.5).

4.2 Impact of Design on Failures Associated with Geosynthetics

4.2.1 Importance of Design in Failures

Structures incorporating geosynthetics are the results of the direct efforts of manufacturers, design engineers, and contractors (in addition to indirect efforts of others such as owners, distributors of materials, and quality assurance providers). Therefore, failures of structures incorporating geosynthetics result from actions by these three categories of individuals. Accordingly, three categories of failures are often considered: material failures, design failures, and construction failures. However, it should be noted that material failures generally result from a misuse of materials rather than from an inherent flaw of the materials. For example, the development of holes in geotextiles exposed to sunlight for an excessive period of time is indeed a material failure, but it is generally not imputable to the geotextile manufacturer; rather, this problem generally results from a decision by the design engineer, the contractor or the facility operator. Essentially, geosynthetic materials are what they are, and it is up to design engineers (and, to a certain degree, to contractors) to learn about geosynthetics and their limitations. Similar comments can be made about mistakes made during construction. Certainly, many of these mistakes are made by the contractor; however, it should be noted that a number of construction mistakes result from inadequate designs or specifications, unclear construction drawings, failure by the design engineer to communicate with the contractor, and failure by the design engineer to educate the contractor and familiarize him/her with certain features of structures incorporating geosynthetics that are not familiar to many contractors. Clearly, the design engineer plays a central role in the design and construction of structures incorporating geosynthetics, and, consequently, in their failures.

The importance of design in failures is illustrated by the following data from a study of 83 problems (some of them failures) that occurred in 73 modern landfills in the United States. The term "modern landfill" refers to a landfill designed with components substantially meeting current State and Federal regulations, and constructed and operated in accordance with the US state of practice (including construction quality assurance) from the mid-1980s forward. The statistical distribution of the problems is as follows (*Gross et al.*):

- 75% of the problems are related to landfill covers and liners, and 25% to leachate control systems; and
- the problems related to covers and liners break down into 60% structure behavior problems (i.e. 45% stability and 15% deformations) and 40% material problems (i.e. 25% defects and 15% degradation).

From the same database, the following breakdown has been obtained regarding the origin of the problems: design, 50%; construction, 35%; and operation, 15%. It should, however, be noted that the percentage attributed to construction would have been higher if the landfills had not been constructed with construction quality assurance. Nevertheless, these data confirm the comment made above that design is a major cause of failures. Accordingly, the emphasis is on design in the discussion that follows.

4.2.2 Inadequate Design Effort

The author reviewed a number of failures, and found that, in many cases where a failure occurred (including cases where construction and operation mistakes significantly contributed to the failure), the design effort was not sufficient. In particular, some potential failure scenarios were not considered, and materials' properties were not properly evaluated.

The facts that budget and time are limited are often mentioned as reasons why design efforts are insufficient. This is certainly true in a number of cases, and owners who restrict the allocated budget and time to the point that the design engineer cannot work in good conditions should not expect a first class design. It should be noted that engineers should never take assignments if they lack the budget or time to do a proper design, because they are liable even if they agree to work for free. Lack of experience of the design engineer is also mentioned as a reason for inadequate design. It is clear that design teams that do not have experience in geosynthetic engineering should use some help from experienced designers.

However, it is important to note that there are a number of cases where mistakes were made by experienced design engineers with an adequate budget and enough time for design. Based on the author's experience, important mistakes that characterize insufficient design effort, i.e. the fact that some potential failure scenarios were not considered and the fact that materials' properties were not properly evaluated, often result from lack of communication between the various parties involved, lack of discussion within the design team, and lack of focus on important issues. These aspects are discussed below.

4.2.3 Causes of Inadequate Design Effort

Lack of Communication between Parties Involved. As shown by several examples in this paper (*Berg and Meyers*, *Christopher*, *Giroud and Beech*, *Rowe and Seychuk*), a number of failures were caused by lack of communication between owner, manufacturer, material supplier, design engineer, and contractor. The worst case is when there are two teams of design engineers who do not communicate (*Berg and Meyers*). The detrimental effect of such lack of communication is obvious. A more subtle, but equally dangerous, situation is discussed below, the lack of communication between members of the same design team.

Lack of Discussion within the Design Team. Discussion within the design team is essential because various engineers may have different opinions or may prefer different approaches, due to the complexity of the technical issues sometimes associated with the use of geosynthetics. Discussion within the design team is successful only if every technical issue is thoroughly and *openly* discussed by *several* knowledgeable individuals. It is important to emphasize two words: *openly* and *several*. First the word "openly": the author of this paper has seen a design team that was making mistakes because project engineers were afraid of making comments to the production-driven project manager. Then, the word "several": the author of this paper knows of a major failure that happened because the project manager always avoided exposing his design decisions to criticism by the members of the design team and never encouraged round-table discussions. This type of attitude is dangerous because geosynthetic applications are so diverse and many of them so complex that it is not possible for one person to have the right answer to all technical issues.

If the design team does not have all the expertise required to conduct a fruitful discussion, it is far less expensive to hire an outside expert as a peer reviewer than to have a failure. As pointed out by Berg (1993), a well planned peer review "permits to identify potential design problems in the early stage of the project, in time to take corrective action." Even when the design team has all the required expertise, an independent opinion may be useful. Also, when a design team is facing a difficult issue or decision, the presence of an outside expert requires the team to make a formal presentation which can only help clarify the issue. Finally, in cases where an overbearing project manager discourages discussion (usually unconsciously) between team members, the presence of an outside expert at a meeting, if only for one day, will encourage discussions. In the case of the major failure mentioned above, the author of this paper believes that such a meeting would have opened the much needed discussion that would have avoided the major failure that happened. Indeed, the investigation showed that the project manager had not listened to members of the team who had envisioned the failure mechanism; he would have had to listen if a round-table discussion had taken place in the presence of an outside expert.

Lack of Focus on Important Issues. There are too many design reports where the design engineers present in great detail the aspects of the design they like most, such as some calculations, and do not put any special emphasis on the important decisions made during the design process, in particular the decisions related to the key aspects mentioned in Section 4.2.2: the identification and analysis of potential failure scenarios, and the evaluation of materials' properties. This is regrettable because, if important decisions are emphasized and documented in detail in the design report, they are more likely to have been taken more rigorously and they are more likely to be peer-reviewed. For example, if it is understood that the justification for the evaluation of key material properties will be emphasized in the design report and that the process leading to the selection of property values to be used in design calculations will be documented in detail in the design report, it is likely that this selection will be made with great care, possibly with the help of, or the review by, an outside expert. The key point here is that the important design decisions should be formally documented: not only documented in a company peer review log, but documented in the design report.

4.2.4 Improvement of Design Quality

Design Quality Control. The approach described above can be called design quality control, because it is similar to construction quality control (voluntary, provided by installer), but not to construction quality assurance (mandatory, provided by third party). Design quality control is more formal than the peer review system typically practiced by design firms, i.e. design quality control includes: (i) a formal process for important design decisions, including round-table meetings, typically with outside experts; and (ii) a formal and detailed documentation in the design report (and not only in confidential company logs) of the process leading to important design decisions. Clearly, design quality control does not require much additional effort compared to typical peer review. In fact, design quality control is practiced occasionally by certain design teams, and lends credibility to their designs. For example, the selection of an "equivalent" geosynthetic is done more seriously and carries more weight when the steps leading to the selection are documented in the design report. Design quality control has other benefits for design firms, as discussed below.

Benefits of Design Quality Control. Design quality control is obviously beneficial to the owner because it results in safer designs. Design quality control is also beneficial to the designer because: (i) safer designs minimize the risk of litigation, thereby protecting the design firms; and (ii) the participation of the entire design team in important design decisions (including meetings with outside experts) educates the design engineers and is, therefore, beneficial to future designs. Design quality control can be as beneficial to design firms as construction quality assurance to geomembrane installers. It is appropriate, at this point to learn a lesson from the history of the geosynthetic discipline. In 1983, geomembrane installers were reluctant to accept construction quality assurance. However, they quickly understood they were major beneficiaries of construction quality assurance.

By placing emphasis on important decisions such as failure mechanisms and materials' properties, design quality control encourages the designers to consider site conditions. As a result, design quality control should benefit designers by discouraging the market-driven practice of copying past designs. It can be understood that some owners, due to the highly competitive market, would like to decrease design costs by reusing old designs. However, copying a past design is dangerous and has been responsible for failures. The design of structures incorporating geosynthetics, such as landfills or retaining structures, must be project-specific because sites are different and, therefore, it is dangerous to consider that a structure incorporating geosynthetics can be "imported" to a site without considering the site conditions. Also, copying a design detail without understanding its intent can lead to a failure. Furthermore, past designs should not be copied because they may be obsolete, as: (i) new materials become available; (ii) new design methods become available due to analytical developments and lessons learned from failures; and (iii) designers become increasingly aware of some potential failure scenarios which may not have been considered in the past design that is being copied. Past designs can be used as partial models, but should not be copied.

Design Quality Control and Regulations. Design quality control does not have to be mandated by regulations, but it could be encouraged by regulators. For example, regulators could issue a notice of deficiency for designs that do not document important decisions and the peer review process related to these decisions. Increased safety is more likely to result from regulators who challenge design engineers to work at their full capacity than from regulations that limit the engineer's freedom. For example, regulations that prescribe design parameters such as a maximum slope angle for a landfill slope may be counterproductive as they may give a false sense of safety. Clearly, it would be preferable to have regulators who encourage design quality control rather than regulations that prescribe minimum values for parameters. In the first case, engineers are encouraged to do good designs — while, in the second case, overconservatism and overconfidence are encouraged.

Some regulatory agencies have sponsored the development of design methods (e.g. for geotextile filter selection, for water infiltration into landfills, and for geosynthetic-reinforced soil walls). When these methods are widely used, they provide agencies and peer reviewers with a tool they can use to compare designs objectively, which is beneficial to design quality. However, design engineers should be, and feel, free to use other methods in parallel.

4.3 Learning Lessons from Failures Associated with Geosynthetics

4.3.1 The Importance of Learning Lessons

Failures of structures incorporating geosynthetics are essential for calibrating designs, but they are rare. Therefore, no opportunity should be missed to learn from failures. Unfortunately, many of the lessons that could be learned cannot be published because they have to remain confidential due to the litigious atmosphere that often surrounds forensic analyses. When case histories of failures can be published, it is important that they are properly interpreted and that the right lessons are learned. These points are discussed below.

4.3.2 Impact of Litigious Atmosphere on the Possibility of Learning Lessons from Failures

The litigious atmosphere that surrounds forensic analyses can be positive or negative: (i) it is positive because it provides an incentive for opposite viewpoints to be expressed; and (ii) it is negative because it discourages open discussion and fruitful sharing of information. When a failure occurs, opposite teams are formed where lawyers play a major role. Engineers of one team are discouraged from communicating with engineers of the opposite team. As a result of this atmosphere, many interesting case histories are never published, and useful lessons are lost. Certainly, engineers and lawyers should work together, and while lawyers should prevail regarding legal matters, engineers should prevail regarding technical matters. However, it is discomforting to see that the civil engineering profession, which has been a leading profession over the centuries, does not have a leadership role in the case of failures. When a failure occurs, civil engineers should not become soldiers fighting under lawyers' command, but should remain what they ought to be, the leaders of civil engineering. If engineers want to achieve better designs through open discussion of technical issues and peer review, they should adopt a cooperative attitude during forensic analyses. Great lessons would be learned if forensic analyses were not battlefields where nobody listens, but forums where various analyses and opinions are discussed with a view to draw objective conclusions. From this viewpoint, grievances settled through arbitration conducted by a technically competent arbitrator are more satisfactory, and owners should be encouraged to add arbitration clauses to contracts. In fact, in certain countries, an expert is always appointed by the court to conduct a non-binding arbitration. This is a very effective system: it helps settle cases and often avoids lengthy litigation.

4.3.3 Importance of Learning Lessons Established on a Rational Basis

When case histories of failures are available, it is important that they be published. These cases histories often make good reading, as it is entertaining to read about mistakes made by others. However, learning lessons from failures means more than reading anecdotes. The right lessons can be learned only if the failures are rationally analyzed. As indicated below, general lessons can be derived from the experience gained by the author of this paper in analyzing failures of geotechnical structures since 1964.

In all of the forensic analyses in which the author has been involved, it was possible to rationally explain what had happened. From this fact, lessons can be drawn: two lessons for experts performing forensic analyses, and two lessons for design engineers.

Two Lessons for Experts Performing Forensic Analyses. The first lesson for experts performing forensic analyses is that it is easier to explain failures than to do a good design. Consequently, experts performing forensic analyses should not be arrogant and should realize they have a responsibility to share what they have learned.

The second lesson for experts is that they have a duty to rationally explain failures. Experts should refrain from using meaningless concepts and phrases such as "engineering judgment" and "common sense". These phrases essentially serve as camouflage for failure to perform a rational analysis or are used as a screen to hide the laziness inspired by the difficulty inherent to rational analyses. The necessity to rationally explain failures is particularly compelling when a claim is settled by arbitration, as arbitrators must found their decisions on a sound basis. Clearly, rational explanations of failures prevent arbitrators from making arbitrary decisions.

Two Lessons for Design Engineers. The lessons for designers can be introduced as two "principles". For those who have heard these lessons repeated many times, they have become "the two Giroud's principles".

> *The first Giroud's principle is that, if a design engineer predicts a failure using a rational method, the failure is likely to occur and, therefore, the design engineer should believe the prediction.*

This is a lesson that the author of this paper learned the hard way, as described in the case history presented in Section 3.8. In 1980, the author, who was designing a project, predicted a failure using a rational analysis, but other parties involved in the project thought that the prediction by the author was just an academic exercise that had nothing to do with reality. The author insisted his analysis was rational, but he had to acknowledge that there were no facts to prove that the failure prediction derived from the analysis was correct since the predicted mode of failure had never been observed before. As a result, the author could not convince the other members of the team (in part because he respected experience and, therefore, was not fully convinced himself by the prediction he had made). Construction was completed with the measures proposed by the author and the failure occurred as predicted.

The lesson is clear, if a failure is predicted using a rational method, the prediction should be believed; and the failure is likely to occur, even if this mode of failure has never occurred before. This does not necessarily mean that the failure "will" occur (because many factors can increase a factor of safety by a decimal point), but it certainly means that *most experts will be able to explain the failure if it occurs*. This leads to second Giroud's principle.

> *The second Giroud's principle is that a design engineer should never take a risk such that, if a failure occurs, it can be explained by an expert using rational methods.*

Indeed, that would be at least very embarrassing and would probably cause the engineer to incur liability.

The author found that these two principles presented above as Giroud's principles are very useful in helping design engineers resist excessive pressures from overdemanding clients or from overzealous project managers, two situations that may lead some engineers to make mistakes that could lead to failures for which they would be held liable.

4.3.4 Importance of Learning Lessons from the Field

As pointed out in Section 4.2.1, a number of the mistakes made during construction are due to inadequate design. Therefore, it is important that design engineers visit construction sites to understand working conditions in the field. Also, design engineers should not miss any opportunity to see failures in the field firsthand. While learning from failures by reading published case histories is irreplaceable because of the extent of knowledge that can thus be acquired in a relatively short period of time, it is important for design engineers to go to the field, especially to see failures — just as reading stories about exotic countries is unsatisfactory and does not provide a full experience if the reader of these stories does not visit at least some of the countries. A design engineer who often goes to the field is better prepared for design than a design engineer otherwise equally qualified who never goes to the field. A design engineer who has seen a failure is better prepared for design than a design engineer otherwise equally qualified who has never seen a failure. A design engineer who has seen a failure and has written a report about the failure is better prepared for design than a design engineer otherwise equally qualified who has seen a failure, but has not written a report about it.

The author of this paper had the opportunity, early in his career (in the mid 1960s), to learn how important it is to go to the field thanks to the sense of humor of a seasoned consulting engineer for whom he was doing some design work. The seasoned consulting engineer told him one day: "You seem to like what you do and you probably want to make a career in geotechnical engineering. If this is the case, believe me, never, never go to the field!" As the author, then a young engineer, was surprised, to say the least, having learned that geotechnical engineering is an "outdoor sport" and having already been in the field a number of times, the seasoned consulting engineer added "Yes, never go to the field if you want to stay in this profession, because you will be disgusted when you see what 'they' do with your neat designs!"

Since that time, the author of this paper has been countless times to the field. Every time the author has to go to the field to see a failure, he reads what is available on the project and, before going to the site, develops a scenario, often wrong, always useful. Design engineers going to the field to see a failure should have in mind a number of preconceived ideas (possibly conflicting) about what happened. It may be a waste of time to go to the field with an empty mind. But, of course, engineers must be prepared to change their minds based on the observations made in the field, and, when coming back to the office, must also be prepared to change their minds based on the analyses made in the office and/or in the laboratory. Finally, it is important to stress that engineers should not go the field only to see failures. As pointed out by Thiel (1999) "Engineers need to learn how things are built in the field. They need to learn how the equipment moves dirt, compacts, the *art* of moisture conditioning soils, the *art* of coordinating mass excavations and fills, the limitations of equipment on slopes, the limitations of welding pipes and geomembranes, how roll goods are deployed, etc. Only by knowing how things are built will they be able to create really effective designs." In fact, design engineers visiting construction sites generally enjoy the visit and are amazed by the magnitude of the efforts and the variety of materials.

4.3.5 Conclusion on Learning Lessons

An essential lesson is that many failures are due to improper design and, therefore, design engineers must make efforts to improve design quality. This includes learning from failures. The lessons learned from failures should not only be the technical lessons summarized in Sections 4.4 and 4.5, but also the more general lessons presented in Sections 4.3.3 and 4.3.4, i.e. not only technical lessons to increase knowledge but also professional lessons to better use knowledge.

Examples presented in this paper show that many failures of structures incorporating geosynthetics are due to mistakes made during construction. Many of these examples show that many of the mistakes made during construction are due to failure at the design stage to foresee problems that could occur at the site. Therefore, the comments presented in Section 4.4 should benefit design engineers as well as contractors.

4.4 Summary of Lessons Learned Regarding Construction

The "lessons learned" presented in Section 4.4 are derived from the examples and case histories presented in Sections 2 and 3, and occasionally a few additional comments.

4.4.1 Geosynthetic Selection

The following comments can be made regarding geosynthetic selection at the construction stage:

- If the contractor is allowed to substitute an "equivalent" geosynthetic for the specified geosynthetic, the contractor should do so only with great precautions, because many geosynthetics that appear to be similar are not. The design engineer or a qualified consulting engineer should be involved in evaluating the equivalency. Contractors should learn that simply comparing tables of geosynthetic properties is not sufficient to establish equivalency between two geosynthetics. Furthermore, contractors should not believe that two geosynthetics are equivalent only on the basis of claims by geosynthetic suppliers.

- If a contractor wants to add a geosynthetic that is not in the construction drawings, the contractor should do so only if the proposed geosynthetic addition is reviewed by the design engineer or a qualified consulting engineer.

- If possible, the contractor should prefer geosynthetics available in wide rolls to minimize the number of seams or overlaps.

4.4.2 Geosynthetic Storage and Placement

The following comments can be made regarding geosynthetic storage and placement:

- Geosynthetics should be stored in a clean place. For example, clogging of geonets due to dust or mud has occurred during storage.

- Geosynthetics should be placed on an adequately prepared soil surface. In particular, the soil surface should be free of elements likely to damage the geosynthetic. Also, the soil surface should not have abrupt changes in grade likely to cause stress concentration in the geosynthetic.

- If a geomembrane liner is placed on a horizontal pond floor and if the geomembrane is not covered with a layer of soil or other material, there is a high risk that the geomembrane will be uplifted by air entrapped on high spots of the pond floor during the first filling of the pond.

- To minimize the risk of stress concentration, the number of seams should be minimized. This can be achieved by using geosynthetics available in wide rolls (as mentioned in Section 4.4.1) and by minimizing the number of samples taken for destructive testing. Also, samples for destructive testing should not be taken at locations where high tensile stresses are likely to develop, such as the top of slopes.

- Geomembrane liners, and sometimes geotextiles, should possess the tensile strength required to resist wind uplift and should be secured using sand bags, anchor trenches, or other appropriate means.

- Contractors should be aware that there are some applications where it is essential that the geosynthetic be in intimate contact with the adjacent material (see Section 4.4.3). In these applications, the surface of the material on, or against, which the geosynthetic is to be placed should not have irregularities that may prevent intimate contact between the geosynthetic and the material.

- Contractors should be aware of the influence of weather during construction on the quality of the installed geosynthetic (see Section 4.4.5).

- Workers and visitors should refrain from any activities that could damage installed geosynthetics.

4.4.3 Intimate Contact Between Geosynthetic and Adjacent Materials

Intimate contact between the geosynthetic and the adjacent material is essential to the performance of the geosynthetic in some applications. Some examples follow:

- A geotextile filter must be in intimate contact with the adjacent soil. In the case of drainage trenches, this can be achieved by using a flexible geotextile and filling the trench with relatively small stones. In the case of a relatively rigid edge drain, sand should be poured in the space between the edge drain filter and the walls of the trench. In the case of bank protection, intimate contact can be achieved by using a flexible geotextile with a layer of relatively small stones between the geotextile and the rocks.

- A geomembrane placed on top of a layer of compacted clay to form a composite liner must be in intimate contact with the clay. It is, therefore, important to place the geomembrane with wrinkles as small as possible (because small wrinkles tend to flatten under the weight of overlying layers).

- A paving geotextile, used on an existing pavement prior to placing an asphalt overlay, must be in intimate contact with the existing pavement. Furthermore, there should be a sufficient amount of asphalt tack coat to impregnate the geotextile and bind it to the existing pavement and the asphalt overlay. The intimate contact between the geotextile and the existing pavement can be achieved by rolling the geotextile (after the tack coat application) with a pneumatic roller. A steel roller is too rigid to ensure intimate contact.

- A geomembrane liner used in a concrete tank should be placed in intimate contact with the corners of the tank to ensure that it will not burst during filling.

4.4.4 Geosynthetic Connections

The following comments can be made regarding geosynthetic connections:

- Geosynthetic overlaps should be sufficiently wide that they will not separate even in case of settlement of the underlying soil. If overlap separation cannot be avoided, the overlaps should be replaced by seams.

- Overheating of a geomembrane during seaming should be avoided.

- Connections of geosynthetics with structures or elements of structures should be such that they will not be damaged by differential settlements. This applies, in particular, to connections of geomembrane liners to appurtenances and connections of reinforcing geosynthetics to facing elements of reinforced soil structures. Geotechnical engineers must review connections very carefully because they are sometimes designed by individuals who have an insufficient knowledge of geotechnical engineering and who, therefore, do not realize that even a small amount of differential settlement may cause the connection to fail.

- In case of high hydraulic gradient, geotextile filters and geomembrane liners should be seamed, not overlapped. Also, in case of high hydraulic gradient, the connection between a geomembrane liner and a clay liner or a clay-filled anchor trench should be done with great care. It is preferable, if possible, to avoid this dangerous situation and to move the connection to a location where the hydraulic gradient is small, such as at the top of a slope.

- The extremity of a geotextile likely to convey undesirable water should be sealed. This is the case, for example, at the toe of a dam or the anchor trench of a landfill. Also, needle-punched nonwoven geotextiles and woven geogrids used in pavements should be impregnated with asphalt to prevent the geotextiles or the geogrids from conveying water into the pavement.

4.4.5 Influence of Weather and Other External Conditions on Geosynthetics

The following comments can be made regarding the influence of weather on geosynthetics:

- Geotextiles should not be exposed to sunlight for more than a few weeks, unless planned otherwise at the design stage. Exposure to sunlight may result in severe degradation of many geotextiles.

- A composite liner that consists of a geomembrane on compacted clay should not remain exposed on a slope, because desiccation and cracking of the clay may occur.

- Geomembrane seaming should not take place under adverse weather conditions because it has been shown that the number of geomembrane seam defects increases under those conditions.

- A geomembrane liner should not be placed on a low-permeability soil having a water content higher than specified (in particular after a rainfall) because this may result in a significant decrease in the interface shear strength.

- Thermal expansion of geomembranes causes wrinkles, which are undesirable for several reasons: (i) they can be damaged by placement of soils on top of the geomembrane; (ii) they prevent intimate contact between a geomembrane and the underlying clay in a composite liner; and (iii) in landfills, they impede the flow of leachate in leachate collection layers. When it is important to minimize wrinkles, it may be necessary to install geomembranes at night and to cover them with soil before the temperature increases. Alternatively, geomembranes that are less likely than others to exhibit wrinkles can be selected, i.e.: (i) geomembranes with a white upper surface and/or a textured lower surface; (ii) geomembranes with a low bending modulus (i.e. flexible geomembranes); (iii) geomembranes with a small coefficient of thermal expansion, such as reinforced geomembranes; and/or (iv) heavy geomembranes.

- Thermal contraction causes tensions in geomembranes exposed to low temperatures. When it is important to minimize tensions, it is desirable to place geomembrane liners with some slack, which is in contradiction with the no-wrinkle requirement mentioned above. Alternatively, the geomembrane may be installed flat at a relatively low temperature (provided that the low temperature is compatible with seaming quality).

- As indicated in Section 4.4.2, geotextiles and geomembranes can be uplifted by wind and they should be secured.

- Contractors should be made aware that geofoam blocks may catch fire if the geofoam is not of the flame-retardant type. Therefore, precautions should be taken when performing activities such as welding. Also, in case of thunderstorm, lightning may set fire to geofoam and other geosynthetics.

4.4.6 Placement of Materials in Contact with Geosynthetics

The following comments can be made regarding the placement of materials in contact with geosynthetics:

- Geomembranes are more damaged by placement of the overlying materials than by any other construction activity. Therefore, it is important that this construction activity be conducted with maximum care and construction quality assurance. Furthermore, the integrity of the geomembrane could be checked after placement of the soil layer by using the electric leak location method or conducting a ponding test.

- To minimize the risk of geomembrane damage by overlying materials, it is advisable to select overlying materials that are least likely to damage the geomembrane: (i) when granular soil is used, the particles should be as small as possible; (ii) if necessary, a thick needle-punched geotextile acting as a cushion should be used between the geomembrane and the granular soil; and (iii) when reinforced concrete is used to protect a geomembrane (as on the upstream face of dams) fiber-reinforced concrete should be preferred to the traditional concrete reinforced with steel bars.

- Clear specifications and instructions should be given to the contractor to minimize damage to geosynthetics used for soil reinforcement.

- Fresh concrete should not be placed in contact with a polyester geotextile, because of potential degradation of the polyester by hydrolysis.

- Contractors should be made aware that placing a geotextile in contact with a geonet can significantly reduce geonet transmissivity and should be done in strict accordance with specifications.

- Placement of soil layers overlying geosynthetics may cause soil settlement or bearing capacity failure if the soil supporting the geosynthetics has high compressibility and/or low strength. The resulting deformation of the soil surface may cause geosynthetics or seams to rupture, or may cause geosynthetic overlaps to separate. Lightweight equipment should be used to place the soil overlying the geosynthetics.

- A bulldozer that pushes a soil layer against a geotextile overlap may cause the overlap to separate. Therefore, the direction of soil placement with respect to geotextile overlaps should be specified.

- A bulldozer that pushes a soil layer downward on a liner system on a slope may cause veneer instability. Therefore, the direction of soil placement on a slope should be specified.

- Construction equipment that maneuvers and brakes on a soil layer overlying a liner system on a slope may cause veneer instability.

- Vibratory rollers should be used with caution on a soil layer overlying a liner system on a slope because the vibration may cause veneer instability.

- When a high strength geotextile is used to reinforce an embankment constructed on soft soil, care should be taken during placement of the soil on the geotextile to not create wrinkles in the geotextile. These wrinkles would prevent the geotextile from mobilizing its strength, which could cause an embankment failure.

- When ore is placed on a leach pad with a slope (e.g. 5%), the ore should be placed in the up-slope direction, because if there is any departure from the designed slope (e.g. 6% slope) the factor of safety increases in case of up-slope placement and decreases in case of down-slope placement.

4.4.7 Required Soil Deformation

The following comments can be made regarding the required deformation of a soil associated with a reinforcing geosynthetic:

- Contractors should be made aware of the need for reinforcing geosynthetics to deform to mobilize their strength when they perform the reinforcement function. For example, contractors should learn that the facing of geosynthetic-reinforced soil walls moves laterally during and after construction.

- Contractors should be made aware that a geosynthetic-reinforced unpaved road must exhibit ruts for the geosynthetic tension to be mobilized. This requirement may cause grading problems if a temporary unpaved road is to be eventually incorporated in a permanent road.

- Contractors should be made aware that it is important to place reinforcing geosynthetics under tension, if possible, or, at least, without slack to minimize the required geosynthetic deformation to mobilize its tension.

- Contractors should be made aware that the facings (particularly the "wrap-around" facings) of vertical, or quasi vertical, geosynthetic-reinforced soil walls deform and, as a result, may come in contact with adjacent structures if not enough space was left between the facings and the adjacent structures.

4.4.8 Impact of Geotechnical Problems on Structures Incorporating Geosynthetics

The following comments can be made regarding the impact of geotechnical problems on structures incorporating geosynthetics:

- The soil of reinforced soil structures must be properly compacted. In particular, if cohesive soil is used, this soil should be compacted at the specified water content and, in particular, should not be compacted too dry.

- Temporary excavations done to construct structures incorporating geosynthetics should be stable.

- Surface drainage must always be effective because excess water is a common cause of failure of structures incorporating geosynthetics as well as geotechnical structures.

- When geosynthetics are supported by a material that is highly compressible or has a low bearing capacity, only lightweight construction equipment should be used.

4.5 Summary of Lessons Learned Regarding Failures

The "lessons learned" presented in Section 4.4 are derived from the examples and case histories presented in Section 2 and 3, and occasionally a few additional comments.

4.5.1 Prediction of Failures

The main lessons regarding the prediction of failures can be summarized as follows:

- Most failures can be predicted using rational analyses based on principles of physics and mechanics, fundamental knowledge of geotechnical engineering, understanding of functions of geosynthetics, and a good knowledge of geosynthetic properties.

- The design engineer who predicts a failure using rational analyses should believe the results of the analyses and convince other parties (i.e. the owner, contractor, etc.) that the failure is likely to happen.

- The design engineer who makes a prediction based on a rational analysis must not be impressed by experience from others, because there are only a few people who really have experience in the field of geosynthetics. The most dangerous of those who have experience are those who rely only on common sense instead of using rational analyses to draw lessons from their experience.

- Most failures are easy to explain, but some are difficult to predict. Therefore, engineers must learn from case histories describing failures.

4.5.2 Prevention of Failures

The main lessons regarding the prevention of failures can be summarized as follows:

- Geosynthetics should not be expected to make miracles in spite of claims by overzealous salespersons. Geosynthetics must be treated like other construction materials, and it must be recognized that, in come cases, geosynthetics may have a detrimental effect.

- Properties of geosynthetics must be measured and not estimated, because geosynthetics that seem identical often have different properties. Tests used to measure geosynthetic properties must be carefully selected because tests that are not representative of the field situation give results that are incorrect.

- At the design stage, design engineers must consider all potential mechanisms of geosynthetic failure. To that end, they must keep themselves abreast of observations made by others on the performance of structures incorporating geosynthetics, and they must learn about new design methods. It should be noted that the presence of a geosynthetic makes possible some new mechanisms of failure, without eliminating most of the traditional mechanisms of failure.

- To keep themselves abreast of modes of failures observed by others, design engineers must, of course, listen to presentations and read papers on this subject (Giroud, 1977a, 1977b, 1983a, 1983b, 1984a, 1984e, 1984f, 1993). They must also learn about the design and construction of structures incorporating geosynthetics that perform satisfactorily (Raymond and Giroud, 1993).

- Design engineers must write specifications that are complete and precise and that address problems that might occur during construction.

- The design engineer should be present or represented at the site during important phases of construction.

- It is necessary to educate designers and contractors on the potentials and limitations of geosynthetics. Regarding designers, this means that, among other things, they should learn from polymer specialists, learn from failures, learn regarding installation constraints for geosynthetics, and educate contractors.

4.5.3 Action in Case of Failure

The main lessons regarding action to be taken in case of failure of structures incorporating geosynthetics can be summarized as follows:

- A complete investigation must be undertaken. This investigation should address all aspects of design and construction and should not be limited to aspects related to geosynthetics. The investigation should be conducted in a way that makes it possible to adequately review all possible failure mechanisms.

- Parties that conduct independent investigations of a failure should be required to communicate at some stages of their investigations. In particular, observations should be compared and an agreement should be reached on tests to be conducted.

- If the investigation includes tests, the tests must be representative of the field conditions. Some tests may give the illusion of being representative. The fact that a test is performed in the field does not guarantee that it will be representative. Some laboratory tests are more representative of field conditions than some field tests. Clearly, on this point, common sense may be wrong.

- One should not automatically take the position that geosynthetics are responsible for failures and can be used as scapegoats. Taking such a position may lead to selecting inappropriate remedial measures.

- If it appears that the design of a structure is flawed, one may take advantage of the repair work to improve the design, instead of systematically reconstructing the structure as it was before.

- On the pretext that the failure was associated with a geosynthetic, one should not systematically eliminate geosynthetics when designing remedial measures. In reality, failures of projects built with geosynthetics are generally repaired successfully using geosynthetics.

4.5.4 Responsibility in Case of Failure

The main lessons regarding responsibility in case of failure can be summarized as follows:

- In case of a failure of a structure incorporating geosynthetics, one should not automatically assume that the supplier and/or installer of the geosynthetics are responsible. Experience shows that observed failures are often due to a number of other causes.

- The designer of a project must take all possible precautions at the design stage to avoid the situation where, if a failure were to occur, any expert would be able to explain it using methods that were available at the time when the project was designed.

- Those who dare writing papers on lessons from observation of failures must avoid, when they design projects, to be in a situation where they could eventually be accused of ignoring what they teach. Alternatively, they should abstain from designing projects.

5 CLOSURE

Geosynthetic engineering is a relatively new discipline; however, it has all the attributes of a full-fledged discipline because it has failures. A discipline needs failures to progress, because failures are an excellent way to calibrate designs, to challenge engineers to learn lessons, and to motivate them for design and construction quality. Fear of failures encourages engineers to work better.

Some people do not want to discuss failures associated with geosynthetics because failures have a negative impact on the credibility of the geosynthetic discipline. Indeed it is true that discussing failures may have a short-term negative impact, but openly discussing failures is a long-term investment because it is an excellent way to increase designers' knowledge and minimize the risk of failures in the future. The author of this paper used the opening speech of the International Conference on Geomembranes in 1984 (Giroud, 1984b) to denounce the poor quality of installed geomembrane liners with maximum visibility. He was criticized for that and his slogan "all liners leak" was repeated thousands of times, not always in a constructive manner. However, this denouncement contributed to the development of liner construct quality assurance, which is now widely recognized as having been the main factor in establishing the respectability enjoyed today by the liner industry and the geomembrane branch of the geosynthetic discipline.

To those who believe that failures should not be openly discussed, it should be pointed out that geosynthetics are not always — in fact not often — the cause of the observed failures of structures incorporating geosynthetics, and they should not be used as scapegoats. Most of the failures discussed in the examples and case histories presented in this paper were not caused by a geosynthetic; they are only failures of structures incorporating geosynthetics. Failures of structures incorporating geosynthetics do not occur because geosynthetics are not good, but because geosynthetics are used extensively. In four decades, of the order of 10 billion m^2 of geosynthetics have been installed in a number of structures of the order of 1 million; since the author of this paper has collected approximately 100 case histories of significant failures, it may be inferred that the number of significant failures is of the order of 1000, hence a percentage of significant failures of the order of 0.1%, which is small. It may also be noted that the review of failures associated with geosynthetics presented in this paper shows the success of geosynthetics in the following ways: first, geosynthetics are used in virtually all branches of geotechnical engineering; and, second, the failures of structures incorporating geosynthetics are usually repaired successfully using geosynthetics. It is possible that structures incorporating geosynthetics are safer than traditional geotechnical structures, i.e. structures that do not incorporate geosynthetics, in part because of the reliability of geosynthetics (which are manufactured with quality control) compared to soils.

A discipline is mature when it can openly discuss its failures. The fact that failures of structures incorporating geosynthetics are so openly discussed today is a sign that the geosynthetic discipline is mature. The geosynthetic discipline is mature today, not only because it has developed an impressive and consistent body of rational knowledge in a rather short period of time, but also because it has early in its history started discussing its failures. At this point, the beneficial example set by the geotechnical engineering discipline must be acknowledged. Lessons learned from failures of foundations, slopes, and embankment dams, are famous, and the *lessons learned from failures associated with geosynthetics* belong to the same engineering tradition.

Geosynthetics have been successful in pervading geotechnical engineering. As a result, geosynthetics have been involved in a wide range of geotechnical structures, which has created a wide variety of opportunities for failures. The wide variety of failures associated with a wide range of geotechnical structures provides many opportunities to learn not only about geosynthetics, but also about geotechnical engineering. Therefore, it is hoped that the *lessons learned from failures associated with geosynthetics* will be considered to be an important addition to the thesaurus of geotechnical failures, and will contribute to a deeper understanding of geotechnical engineering.

It is always entertaining to learn about, and from, failures, especially if the failures result from mistakes made by others. However entertaining it may be, learning from failures is not easy. First, it requires the availability of a number of well documented case histories, which is not easy considering the confidentiality of so many interesting cases, and knowing that those who are busy analyzing important cases do not have much time to write. Furthermore, learning from failures requires a strict intellectual discipline. Forensic analyses should be based on rational deductions conducted with Cartesian rigor. It is clear from many examples presented in this paper that common sense should not be used in forensic analyses. It has been shown in this paper that common sense is a random process that can have credibility only with those who prefer a veneer of satisfaction to the depth of understanding, and who prefer the comfort of illusion to the rigor of logic. To learn technical lessons, common sense is a common temptation that does not make sense.

Common sense is not only dangerous in forensic analyses, it is also dangerous in design and construction. Indeed, this paper shows, on the basis of numerous examples and case histories, that decisions based on common sense were involved in the process that led to many of the observed failures. It appears clearly that common sense cannot serve as a basis for rational decisions in a scientific discipline such as geotechnical engineering. This paper also shows that common sense, because of its preference for traditional solutions, is particularly detrimental in the case of novel technology such as geosynthetics. It is clear that the use of common sense must be banished from all scientific disciplines, in particular those which are under development. But, understanding this recommendation may require more than common sense.

As mentioned many times in this paper, it is important to educate designers and contractors on the possibilities and limitations of geosynthetics. Learning from failures is the best way to decrease the number of failures in the future. Therefore, this paper should hopefully help reduce the number of failures. Society at large will benefit because of a decreased number of claims relative to the total number of structures constructed with geosynthetics. The geosynthetic discipline will benefit because its credibility will increase. The author of this paper will benefit because he will not have to write an update. Finally, this paper should benefit those who have read this paper beyond its first sentence, which is repeated below:

> *Geotechnical engineers who do not learn from mistakes made by others will learn from their own mistakes.*

ACKNOWLEDGMENTS

The author is very grateful to R.R. Berg, B.A. Gross, G.N. Richardson, K.L. Soderman, R.S. Thiel, R.W. Thomas, and J.G. Zornberg for their detailed review of this paper The author is also grateful to R.C. Bachus, R. Bonaparte, and P.C. Lucia for their review of parts of this paper, and to K.E. Holcomb for assistance in the preparation of the paper. Finally, the author acknowledges the contribution of the 135 authors who prepared documented case histories for the book on failures he is finalizing, which provided him with a unique wealth of information on the subject of this paper.

REFERENCES

Note: References which appear in the text in italics are those of papers included in the book on lessons learned from failures in preparation by J.P. Giroud. Other references are as follows.

Badu-Tweneboah, K., Tisinger, L.G., Giroud, J.P. and Smith, B.S. (1999), "Assessment of the Long-Term Performance of Polyethylene Geomembrane and Containers in a Low-Level Radioactive Waste Disposal Landfill", *Proceedings of Geosynthetics '99*, Boston, Massachusetts, USA, April 1999.

Berg, R.R, (1993), "Project Peer Review in Geosynthetic Engineering", *Geotechnical Fabrics Report*, Vol. 11, No. 3, April 1993. pp. 34-35.

Delmas, P., Nancey, A., Faure, Y., and Farkouh, B. (1992), "Long term behaviour of a non-woven geotextile in a 21 year-old dam", *Geo-Filters '92, Proceedings of the International Conference on Filters and Filtration Phenomena in Geotechnical Engineering*, Karlsruhe, Germany, October 1992, pp. 331-338.

Delmas, P., Faure, Y., Farkouh, B. and Nancey, A. (1993), "Comportement à long terme d'un géotextile non-tissé dans un barrage de plus de 20 ans — Long term behaviour of a non-woven geotextile in a dam older than 20 years", *Proceedings of Rencontres 93*, Joué les Tours, France, pp. 305-314. (in French and in English)

Fluet, J.E. and Luettich, S.M. (1993), "Geotextile Filter Criteria for Gap-Graded Silty Sands", *Proceedings of Geosynthetics '93*, Vol. 1, Vancouver, Canada, March-April 1993, pp. 469-481.

Girard, H., Fischer, S. and Alonso, E. (1990), "Problems of Friction Posed by the Use of Geomembranes on Dam Slopes — Examples and Measurements", *Geotextiles and Geomembranes*, Vol. 9, No. 2. pp.129-143.

Giroud, J.P. (1977a), "Conception de l'étanchéité des ouvrages hydrauliques par géomembranes" (Conceptual Design of Geomembrane Waterproofing for Hydraulic Works), *Proceedings of the First International Symposium on Plastic and Rubber Waterproofing in Civil Engineering*, Vol. 1, Session 3, Liège, Belgium, June 1977, pp. III.13.1 - III.13.17. (in French)

Giroud, J.P. (1977b), "Discussion sur la conception de l'étanchéité des ouvrages hydrauliques par géomembranes" (Discussion of Conceptual Design of Geomembrane Waterproofing for Hydraulic Works), *Proceedings of the First International Symposium on Plastic and Rubber Waterproofing in Civil Engineering*, Vol. 2, Discussions de la Session 3, Liège, Belgium, June 1977, pp. III.13.19-III.13.21. (in French)

Giroud, J.P., Gourc, J. P., Bally, P., and Delmas, P. (1977a), "Comportement d'un textile non tissé dans un barrage en terre" (Behaviour of a Non-Woven Fabric in an Earth Dam), *Proceedings of the International Conference on the Use of Fabrics in Geotechnics*, Vol. 2, Paris, France, April 1977, pp. 213-218. (in French)

Giroud, J.P., Gourc, J.P., Bally, P., and Delmas, P. (1977b), "Commentaires sur le comportement d'un textile non-tissé dans un barrage en terre" (Discussion of Behavior of a Non-Woven Fabric in an Earth Dam), *Proceedings of the International Conference on the Use of Fabrics in Geotechnics*, Vol. 3, Paris, France, April 1977, pp. 68-71. (in French)

Giroud, J.P. (1980), "Introduction to Geotextiles and their Applications", *Proceedings of the First Canadian Symposium on Geotextiles*, Calgary, Canada, September 1980 , pp. 3-31.

Giroud, J.P., (1981), "Designing with Geotextiles", *Matériaux et Constructions*, Vol. 14, No. 82, July-August 1981, pp. 257-272.

Giroud, J.P., (1982a), "Design of Geotextiles Associated with Geomembranes", *Proceedings of the Second International Conference on Geotextiles*, Vol. 1, Las Vegas, Nevada, USA, August 1982, pp. 37-42.

Giroud, J.P. (1982b), "Filter Criteria for Geotextiles", *Proceedings of the Second International Conference on Geotextiles*, Vol. 1, Las Vegas, Nevada, USA, August 1982, pp. 103-108.

Giroud, J.P. and Goldstein, J.S. (1982), "Geomembrane Liner Design", *Waste Age*, September 1982, pp. 27-30.

Giroud, J.P. (1983a), "Rapport général sur les bassins revêtus de géomembranes" (General Report on Geomembrane-Lined Reservoirs), *Comptes-rendus du colloque sur l'étanchéité superficielle des bassins, barrages et canaux*, Vol. 2, Paris, France, February 1983, pp. 75-90d. (in French)

Giroud, J.P. (1983b), "Discussion du rapport général sur les bassins revêtus de géomembranes" (Discussion of General Report on Geomembrane-Lined Reservoirs), *Comptes-rendus du colloque sur l'étanchéité superficielle des bassins, barrages et canaux*, Vol. 3, Paris, France, February 1983, pp. 16-18. (in French)

Giroud, J.P. (1984a), "Geomembrane Liners: Accidents and Preventive Measures", *Proceedings of the Second International Symposium on "Plastic and Rubber Waterproofing in Civil Engineering*, Session 4, Liège, Belgium, June 1984, pp. 4.2.1-4.2.6.

Giroud, J.P. (1984b), "Opening Address", *Proceedings of the International Conference on Geomembranes*, Vol. 1, Denver, Colorado, USA, June 1984, pp. 1-3.

Giroud, J.P. (1984c), "Aging of PVC Geomembranes in Uranium Mine Tailing Ponds", *Proceedings of the International Conference of Geomembranes*, Vol. 2, Denver, Colorado, USA, June 1984, pp. 311-316.

Giroud, J.P. (1984d), "Analysis of Stresses and Elongations in Geomembranes", *Proceedings of the International Conference on Geomembranes*, Vol. 2, Denver, Colorado, USA, June 1984, pp. 481-486.

Giroud, J.P. (1984e), "*Geotextiles and Geomembranes. Definitions, Properties and Design*", IFAI, St. Paul, Minnesota, USA, 1984, 325 p.

Giroud, J.P. (1984f), "*Assessment of Synthetic Membrane Performance at Waste Disposal Facilities*", USEPA Contract 68-03-1772, United States Environmental Protection Agency, Cincinnati, Ohio, USA, November 1984, 59 p.

Giroud, J.P., Gourc, J.P., Bally, P., and Delmas, P. (1984), "Behavior of a Nonwoven Geotextile in an Earth Dam", Section 1.4 in *Geotextiles and Geomembranes, Definitions, Properties and Design*, by J.P. Giroud, IFAI, St. Paul, Minnesota, USA, 1984, 10 p.

Giroud, J.P. (1985), "Tomorrow's Designs for Geotextile Applications", *"Geotextile Testing and the Design Engineer"*, Fluet, J.E., Editor, ASTM STP 952, 1987, Proceedings of a symposium held in Los Angeles, California, USA, June 1985, pp. 145-158.

Giroud, J.P., Arman, A., and Bell, J.R. (1985), "Geotextiles in Geotechnical Engineering, Practice and Research", Report of the ISSMFE Technical Committee on Geotextiles, *Geotextiles and Geomembranes*, Vol. 2, No. 3, Elsevier, London, England, July 1985, pp. 179-242.

Giroud, J.P. and Fluet, J.E., Jr. (1986), "Quality Assurance of Geosynthetic Lining Systems", *Geotextiles and Geomembranes*, Vol. 3, No. 4, Elsevier, London, UK, 1986, pp. 249-287.

Giroud, J.P. (1989), "Panelist Contribution on Geotextile Filters", *Proceedings of the Twelfth International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, Brazil, August 1989, Vol. 5, pp. 3105-3106.

Giroud, J.P. (1990), "The IGS and the Geopolymer Discipline", Opening Address by the President of the IGS, *Proceedings of the Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Vol. 3, The Hague, The Netherlands, May 1990, pp. 893-896.

Giroud, J.P., Bonaparte, R., Beech, J.F., and Gross, B.A. (1990a), "Design of Soil Layer-Geosynthetic Systems Overlying Voids", *Geotextiles and Geomembranes*, Vol. 9, No. 1, Elsevier, London, UK, 1990, pp. 11-50.

Giroud, J.P., Swan, Jr., R.H., P.J. Richer, and P.R. Spooner (1990b), "Geosynthetic Landfill Cap: Laboratory and Field Tests, Design and Construction", *Proceedings of the Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Vol. 2, The Hague, The Netherlands, May 1990, pp. 493-498.

Giroud, J.P., Swan, R.H., Jr., Richer, P.J., and Spooner, P.R. (1990c), "Discussion of Geosynthetic Landfill Cap: Laboratory and Field Tests, Design and Construction", *Proceedings of the Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Vol. 3, The Hague, The Netherlands, May 1990, pp. 1097-1098.

Giroud, J.P. and Morel, N. (1992), "Analysis of Geomembrane Wrinkles", *Geotextiles and Geomembranes*, Vol. 11, No. 3, pp. 255-276. (Erratum: 1993, Vol. 12, No. 4, p. 378.)

Giroud, J.P. and Gross, B.A. (1993), "Geotextile Filters for Downstream Drain and Upstream Slope, Valcros Dam, France", *Geosynthetics Case Histories*, Raymond, G.P. and Giroud, J.P., Editors, ISSMFE, March 1993, pp. 2-3.

Giroud, J.P. (1993), "Leçons tirées de l'étude de la performance des géosynthétiques — Lessons Learned from Studying the Performance of Geosynthetics", *Proceedings of Rencontres 93*, Comité Français des Géosynthétiques, Joué les Tours, pp. 15-46. (in French and English).

Giroud, J.P., Luettich, S.M., and Charron, R.M. (1993), "Specify, Don't Overspecify", *Geotechnical Fabrics Report*, Vol. 11, No. 9, December 1993, pp. 4, 6, 8, 9-10.

Giroud, J.P. (1994a), "Determination of Geomembrane Shattering Cracks", *Geosynthetics International*, Vol. 1, No. 2, pp. 149-180.

Giroud, J.P. (1994b), "Quantification of Geosynthetics Behavior", *Proceedings of the 5th International Conference on Geotextiles, Geomembranes and Related Products*, Singapore, September 1994, Vol. 4, pp. 1249-1273.

Giroud, J.P.(1995a), "Wrinkle Management for Polyethylene Geomembranes Requires Active Approach", *Geotechnical Fabrics Report*, Vol. 13, No. 3, April 1995, pp. 14-17.

Giroud, J.P. (1995b), "Determination of Geosynthetic Strain due to Deflection", *Geosynthetics International*, Vol. 2, No. 3, pp. 635-641.

Giroud, J.P., and Soderman, K.L. (1995), "Design of Structures Connected to Geomembranes", *Geosynthetics International*, Vol. 2, No. 2., pp. 379-428.

Giroud, J.P., Pelte, T., and Bathurst, R.J. (1995a), "Uplift of Geomembranes by Wind", *Geosynthetics International*, Vol. 2, No. 6, pp. 897-952. (Erratum, Vol. 4, No. 2, pp. 187-207.)

Giroud, J.P., Tisseau, B., Soderman, K.L., and Beech, J.F. (1995b), "Analysis of Strain Concentrations Next to Geomembrane Seams", *Geosynthetics International*, Vol. 2, No. 6, pp. 1049-1097.

Giroud, J.P. (1996), "Granular Filters and Geotextile Filters", *Proceedings of GeoFilters '96*, Lafleur, J. and Rollin, A.L., Editors, Montréal, Canada, May 1996, pp. 565-680.

Giroud, J.P., Houlihan, M.F. Bachus, R.C. and Qureshi, S. (1998), "Clogging Potential of Geosynthetic Leachate Collection Layers by Fine Particles from Soil Protective Layers", *Proceedings of the Sixth International Conference on Geosynthetics*, Vol. 1, Atlanta, Georgia, USA, March 1998, pp. 185-190.

Gourc, J.P. and Faure, Y.H. (1990), "The Soil Particle, the Water and the Fibre, a Fruitful Interaction now Controlled", General Report, *Proceedings of the Fourth International Conference on Geotextiles, Geomembranes and Related Products*, Vol. 3, The Hague, The Netherlands, May 1990, pp. 949-971.

Holtz, R.D., Christopher, B.R. and Berg, R.R., (1997), "*Geosynthetic Engineering*", Bitech Publishers Ltd., Richmond, BC, Canada, 451p.

Koerner, R.M. (1998), "*Designing with Geosynthetics*", 4th edition, Prentice Hall, 761p.

Raymond, G.P. and Giroud, J.P., Editors, (1993), "*Geosynthetics Case Histories*", ISSMFE, March 1993, 277 p.

Thiel, R.S. (1999), "*Personal Communication*", February 1999.

Tisserand, C., Breul, B. and Herment, R. (1997), "Le barrage d'Ortolo et ses prédecesseurs: retour d'experience — Feedback from Ortolo Dam and its Forerunners", *Proceedings of Rencontres 97*, Reims, France, October 1997, pp. 135-140. (in French and in English)

Van Zyl, D. (1993), "*Personal Communication*", July 1993.

Williams, N., Giroud, J.P., and Bonaparte, R. (1984), "Properties of Plastic Nets for Liquid and Gas Drainage Associated with Geomembranes", *Proceedings of the International Conference on Geomembranes*, Vol. 2, Denver, Colorado, USA, June 1984, pp. 399-404.

Zornberg, J.G. and Giroud, J.P. (1997), "Uplift of Geomembranes by Wind — Extension of Equations", *Geosynthetics International*, Vol. 4, No. 2, 1997, pp.187-207.

Managing Litigation Risk: Actions Geosynthetic Manufacturers Can Take to Minimize Litigation Exposure

John V. Snyder, Esquire
Arlington, Virginia, USA

INTRODUCTION

This paper will identify steps product manufacturers can take to reduce the likelihood of getting sued in connection with a construction project design, construction or product failure. There are two basic principles to keep in mind in trying to manage litigation risk.

First, we must understand that people and companies get sued not because a failure IS their fault, but because the failure MIGHT be their fault. Failures on a project involving geosynthetic products present difficult questions of the cause, and resulting responsibility. Is it the product, the installation, or the design? Is it an intervening outside factor? Is it a combination of factors? Who is responsible for the design? For the selection of the product? For the installation means and methods? For the installation workmanship? At the start of litigation, the plaintiff (perhaps the project owner, the government, or a victim of catastrophic failure) does not know the answers, and thus will try to bring every potentially responsible party into the litigation. In addition, every party will seek to shift responsibility to others, adding parties to the litigation that it asserts are responsible. Thus the guilty, the potentially guilty and the innocent bystanders are likely to be brought into the litigation. Actual responsibility and liability are not determined until the END of the lawsuit.

Second, once your company is brought into litigation as a defendant, only bad things can happen until you get out. No one wins, it is just a matter of who loses the worst. At the end of the day, your company may be vindicated, but only after incurring legal fees, other litigation costs, and the distraction to your business. And no matter how certain of success you may be, the risk of losing (and losing big) remains as long as your company is a party to the case. Often, the best approach for a defendant is not to try to win, but to try to stop playing.

With these principles in mind, here are steps your company can take to reduce the chance of getting sued, and to try to get out of the lawsuit if you are sued.

LIMIT EXPRESS WARRANTIES TO THAT WHICH YOU CAN CONTROL

The safest warranties are those that warrant specific, objective facts about the physical properties of the product--what it IS, rather than what it will DO. Warranties of performance carry the risk inherent in predictions of the future. However, performance is ultimately what the customers seek, and sellers must often assume such risk to distinguish their products from their competitors'. If performance criteria are warranted, risk can be limited by precisely stating the most precise and objective standard practicable. Reasonable tolerances (measured by reasonable and objective standards) should be specified, and the standards should be stated. The specificity limits risk by allowing the manufacturer to manage its production process to the standard. In addition, if the standard is ambiguous or loosely defined, another party may have a contrary, more stringent, but reasonable interpretation. The ambiguity can thus create a conflict that may ultimately be resolved against the manufacturer. Keep in mind that if both interpretations are equally reasonable, ambiguities are resolved against the drafter of the ambiguous language.

Warranties should be written with close attention to what the manufacturer can actually control after the manufacturing process is completed. Often this means physical control of the product. If the manufacturer can only guarantee the condition of the product when shipped or delivered, (due, for example, to deterioration from improper storage), the manufacturer should expressly limit the warranty to the point of shipment or delivery.

Similarly, a warranty for a product that applies when the product is "properly installed" involves the manufacturer in a process (installation) over which it has no control. If a problem arises, the manufacturer may be required essentially to demonstrate that the installation is not proper. The manufacturer must rely on its ability to distinguish--and prove--the difference between proper and improper installation after the fact. Remember that in litigation, this decision is ultimately made at the end of the trial--after the cost and distraction has been incurred--by people who are not familiar with the technology.

Manufacturers will sometimes actually expressly guarantee both the product and the installation if the installation is performed by certified installers. This guarantee is appealing to the project owner, who wants someone to hold responsible in the event of a failure, instead of being caught between the mutually exclusive denials of the installer and manufacturer. Manufacturers choosing this approach must ensure that the certified installers are properly trained, and financially strong enough to assume responsibility in the case of installation errors.

Ultimately, manufacturers will decide the contents of their written warranties at high levels within their companies, often with the advise of legal counsel. This is as it should be. The manufacturer will do whatever it can to ensure that its written warranty imposes no more risk than the manufacturer is willing to assume and manage.

The manufacturer will assume that its warranty document states the full extent of its warranty obligations. However, on the contrary, suppliers of construction material often unwittingly (and unwisely) guarantee particular results or product features above and beyond those stated in the company's "official" warranty document through express warranties enforced by the Uniform Commercial Code ("UCC"). Ideally, a manufacturer will provide a warranty of conformance to the product specification and against defects in the product (within applicable tolerances). However, under UCC Article 2-313, the manufacturer will also be legally responsible for affirmations of fact related to the goods made on its behalf. Such affirmations would include statements regarding strength, product compatibility, or extended performance capability. The affirmations need not be written to be binding, and specific words or intent to provide a warranty are not necessary. A seller of goods warrants the goods will conform to its statements about the goods by making the statements. A seller is wise then, to limit the scope of the statements it makes about the goods.

The best approach to avoiding unintended express warranties is two-fold: First, train sales representatives to concentrate on the specific, objective facts about the product that can be safely guaranteed. Consider each statement in a sales presentation to be a warranty. As an exercise, say the phrase "I guarantee. . ." before any statement you might want to make to a prospective customer. If you feel uncomfortable about the statement, perhaps it should not be part of the presentation. This is not to say that the statement is dishonest. You may think and believe the statement to be correct, but are you sure enough to guarantee it?

Second, disclaim express warranties other than those the company is prepared to make in writing. This disclaimer should be included in the contract or order acknowledgement, and in the written warranty often provided to the customer. The disclaimer words to the effect of "seller hereby disclaims and excludes all warranties, express or implied, except the warranties specifically stated in [reference to official warranty]." The disclaimer is especially important to because a company cannot control everything its sales representatives may say. More importantly, a company cannot control everything that the customer may later claim was said.

Express warranties are also created by descriptions (labels) and samples. The seller warrants that the goods will conform to the description and to samples provided. The seller must ensure that it can comfortably guarantee the description, and that the sample is representative.

DISCLAIM THE IMPLIED WARRANTY OF FITNESS FOR A PARTICULAR PURPOSE.

Often the cause of a product failure claim is not the product itself, but that the product has been used in a manner for which it is not suited. A manufacturer would like to be able to rely on the principle that the manufacturer is responsible only to provide product. Others are is responsible to specify particular products for particular applications. If the product is not proper for the particular application, the problem is not the product; the problem is the choice of product.

This argument is essentially correct, and the wise seller, having read the previous section, has carefully avoided expressly stating that the product is suitable for the particular application. Nonetheless, the seller may have provided a warranty under the circumstances. Under UCC Article 2-315, the seller gives the buyer an implied warranty that the goods sold are suitable for a particular purpose if the seller knows the buyer is relying on the seller's skill or judgment to select suitable goods. The buyer usually has a specific requirement, and often asks the seller to recommend one of its products that will meet the requirement. Similarly, the seller may know, or have reason to know the purpose for which the product is being purchased. In such a situation, the seller has warranted to the buyer that the product is suitable for the particular application or purpose.

If it turns out the product is not suitable for the purpose, the seller is responsible. The product may be perfect in every way, but in that situation, the seller is still liable because the law imposes responsibility for choice of the product on the seller. For example, if buyer asks for something to paint a concrete deck, and the seller hands him a can of paint and rings up the sale, (even without saying anything), the buyer ought to be able to rely on the selection of the product as suitable for painting concrete. Once implied by the circumstances, the warranty of fitness for a particular purpose applies unless excluded. However, this warranty will not apply when the seller does not know the buyer's purpose (such as when a product is purchased without reference to any particular application), or when the product is used for a purpose that could not be reasonably anticipated.

How does the seller avoid responsibility for the product selection? By disclaiming the implied warranty of fitness for a particular purpose. To be effective, the disclaimer must be in writing and must be conspicuous. Expanding on the disclaimer example above, "seller hereby disclaims and excludes all warranties, express or implied, including any implied warranty of fitness for a particular purpose, except the warranties specifically stated in [reference to official warranty]." In addition, as a matter of prudent business practice, the manufacturer should make it clear that it is not responsible for design or product selection. This can be done through a disclaimer on sales literature and specification sheets, and as part of presentations to potential customers.

DISCLAIM THE IMPLIED WARRANTY OF MERCHANTABILITY

Like the implied warranty of fitness for a particular purpose, the implied warranty of merchantability is imposed by the law (UCC 2-314) in certain circumstances, even in the absence of express warranties. The implied warranty of merchantability imposes responsibility on the seller that the goods will pass in trade, that they are fit for their ordinary purpose, and that they will conform to affirmations of fact on the label. These warranties should not be unreasonable for a reputable manufacturer.

However, UCC 2-314 also imposes other implied warranties "from course of dealing or usage of trade." These standards are not precise, and are open to differing interpretations, especially in an industry where standards are evolving. Such differing interpretations of industry standards are generally resolved through expert testimony at trial (meaning additional cost and unpredictability of the outcome).

To avoid potential responsiblity based on the unstated expectations of buyers or others in the industry, this implied warranty should be disclaimed. Expanding on the disclaimer above, "seller hereby disclaims and excludes all warranties, express or implied, including any implied warranty of merchantability or fitness for a particular purpose, except the warranties specifically stated in [reference to official warranty]." To be effective, the disclaimer must be in writing, be conspicuous and must mention merchantability.

IF YOU MUST GIVE ADVISE, ONLY GIVE GOOD ADVISE

Often manufacturer's representatives will advise owners representatives or installation contractors on proper installation techniques or other such matters related to use of the product. Training on proper techniques, and giving installers the benefit of broader experience is is useful and appropriate, both for customer relations, and to limit problems that could result in claims. Reducing problems due to improper installation reduces the number of problems that might be incorrectly blamed on the product itself.

However, bad advise can be worse than no advise at all. The company may not be directly liable for the advise, similar to the potential exposure of design professional for a defective specification. But if a poorly trained representative suggests installation methods that turn out to be faulty, the manufacturer cannot credibly criticize the installation as the source of the problem. (Similarly, observation of installation work in progress can create the negative inference that the manufacturer approves of the installation, because it did not object.) Moreover, arguments have been advanced that the advise on installation is really part of the product (service with the sale). Under this argument, the manufacturer would be liable for defective advise as it would be for a defective product.

AVOID "IMPROVING" THE SPECIFICATION

A manufacturer suggesting a change in the specification and substitution of an alternative product for the product specified may assume significant design responsibility, and lose protection afforded it by compliance with the specification. By substituting products (even if the substitution is approved) the seller is warranting that the product is fit for the particular purpose under UCC 2-315. In addition, the seller is no longer able to use the specification as a defense. The owner and engineer are responsible for the design. Those who follow a specification provided to them are not responsible, with rare exceptions, if the design turns out to be defective. Substitution of products negates that defense, as the manufacturer/supplier is not following the specifications.

EXERCISE QUALITY CONTROL OVER QUALITY CONTROL DOCUMENTATION

Mistakes in quality control documentation are never forgiven in litigation. The mistakes can and will be used against you to attack the credibility of the entire quality control process, the integrity of the products and manufacturing methods, and even the integrity of the individuals involved. Faulty quality control documentation (even of product not directly associated with a failure) can be used to undermine the credibility of the quality control process for the product that is alleged to be defective. It can also create the impression that quality control is not important to the company, and even in some cases create the impression that the company is deliberately misleading customers by providing the information.

AVOIDING EXPOSURE IN "SHOTGUN LAWSUITS"

In "Shotgun Lawsuits," a plaintiff may sue multiple defendants. Responsibility for the underlying failue, injury and damages is unclear. Defendants often assert claims against each other, or bring in other parties that they claim are responsible for the problem. For example a contractor accused of using defective material may bring the supplier of the material into the lawsuit claiming that if it (the contractor) is responsible for using defective material, is it only because the supplier provided defective material. Counterclaims and cross-claims multiply, with fingers pointed in every direction. The goal is to get out of the lawsuit as soon as possible, but how?

SOLVE THE PROBLEM

Solve the problem before it grows into a lawsuit. Construction failures do not resolve themselves through the passage of time. Leaks turn into slope failures. Disruptions turn to delays and into delay damages that increase and ripple through a project. And the opportunities to resolve problems diminish with the passage of time.

The first opportunity to resolve the problem involves industry professionals--engineers and contractors who know the business, know the technology, and are trained to work in a collaborative problem-solving environment. The second opportunity involves industry professionals and their lawyers, who will get paid by the hour to learn more about the business and the technology, and who are trained to work in an adversarial environment. The third opportunity involves industry professionals and their lawyers, and a judge and jury who know nothing about the business or the technology. They are learning in the courtroom -- in part from the other parties' lawyers.

Take control of the problem. Try to solve it on engineering terms before it turns into a claim. Immediate resort to a defensive, uncommunicative, or hostile posture is counterproductive, and is not necessary to protect your interests. Investigate the situation; gather as much information as possible.

If the issue cannot be resolved on engineering or business terms, four elements of a defense strategy can be applied to expedite resolution of a "shotgun lawsuit" --

- Aggressive Motions Practice,
- Targeted Discovery, and
- Mediating Early.

The strategy for each case depends of course, upon the particular facts of the case and the applicable law, but these elements can serve as general principles to guide management of the litigation.

AGGRESSIVE MOTIONS PRACTICE

United States federal courts and most state courts allow parties to file motions to dismiss, or for summary judgment. A motion to dismiss will assume that the plaintiff's claim is factually correct, but argue that there is no legal basis on those facts to sustain the claim. Similarly, technical or procedural defenses, like the jurisdiction of the court or the statute of limitations (time for filing suit) may be asserted. Motions are resolved by the judge without a trial, and can resolve the case at the outset. Motions for summary judgment may be made on the basis of undisputed facts; a party will argue that on those facts, it is entitled to judgment in its favor, without going through a trial.

At the time a lawsuit is filed, the defendant should conduct a thorough analysis of every potential motion. Aside from legal procedural issues, a motion may be based on specific disclaimers in the contract between the parties, on notice provisions, or on other legal defenses. For example, contracts often limit the seller's liability to the buyer to the amount of the purchase. If the seller is sued by the buyer for more than that amount, the seller should file a motion to dismiss the claim to the extent that it exceeds the amount of the sale. The seller's liability could thus be immediately limited, perhaps making the lawsuit no longer economically wise for the buyer.

Such defenses are not always available. (It depends on the contract.) But parties and their counsel must actively search for them at the beginning of litigation--not later on after thousands of dollars have been spent in discovery. Such motions should be made at the beginning of the process, before extensive legal fees and other expenses are incurred. This is the time when disclaimers of warranties, if any, should be asserted.

A second level of analysis is to ask "why am I here?" There are three elements to every claim:

1) a duty on the part of the defendant to the claimant,
2) breach of that duty, and
3) injury proximately caused by the breach.

The defendant can defeat a claim by demonstrating that any one of these elements is missing. When challenged, the claimant must allege specific facts in support of each necessary element of its claim. If a defendant's analysis indicates that an element of the claim may be missing, it should raise the issue immediately, not wait until the trial.

For example, a project design engineer might be sued for negligence by the owner arising out of a slope failure. The project engineer might decide to assert a claim against the manufacturer of material specified in the slope construction (sold to the owner) for breach of warranty, claiming that the product is defective. Putting this claim through the analysis above reveals a significant problem for the engineer. The manufacturer owes the engineer no duty. Its contractual (and warranty) duty is to the owner.

The manufacturer would file a motion to dismiss the engineer's claim, asserting that an element of the claim is missing. In response, the engineer would have to identify the duty owed it by the manufacturer. The motion can be brought right away, because the problem with the claim is apparent in the claim itself.

TARGETED DISCOVERY

Discovery is the process by which parties can obtain information from the other parties to the lawsuit and from independent witnesses and others with relevant information. Discovery is used to prepare for trial--to learn everything about the case. But discovery also imposes substantial costs in examination of documents and other evidence, and sworn depositions of witnesses.

"Targeted discovery" is discovery aimed at demonstrating that one of the elements of the claim is missing. For example, if the claim is that particular material did not meet specifications, causing the slope failure, targeted discovery (through an interrogatory in this case) would ask the claimant to state--specifically--the nonconformity of the product. If the claimant cannot identify what is non-conforming about the material (thickness, shear strength, composition or some other product attribute), the claim may be vulnerable to a motion for summary judgment.

A motion for summary judgment is essentially a request for the judge to rule in your favor, without a trial, because the undisputed facts show that you would win as a matter of law. It can be supported by factual material obtained during discovery. In case of the defective material claim, the motion would argue that the undisputed facts demonstrate that nothing is wrong with the material--because the claimant cannot identify anything wrong with it. Therefore, the claim that the material is defective should be dismissed. The claimant would have to have to show that there is a dispute regarding the facts by showing that something is wrong with the material. The claimant does not have to win the dispute, just show that there is a dispute that must be resolved at trial.

The timing of the targeted discovery and motion for summary judgment is important because getting out of the case sooner is better than later-- and to give the unprepared claimant less time to come up with evidence in support of its claim. There is no real down-side to the targeted discovery. If the discovery reveals that the other side does already have evidence in support of its claim, you can proceed to examine that evidence and prepare for trial (or if the evidence is very strong, to prepare to settle.)

MEDIATE EARLY

Mediation is usually the best process to facilitate settlement of a case. Mediation is a process in which the parties try to negotiate a settlement with the assistance of a neutral third party mediator (often called a neutral advisor). The parties, each represented by an individual with authority to settle the case and by their lawyers, will meet to discuss the case. The mediator will attempt to broker a settlement by ensuring communication and by transmitting settlement offers between the parties. Often the parties will make a summary presentation of their case to the other parties and the mediator in an attempt to demonstrate the strength of their case.

The advantage of mediation as an alternative dispute resolution process is that the parties retain control of the settlement. In a court trial or arbitration, the parties give up control of the outcome to others. The parties can resolve the case as a business decision, not gamble on the result that might come out of a trial or arbitration.

A common mistake is to put off mediation to right before the trial. (In many states and federal courts, the parties may not proceed to trial without first going through mediation.) Mediation should be scheduled early in the process, not later, to fully take advantage of the opportunity to settle.

The first reason to mediate early is to avoid the cost of extensive discovery before it is incurred. The second reason is that relations among the parties and their attorneys--even among professionals--are likely to be better at the beginning of the case. The atmosphere for settlement is better.

Many will resist mediating a case too early, as they will want to conduct discovery before the mediation to strengthen their own cases and to examine the other parties' evidence. This is a valid point, however, discovery can be scheduled to be tailored to what the parties need for mediation, with the option to go into more depth later. Scheduling mediation early in the process forces the parties to get serious, to get prepared, and to focus their attention on the critical issues of the case. Preparing for the brief presentation generally allowed in mediation forces the parties to focus on the very best elements of their case and the worst elements of the other parties.

The mediation process will reveal the most important issues of the case. Even if the mediation is not successful, the parties can go forward with the issues more focused for discovery and other proceedings to follow. In addition, the process opens communication among the parties, and creates an environment in which the gap between the parties can be narrowed. (It is not unusual for a case to settle a few weeks after mediation has adjourned without a settlement.)

CONCLUSION

Ultimately, success in avoiding litigation and resolving litigation that cannot be avoided comes down to control. Control warranty exposure by limiting warranties to controllable physical properties or performance standards. Control the purchaser's expectations by avoiding (and disclaiming) express warranties and warranties implied by law. And if all else fails, control the litigation through aggressive motions practice, targeted discovery and mediation early in the process.

PRODUCT LIABILITY LITIGATION: A PLAINTIFF'S PERSPECTIVE

NICHOLAS C. MORAITAKIS
MILLS, MORAITAKIS & KUSHEL, USA

ABSTRACT

The purpose of this paper is to give insight into the manner in which counsel for plaintiffs approach the pursuit of product liability litigation. The author has been involved in one relatively significant product liability matter involving a geosynthetic material and an effort will be made to tailor the remarks as it may pertain to geosynthetic manufacturers. Nevertheless, a general discussion of product liability litigation and the plaintiff's approach to such litigation is generally pertinent to manufacturers and distributors of products of any nature.

Any product which is ultimately subject to exposure to the general public for use or effect is subject to become the object of product liability litigation. The pre-suit considerations which counsel for a prospective plaintiff must contemplate will be reviewed as well. Hopefully, once an examination is complete concerning these pre-suit considerations, a better understanding regarding the lack of desirability of "shotgun lawsuits" will be evident.

I. COMPONENTS OF PRODUCT LIABILITY CASES

Attached to this paper is a copy of the principal product liability statute for the State of Georgia. It is delineated as Official Code of Georgia Annotated Section 51-1-11. In reviewing the statute in detail, it sets forth the fact that "privity" is not necessary to support a tort action against a manufacturer of goods. "Privity" is the relationship of purchaser and seller. In other words, in order to bring an action for product liability negligence, the plaintiff need not have been the individual who made the purchase from the manufacturer or distributor.

This statute and those similar to it across the country are also known as "strict liability" statutes. They permit basically a cause of action against a manufacturer or a dealer of a product without regard to privity and basically without regard to proof of fault, as long as the injury occurs because the product, when sold, was not merchantable or reasonably suited for the use for which it was intended. In other words, if the plaintiff can prove a defect in the product which existed at the time the product was sold, no proof of negligence is actually necessary.

This particular statute in Georgia has some quirks in it which may not apply in other states. For instance, Subsection (b)(2) contains a ten year statute of repose, which arose in Georgia as a result of the tort reform movement. The effect of a ten year statute of repose, which may in fact be pertinent to geosynthetic manufacturers, is that no action can be brought against the manufacturer of a product for injuries stemming from a product which is over ten years old. This rule imposes a duty upon the plaintiff's lawyer to quickly examine the manufacture date of products which have caused injury when a potential plaintiff seeks to retain counsel to pursue a potential product liability claim. Subsection (c) of the statute provides opportunities to circumvent the statute of repose limitations.

A copy of the statute is provided and reviewed simply for a general understanding of what these statutes look like. These statutes do not necessarily limit the avenues of approach which may be taken toward product liability cases, as theories of product liability are refined and often created by decisions from appellate courts. Statutes such as product liability laws, seldom speak clearly to every consideration and as a result, appellate courts are called upon to rule upon a statute's meaning. In doing so, these appellate courts oftentimes create new causes of action or new theories for pursuit of older causes of action.

A claim for product liability can come in several different forms, depending on the circumstances. The plaintiff's lawyer must examine the facts as presented by a potential client to make a determination as to whether or not any particular theory is available. These theories generally are negligent manufacture, negligent design, negligent failure to warn, and strict liability. Each of these theories will be addressed briefly at this point.

Negligence in manufacturing is probably the least common claim found. In order to prove a case based on negligent manufacture, the plaintiff needs to find a circumstance where the product at issue was in fact manufactured incorrectly or contrary to specifications of the manufacturer. These types of cases arise only when something unusual happened on the assembly line or during the course of manufacture where the product was in fact not put together as it was supposed to be. If such a negligent act can be found and proven, and if damages do in fact stem from such a negligent act, a cause of action for negligent manufacture is in fact the easiest product liability action to pursue. It should be reiterated that these circumstances are the rarest in the product liability litigation arena.

Negligent design is probably the most common theory pursued by plaintiffs in product liability cases. Attacking the design of a particular product can be difficult and expensive. It requires from the plaintiff's perspective an evaluation of the evolution of the product design by the designer/manufacturer. Generally speaking, this cannot be done until suit is filed and extensive discovery is pursued. In propounding interrogatories and requests for production of documents to a defendant manufacturer, the plaintiff is forced to pursue internal documentation including design drawings and memoranda over a lengthy period of time. As one can imagine, defendant manufacturers resist producing this material with significant vigor, and the plaintiff is generally forced to pursue orders from the court to compel production of materials which may be reasonably calculated to lead to admissible information. The downside of pursuing the negligent design case is that the plaintiff is not always in the position of knowing how strong the case for negligent design is until well into the discovery process. Clearly, this puts the plaintiff in a position of disadvantage in evaluation of the litigation.

Having been involved in a claim of defective design in a geosynthetic product, the author of this paper can state that it is a difficult proposition to prove such claim in this specialized area. In asserting proof of a design defect, the plaintiff must employ experts in the field in order to first assist in understanding the nature of an appropriate design and the manner in which the subject design may in fact be defective. Surely the employment of such experts costs significant money and time, and it can oftentimes be difficult to find people in the field who are willing to attest to deficiencies in the design of others also in the same field. Therefore, the plaintiff's task in pursuing negligent design cases, particularly in limited and specific environments, can be very difficult.

The negligent design issue is generally the same issue which comes up in strict liability cases. The plaintiff is attempting to prove that at the time of manufacture, the product was defective, not because it was negligently manufactured, but because the design itself was defective. All jurisdictions in the United States recognize that under negligence or strict liability principles, a manufacturer first has a duty to use reasonable care and skill in designing a product so that it is reasonably safe for its intended use and other foreseeable uses. This standard has been stated by many authors on the subject of negligence. For instance, Harper and James on Torts, at 1541, 1584, Section 28.4, 28.22 state the standard as follows:

> The maker of an article for sale or use by others must use reasonable care and skill in designing it and in providing specifications for it so that it is reasonably safe for the purposes for which it is intended and for other uses which are reasonably foreseeable.

Generally speaking, when a product is being put to a use at the time of injury, which was not the use originally intended by the manufacturer, liability of the manufacturer depends upon the foreseeability of the use to which the product was being put. Likewise, it is true that the maker of an article for sale or use by others must use reasonable care and skill in designing it so that it is reasonably safe for the purposes for which it is intended and for other uses which are reasonably foreseeable. On this point, the case of Ford Motor Co. v. Stubblefield, 171 Ga. App. 331, 319 S.E.2d 470 (1984) is very instructive.

Most states have now adopted what is known as a "risk utility" test in its approach to design defect cases. This was adopted in Georgia in 1994 in the case of Banks v. ICI Americas, Inc. The Georgia court found that the overwhelming majority of states recognize the general consensus regarding the utilization in design defect cases of a balancing test whereby the risks inherent in a product design are weighed against the utility or benefit derived from the product.

The court enumerated certain factors which a jury should consider in making a determination as to whether or not a product was defective. These factors included such things as the usefulness of the product, the severity of the danger proposed by the design, the likelihood of that danger, the avoidability of the danger, the user's knowledge of the product, publicity surrounding the danger, and efficacy of warnings.

Most importantly as it relates to this risk utility analysis, manufacturers must be aware that simple proof of compliance with industry-wide practices, state of the art, or federal regulations does not necessarily eliminate liability for design of allegedly defective products.

The practical effect of most courts now having adopted the risk utility analysis for product liability cases is that when these cases go to trial, evidence will center around whether or not there was a safer alternative design. Other manufacturers in the field will become targets of information which will presumably lead to evidence that safer alternative designs existed and therefore the product at issue is in fact defective. It is not essential that a plaintiff prove that there was a safer design, but clearly such proof will be the approach that a plaintiff will pursue.

Negligence in failure to warn is another avenue utilized by plaintiff's lawyers to pursue product liability cases. It has been held in most states that a failure to warn of a latent defect is in essence a design defect. The classic example here is the failure of automobile manufacturers early on to warn of the dangers that air bags presented to children when deployed upon frontal impact. Thus, you see in more recently manufactured automobiles the explicit warnings contained on the sun visor, particularly in the passenger seat. A good deal of litigation is now being pursued regarding the sufficiency of those warnings now that they do exist.

These are the principal avenues of approach to potential product liability cases which plaintiff's lawyers must give thought to, and all of them are pertinent to manufacturers of geosynthetic products.

II. THE PURSUIT OF THE CASE

Products liability cases are extremely complex and difficult to pursue from a plaintiff's standpoint. In addition, they generally end up being quite expensive. Plaintiff's lawyers have become far more sophisticated than they used to be, and there now are data banks of information available as well as networks of communication between plaintiff's counsel throughout the country on the pursuit of various products. This is true in the area of automobile manufacturers, medical products manufacturers, prescriptive medicine manufacturers and the like. So far, the geosynthetic manufacturing industry has not been one which is viewed by the plaintiff's bar as a frequent target in products liability cases. Obviously, this is good for the industry. A brief and general discussion of how one pursues a products claim may be enlightening.

Once a complaint is filed, discovery usually accompanies the complaint. From the plaintiff's perspective it is imperative that the discovery include requests for all design drawings and accompanying documentation which led to the manufacture and design of the product at issue. Names of engineers involved in the design and review of the design will be pursued. Another important area of discovery will be knowledge of prior incidences involving similar designs. It is essential that a plaintiff's lawyer pursue these incidences whether they resulted in litigation or not. The better plaintiff's lawyers will also pursue this line of information independently. It is not uncommon for defendants to misstate their prior experience with similar incidences and clearly when one is caught in not coming forward with the absolute truth on this issue, it can be rather devastating in terms of its effect on a judge and a jury.

Once the initial "paper discovery" is done, the plaintiff's lawyer will then pursue discovery through deposition testimony seeking to depose those involved in the design and manufacture and those with knowledge of potential problems in the design, manufacture or failure to warn of the product at issue. Retention of experts to work with plaintiff's counsel to educate plaintiff's counsel as to the industry expectations and standards is a part of the upfront preparation for pursuing these types of cases. Under the federal rules of discovery, as well as rules of discovery in all of the states, there are two categories of witnesses. These classifications of witnesses are witnesses used for consulting purposes only and witnesses retained to give testimony at trial. The rules of civil procedure recognize a litigant's right to obtain an expert who may be used only as a consulting expert so that a litigant can be made aware of the pros and cons of his or her case. The names of consulting experts need not be provided in most instances and at the very least, they are not subject to deposition discovery. Once a decision is made as to whether an expert witness is going to testify at trial, then the decision will be made as to whose depositions will be taken. This leads us to the second class of expert and that is the testifying expert. Whoever the plaintiff or defendant is going to use to testify must be set forth in responses to discovery and those experts are subject to deposition prior to the trial to eliminate trial by surprise.

III. DAMAGES

There are a number of defenses which can be advocated by lawyers for the defendant manufacturers and many of these are valid defenses and sometimes they are not. Those will not be discussed in detail, but brief mention should be made of them before going into the issue of damages because oftentimes these defenses affect ultimate damage awards. Generally, the defenses can be categorized into simple denial of the existence of a defect and using the risk utility test to the advantage of the defendant, or the assertion of an "open/ obvious defense" and an assertion of comparative negligence by the plaintiff. Oftentimes, product liability cases arise as a result of a less than completely careful act performed by the plaintiff, and in these instances the defense generally raises comparative negligence and a defense of the fact that the plaintiff fell victim to an open and obvious problem or defect. These defenses and the comparative negligence doctrine can be problems for the plaintiff and clearly a hurdle which the plaintiff must prepare to clear.

Once having cleared those, proof of damages becomes as important for jury consideration as proof of a defect. In cases where death has ensued, most states permit the recovery of damages for the wrongful death and for pain and suffering. Generally, damages relative to wrongful death are measured by the economic full value of one's life and the non-economic value of life. On the other hand, some states, such as Florida, measure damages by the grief that the death has caused the survivor or survivors who may be bringing the claim.

The estate of a decedent can also bring damages for pain and suffering prior to the death. Where the death occurred as a result of an excruciating event, this can oftentimes be a significant element of recovery. Generally speaking, in wrongful death cases, most courts have determined that wrongful death statutes are punitive in nature and therefore punitive damages are not recoverable. However, in Georgia for instance, if there does exist in fact a claim for pain and suffering prior to death, that would give rise to the recovery of a punitive damage amount in the event the court deemed it appropriate.

In injury cases, one can generally recover for medical expenses, time lost from work, and pain and suffering. Items to be considered as elements of pain and suffering may be permanent disfigurement, as well as simple pain one suffers on a daily basis. Future inability to work may also be an element of pain and suffering.

IV. SHOTGUN LAWSUITS

This issue has been discussed previously and the author has submitted an article for publication on this before. This article is reprinted as part of this paper because it addresses the shotgun lawsuit issue which is, of course, of concern to many manufacturers, including geosynthetic manufacturers. Therefore, immediately following this paragraph is a reprint of the article which has been submitted previously.

> One irritant for geosynthetic manufacturers is the "shotgun lawsuit." By that, we refer to a lawsuit stemming from a significant failure where a number of parties are involved, including architects, engineers, contractors, municipalities and a geosynthetics manufacturer, whose product may have actually been a minor contributor to a large failure. How can geosynthetics manufacturers protect themselves from such lawsuits?
>
> First of all, in many states as a result of tort reform, it has now become riskier and more difficult for a plaintiff's lawyer to sue defendants. Many states now require that an expert affidavit accompany the complaint when suing even a manufacturer, if the manufactured product is the result of a design prepared by a registered professional, such as an engineer. In the State of Georgia, for instance, failure to include such an affidavit when alleging a complaint against a geosynthetic manufacturer may result in dismissal of the complaint.
>
> Furthermore, most state codes relating to civil practice, and certainly the federal court rules contain provisions whereby a plaintiff <u>and</u> a plaintiff's lawyer may be penalized financially for filing groundless or frivolous claims against another party. Thus, there are mechanisms in place which can be utilized to hopefully prevent clearly nonculpable parties from being dragged into lawsuits where they have no reason to be.
>
> Prior to the filing of a lawsuit, the information which is available to a lawyer for the plaintiff is very limited. There is no subpoena power and generally speaking there is no way to compel potential defendants to provide information. Therefore, the plaintiff is operating from a point of disadvantage. The tendency to sue everyone potentially involved and dismissing those nonculpable parties down the road is generally more strategically sound than suing only some of the parties and looking to bring others in later. Therefore, good advice to a geosynthetics manufacturer who wishes to protect itself from such "shotgun lawsuits" is to come forward with information of lack of culpability as early as possible. Communication with counsel for the plaintiff in an effort to show that the geosynthetic product was not a contributing factor to

the incident is a good idea. Good plaintiff's lawyers have no interest in suing corporate defendants who do not belong in their litigation. We all know that we are going to have our hands full with the lawyers representing the culpable parties.

CONCLUSION

This paper has attempted to address plaintiff's considerations in products liability litigation generally. The rules and considerations applicable in general product liability cases are certainly applicable to products liability cases involving the manufacture, distribution and sale of geosynthetic products. Personal injury litigation stemming from failures in design of geosynthetic products does not seem to have reached a level of prevalence. Naturally for the manufacturers, this is good. It is even better for those who may find themselves ultimately injured as a result of a potential defect in design of geosynthetic products. Thus, incidences of personal injury product liability litigation are not easy to cite, nevertheless, the general approach that is taken by counsel for plaintiffs in products cases is pertinent, because it would apply in the geosynthetics arena as well.

Official Code of Georgia Annotated 51-1-11.
When privity required to support action; product liability action and time limitation therefor.

(a) Except as otherwise provided in this Code section, no privity is necessary to support a tort action; but, if the tort results from the violation of a duty which is itself the consequence of a contract, the right of action is confined to the parties and those in privity to that contract, except in cases where the party would have a right of action for the injury done independently of the contract and except as provided in Code Section 11-2-318.

(b) (1) The manufacturer of any personal property sold as new property directly or through a dealer or any other person shall be liable in tort, irrespective of privity, to any natural person who may use, consume, or reasonably be affected by the property and who suffers injury to his person or property because the property when sold by the manufacturer was not merchantable and reasonably suited to the use intended, and its condition when sold is the proximate cause of the injury sustained.

(2) No action shall be commenced pursuant to this subsection with respect to an injury after ten years from the date of the first sale for use or consumption of the personal property causing or otherwise bringing about the injury.

(3) A manufacturer may not exclude or limit the operation of this subsection.

(c) The limitation of paragraph (2) of subsection (b) of this Code section regarding bringing an action within ten years from the date of the first sale for use or consumption of personal property shall also apply to the commencement of an action claiming negligence of a manufacturer as the basis of liability, except an action seeking to recover from a manufacturer for injuries or damages arising out of the negligence of such manufacturer in manufacturing products which cause a disease or birth defect, or arising out of conduct which manifests a willful, reckless, or wanton disregard for life or property. Nothing contained in this subsection shall relieve a manufacturer from the duty to warn of a danger arising from use of a product once that danger becomes known to the manufacturer.

Geosynthetics for Earthquake Hazard Mitigation

Yegian, M. K, Kadakal, U., and Catan, M.

Northeastern University, Boston, MA

ABSTRACT

Geosynthetic or related materials placed under foundations can absorb seismic energy, and hence transmit smaller levels of excitation to an overlying structure. This concept of using geosynthetics as foundation isolation can be a cost-effective way of mitigating earthquake hazards to civil engineering structures. The authors have been exploring the suitability of various synthetic materials for the purpose of foundation isolation. The dynamic interface properties of these materials are being investigated using a shaking table to identify the most promising material for this application.

To demonstrate the technical feasibility of using synthetic materials for foundation isolation, shaking table tests were performed. A single-story building model was placed on the shaking table and its response to harmonic and earthquake motions was measured. The accelerations and story drifts of the model building with and without foundation isolation were measured. The results from these tests demonstrate that using geosynthetics as foundation isolation reduced the column shear forces in the building model by as much as 70%. Associated with this reduction are slip deformations along the geosynthetic interface ranging from a 1 to 10 cm depending on the earthquake record and its intensity.

INTRODUCTION

During the past few years, significant advancements were made in our understanding of the dynamic interface shear properties of geosynthetic interfaces. Kavazanjian et al. (1991), Yegian and Lahlaf (1992), and Zimmie et al. (1994) have demonstrated that under dynamic shear excitations, slip deformations occur along smooth geosynthetic interfaces. As a result of such slip, the energy transmitted through the interfaces is limited. Thus, in a landfill application, seismically induced slip deformations along a bottom geosynthetic liner can result in reduced accelerations transmitted to the landfill waste.

This potential benefit of smooth geosynthetics reducing landfill or other structural response during an earthquake was first investigated by Kavazanjian et al. (1991) and Yegain and Lahlaf (1992). Their preliminary shaking table tests on smooth High Density Polyethylene (HDPE) and geotextiles showed that this concept of using geosynthetics to isolate a structure from incoming seismic waves had great promise.

The authors have been investigating the technical feasibility and practicality of using geosynthetic liners to mitigate the potential damaging effects of earthquakes to buildings with an initial grant from the North American Geosynthetics Society, and subsequently with a major grant from the National Science Foundation.

This paper presents selected results from shaking table tests that were carried out to identify a geosynthetic interface that is ideally suited for this new application. The paper also describes shaking table tests of a building model placed on a selected geosynthetic liner. The results from these tests are presented to demonstrate the benefits of utilizing a special geosynthetic liner as an energy absorbing system that can reduce building response during an earthquake.

FOUNDATION ISOLATION

Figure 1.a shows a typical structure founded on a soil profile experiencing earthquake-induced ground motions. In a conventional design, the foundation of the structure rests firmly on the soil. During an earthquake, because of the large friction between the foundation and the underlying soil, the ground motions are fully transmitted to the superstructure (the building above the foundation). This seismic energy then causes lateral distortion of the building and introduces shear forces in the columns.

To limit the seismic energy transmission to a structure, structural engineers have been developing mechanical devices referred to as base isolators. In a building application, a base isolator provides a discontinuity between a footing and the overlying column. Typically, a base isolator performs two functions: (1) It shifts the natural period of the building away from that of the earthquake (2) It provides additional damping to absorb the energy. Figure 1.b shows a schematic drawing of a building using conventional base isolators. Structural isolation systems have been used in a number of important buildings and bridges in the United States and Japan. During the 1994 Northridge earthquake, structures on base isolators, generally performed well (Hussain, 1994). However, at the present, the cost of installation and maintenance of such isolation systems is prohibitively high for their wide application in engineering practice.

The authors have been investigating the use of geosynthetic materials as seismic energy absorbing systems for application in earthquake hazard mitigation. A concept that is being

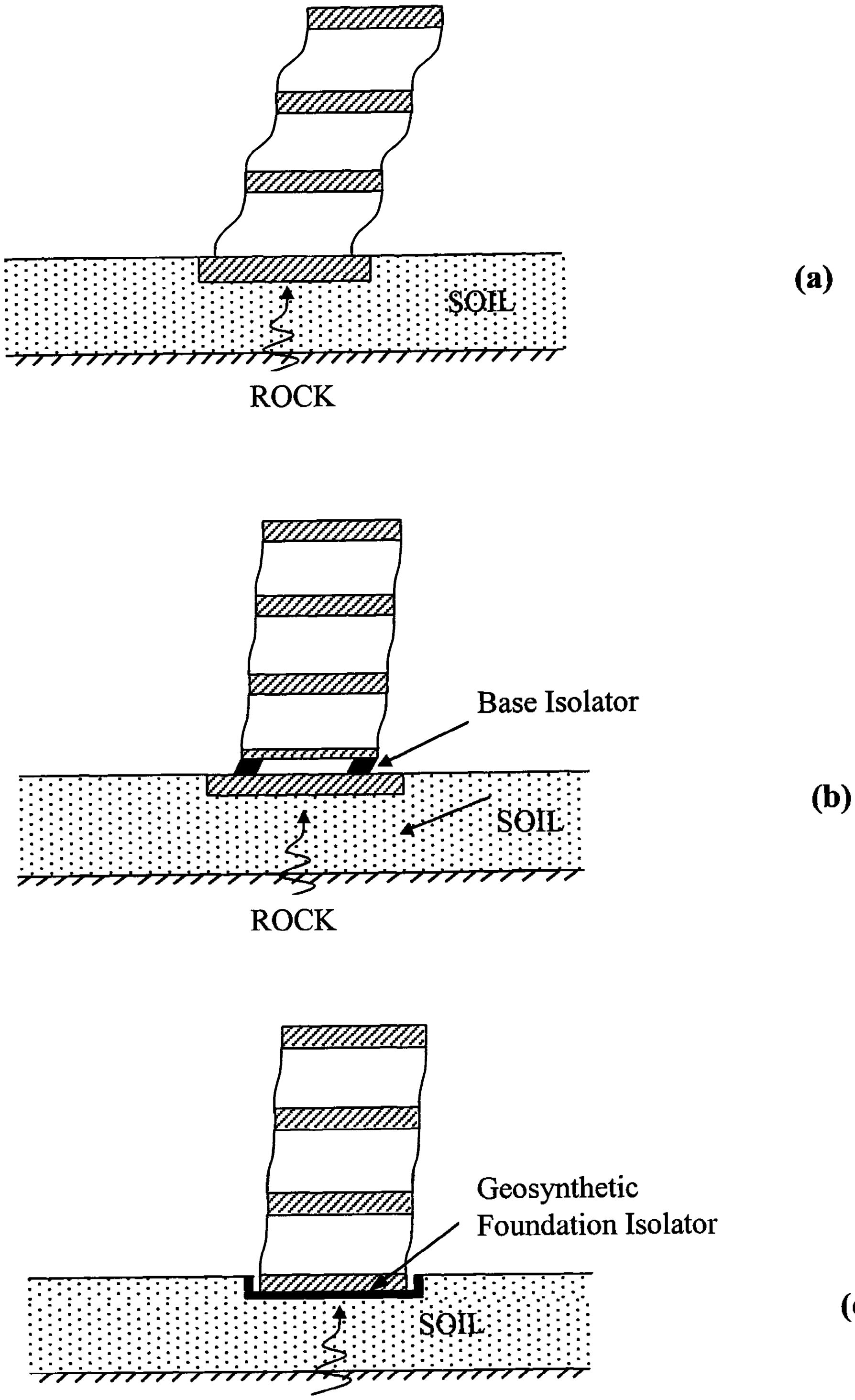

Figure 1. Seismic response of a typical building (a) founded on soil, (b) with base isolation, (c) with geosynthetic foundation isolation.

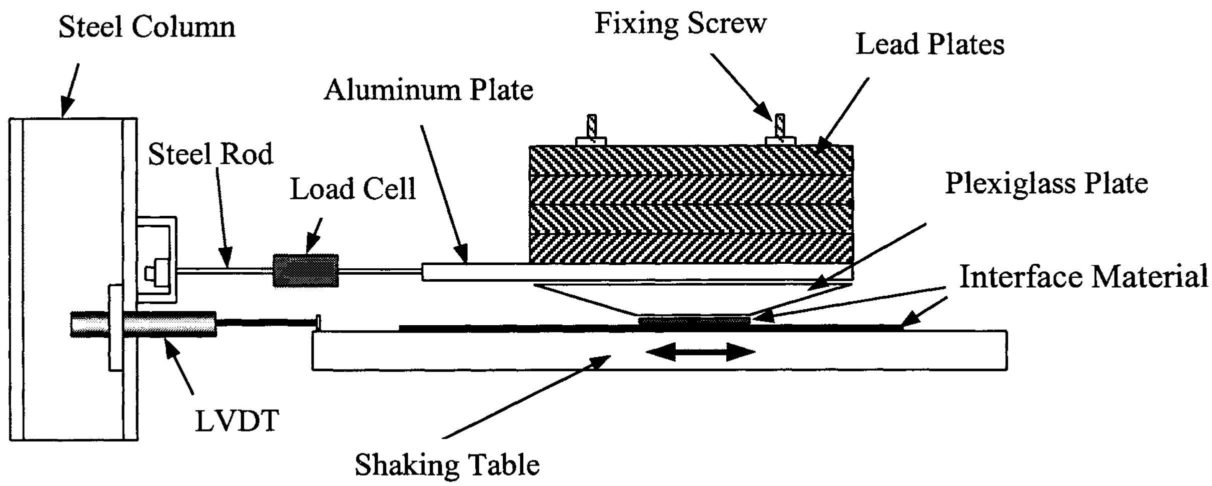

Figure 2. Schematic diagram of the cyclic load test setup.

evaluated for its technical feasibility is the use of horizontally placed smooth geosynthetics underneath building foundations that will absorb seismic energy, and thus transmit significantly smaller accelerations to the overlying structure. This concept hereafter referred to as **foundation isolation** is similar to base isolation except that, in this case, the entire building is isolated from the ground through the use of a geosynthetic liner. Figure 1.c shows a schematic depiction of the seismic response of a building that utilizes foundation isolation.

The following sections of this paper will present some of the results that demonstrate the technical feasibility of foundation isolation using geosynthetics.

SHAKING TABLE TESTS

The first task of this research was to identify geosynthetic interfaces that are suitable for use as foundation isolation. Initially, three interfaces were selected for testing based on earlier test results that showed that the interfaces had low dynamic friction angles. These interfaces were 1) Smooth HDPE/HDPE; 2) Smooth HDPE/Nonwoven spunbonded Geotextile; and 3) Polytetrafluoroethylene (PTFE)/PTFE. A special cyclic test setup was devised to investigate the response of these interfaces under varying conditions. Figure 2 shows a schematic diagram of the cyclic test arrangement. The bottom plate shown in Figure 2 is the top of a shaking table that was used to apply the horizontal shear along the geosynthetic interface tested. Tests were carried out by varying the normal contact stress, amplitude of displacement (slip) and the rate of slip. Under these different test conditions, the friction coefficients of the interfaces were measured and evaluated.

Figure 3 shows a summary results of friction coefficients as a function of slip rate. From

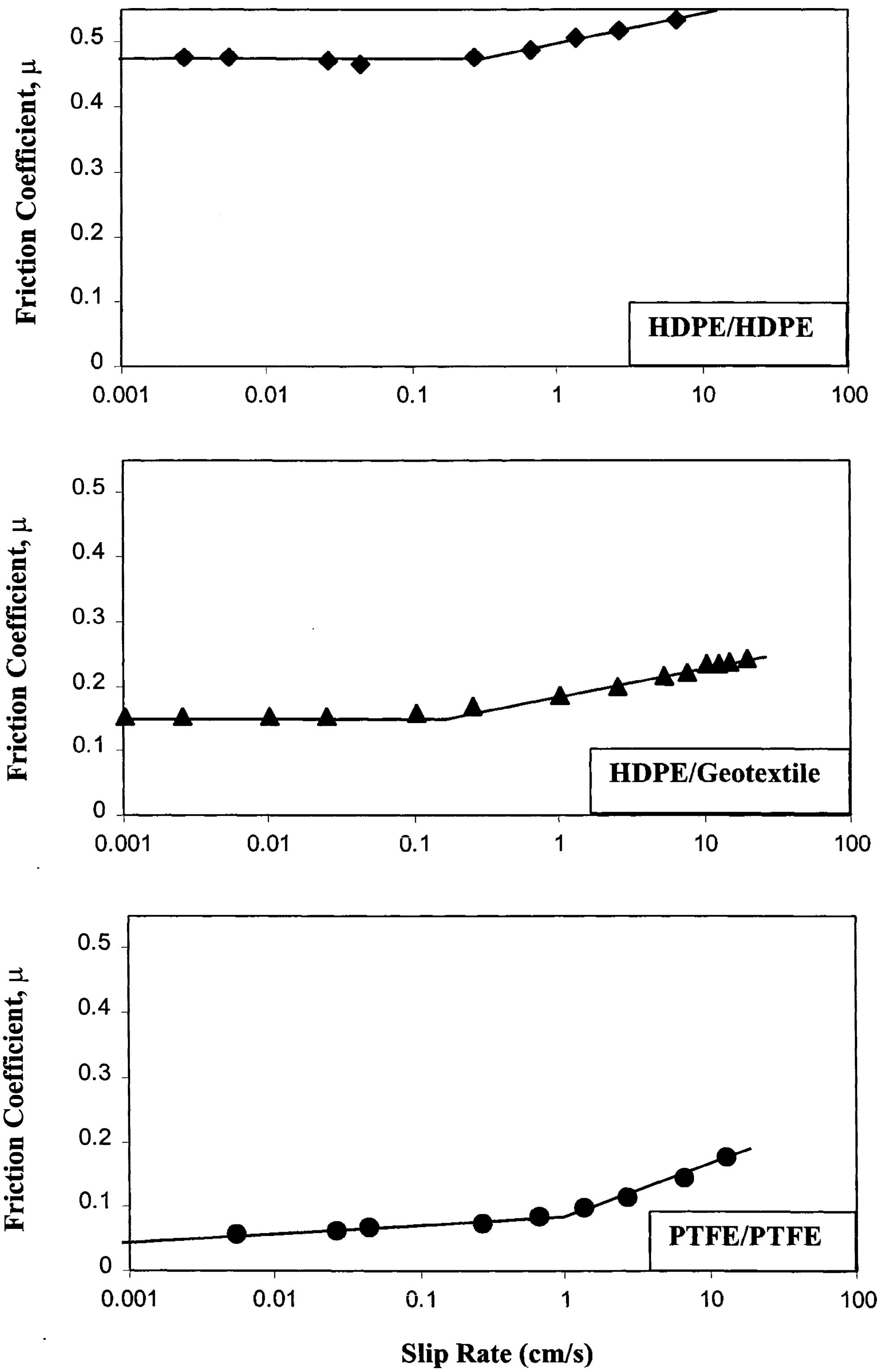

Figure 3. Friction coefficients as a function of slip rate, from cyclic load tests.

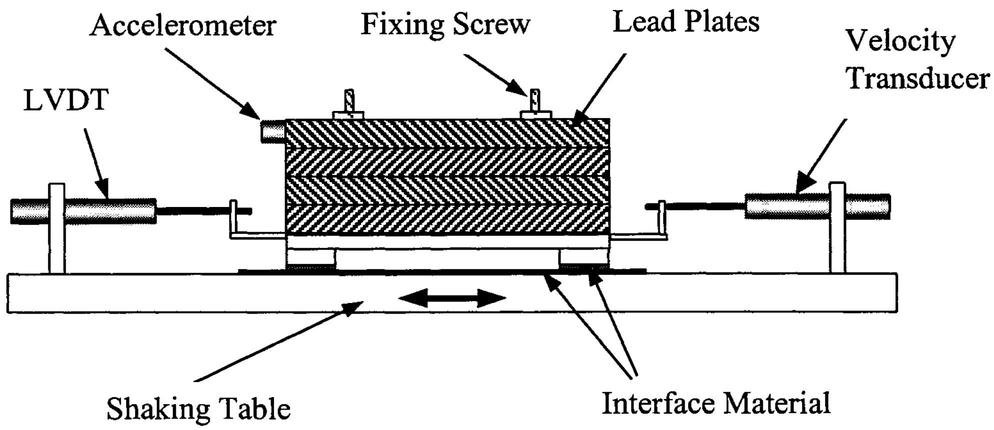

Figure 4. Schematic diagram of the cyclic load test setup.

these and other test results it was observed that the interface consisting of PTFE against PTFE had the lowest coefficient, about 0.06 at very small slip rates. Also, the test results showed that the friction coefficients for all the interfaces depended on the slip rate. At slip rates of larger than 2cm/s, comparable to rates expected during a moderate size earthquake, the friction coefficients increased substantially, especially for the PTFE/PTFE interface.

Similar behaviors were observed from rigid block tests. Figure 4 shows the schematic of the test setup in which a rigid block is placed on the interface and subjected to sinusoidal table accelerations. The response of the block was measured by an accelerometer, LVDT, and a velocity transducer. Tests were carried out at increasing levels of table accelerations.

Figure 5 shows results obtained from the rigid block tests carried out at 2 and 5 Hz table excitations. Again, it is observed that the PTFE/PTFE interface has the smallest transmitted acceleration of about 0.15g, at a base acceleration of 0.15g, after the initiation of sliding. Beyond a base acceleration of 0.15g the acceleration transmitted to the block slightly increased due to the effect of the slip rate. These shaking table test results are consistent with the cyclic test results shown in Figure 3.

It was concluded from the tests described above that the PTFE/PTFE interface is better suited for foundation isolation than HDPE/HDPE or HDPE/Geotextile interfaces. Yet, the transmitted acceleration through this interface is still relatively high at high slip rates for its application as foundation isolator. Furthermore, the velocity dependence of the friction coefficient (Figure 3) poses a difficulty for proper modeling of its behavior. Efforts were made to identify an alternate interface that has a friction coefficient as small as the PTFE/PTFE interface and is not so dependent on slip rate. Through interaction with geosynthetic manufacturers, we learned that the friction coefficient of a plastic material is influenced by its molecular weight. Research on the availability of different plastics resulted in the identification

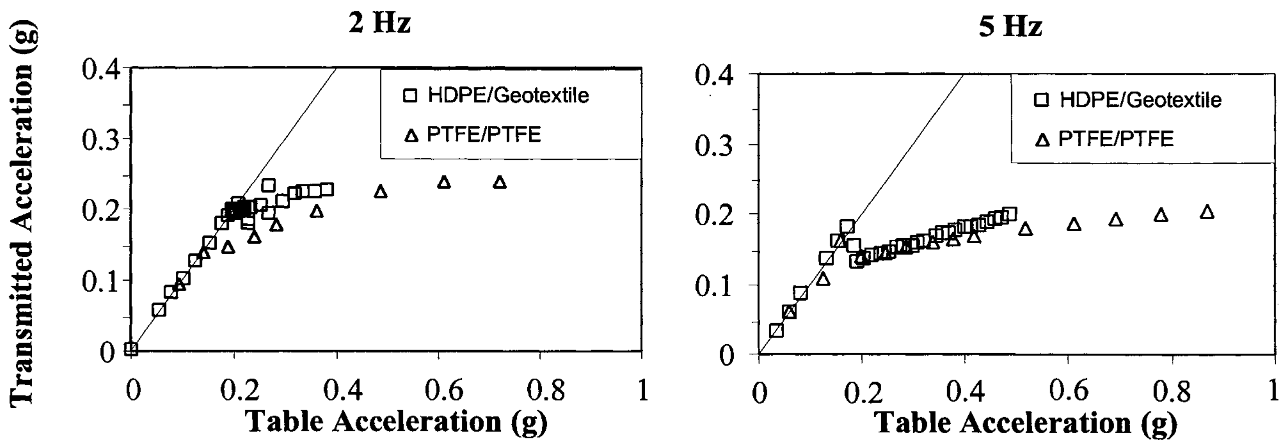

Figure 5. Accelerations transmitted through geosynthetic interfaces tested at 2 and 5 Hz sinusoidal table excitations.

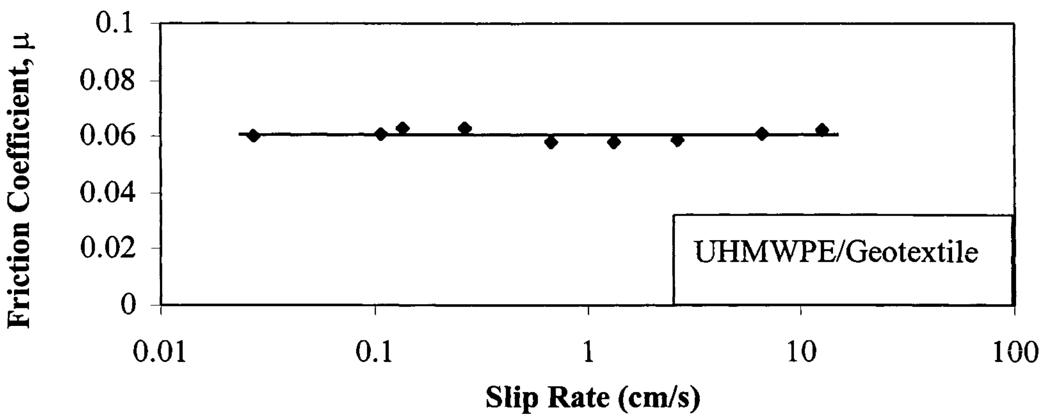

Figure 6. Friction coefficients of UHMWPE/Geotextile interface tested in the cyclic load apparatus.

of one that showed the best promise for application as foundation isolation. The interface thus identified is an Ultrahigh Molecular Weight Polyethylene (UHMWPE) and a nonwoven spunbonded geotextile.

Cyclic load and shaking table tests were conducted using an UHMWPE/geotextile interface, and the frictional characteristics were evaluated. Figure 6 shows sample test results from the cyclic load tests which indicate that the friction coefficient of the interface is quite low (0.06), and is nearly constant over a wide range of slip rates. In Figure 7, the slip rate dependency of the various interfaces tested is compared. Friction coefficient from each interface was normalized with its value (μ_0) measured at small slip rates (0.001-0.01 cm/s). Clearly, UHMWPE/geotextile is a superior interface that has a friction coefficient independent of slip rates.

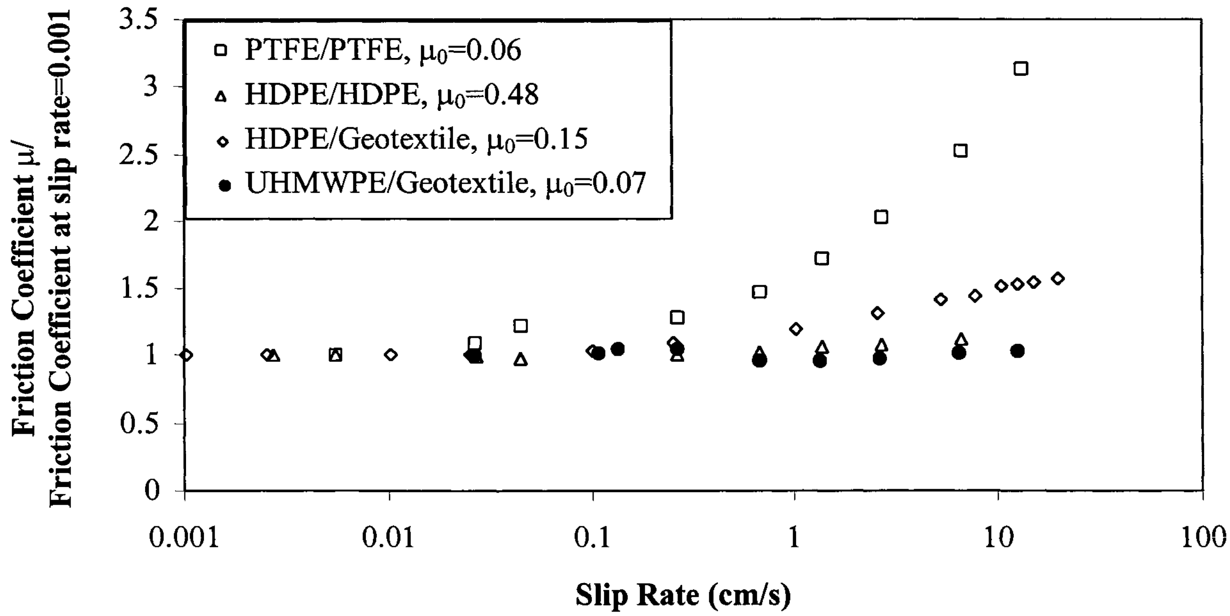

Figure 7. Influence of slip rates on friction coefficients normalized with friction coefficients (μ_0) measured at small slip rates (0.001-0.01 cm/s)

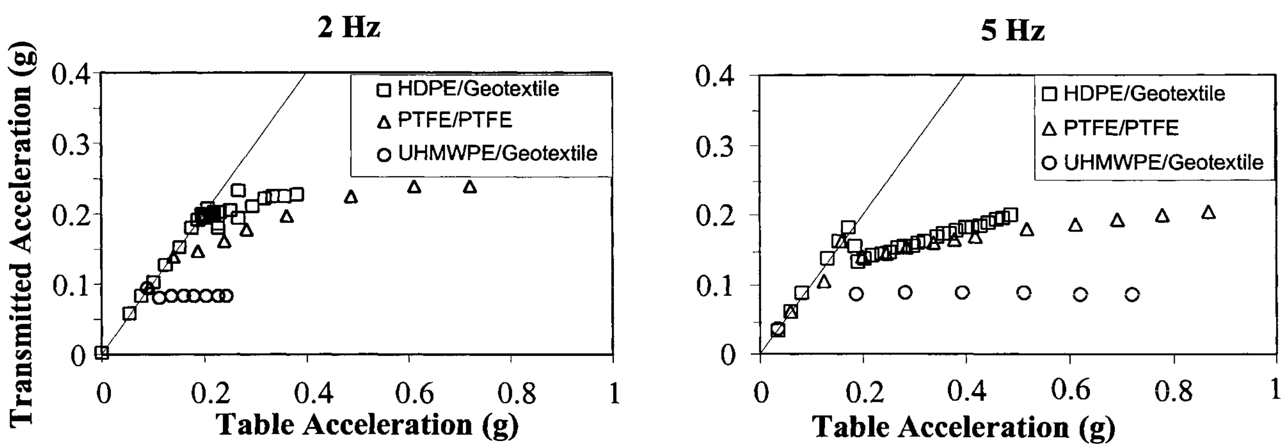

Figure 8. Accelerations transmitted through geosynthetic interfaces tested at 2 and 5 Hz sinusoiadal table excitations.

The UHMPE/geotextile interface was tested further using the shaking table. Figure 8 shows the accelerations transmitted through this interface, and are compared with those measured from other two interfaces tested. Again, the better suitability of the UHMWPE/geotextile interface is clearly observed. Sliding is initiated when the base acceleration exceeds 0.1g, a value associated with the peak shear resistance of the interface. Once slip occurs, the transmitted acceleration drops to about 0.07g. Such a low friction coefficient indicates excellent suitability of the UHMWPE/geotextile interface as foundation isolator even for small levels of ground shaking.

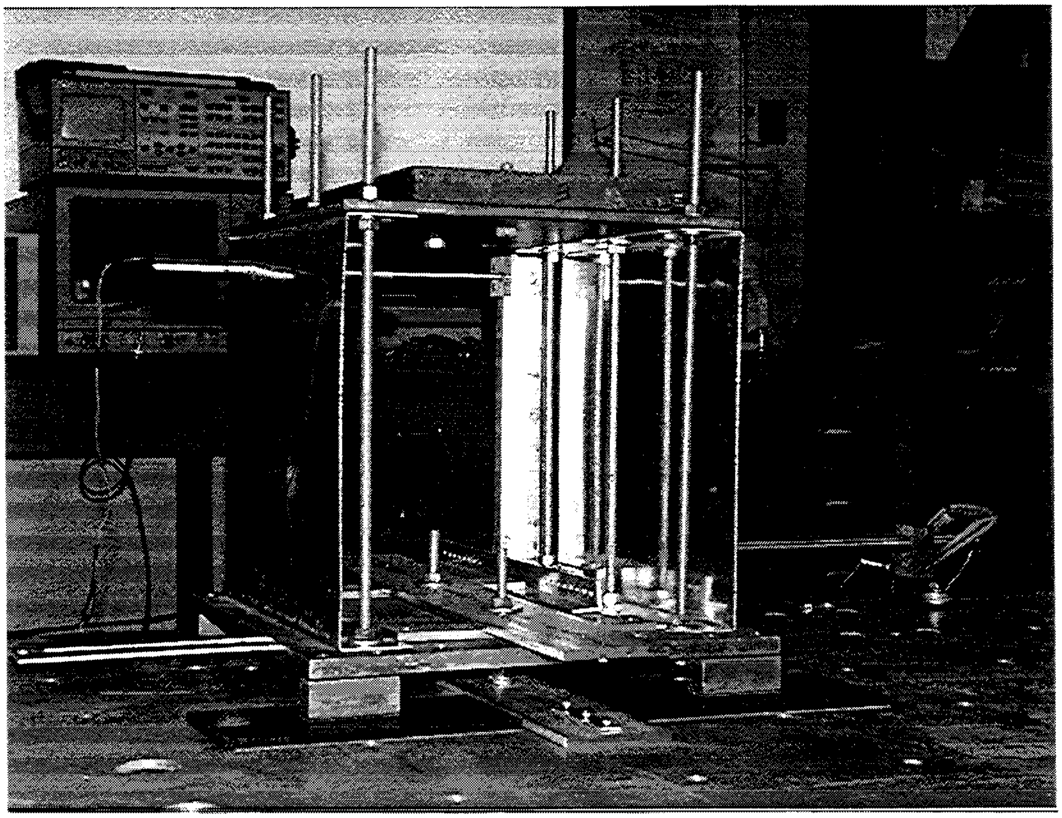

Figure 9. Photograph of the single-story building model tested on the shaking table.

MODEL BUILDING RESPONSE

The above geosynthetic interfaces were tested using harmonic base excitations. Also, the measured transmitted accelerations were those of a rigid block placed on the interface. These tests were useful to identify the geosynthetic interface that showed the best promise for application as foundation isolator. To evaluate the technical feasibility and benefits of using the selected geosynthetic interface as a foundation isolator for buildings, shaking table tests were carried out on a single story building model placed on the UHMWPE/geotextile interface.

Figure 9 shows a photograph of the building model that is resting on the shaking table. Figure 10 shows the measurement instruments used which included accelerometers to measure the building top floor and base accelerations, as well as the acceleration of the shaking table. Displacement transducers were used to measure the slip along the UHMWPE/geotextile interface, and to measure the distortion of the columns of the building model. Dynamic characteristics of the model were determined by free vibration tests. Its natural frequencies and critical damping value were measured as 8.6 Hz and 1% respectively. Two sets of test were performed. In the first set, the base of the building model was fixed on the table representing

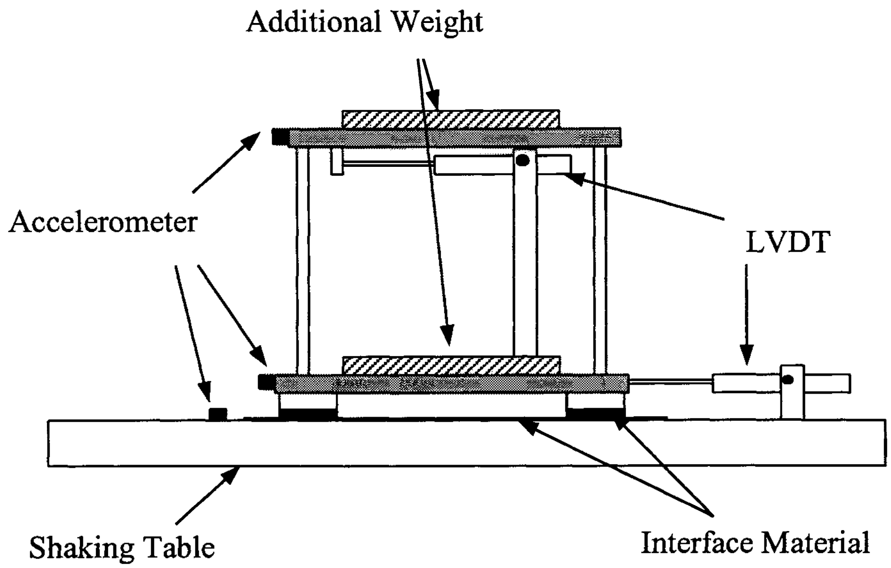

Figure 10. The experimental setup and measurement instruments used in testing the building model on the shaking table.

conventional design without foundation isolation. In the second set, the building model was placed on a geotextile, that was free to slide over the UHMWPE geomembrane.

Tests were run to understand the real dynamic interaction between the building top mass and its foundation, under earthquake excitations using three acceleration-time history records obtained from the 1989 Loma Prieta Earthquake. The records that were selected based on their frequency contents were: 1) Santa Cruz (with high frequency) 2) Capitola (with intermediate frequency), and 3) Corralitos (with low frequency). Different Tests were carried out by scaling the peak accelerations of the records, and by changing the mass ratios (top mass divided by the total mass) of the building model.

Figure 11 shows a comparison of the model responses with and without foundation isolation. The input table motion is the Santa Cruz record scaled to 0.35g. The results on the left side of Figure 11 show the building accelerations when the model was fixed to the table (without foundation isolation). It is observed that the dynamic response of the building model experiencing this earthquake record has amplified the base motion of 0.35g to a value of 0.77g at the roof level. The results of the shaking table tests on the model that was placed on UHWMPE/geotextile interface, as a foundation isolator, are presented on the right hand side of Figure 11. In this case, the peak acceleration at the roof level is only 0.33g, a reduction of 60% in comparison to the fixed based conditions.

As described earlier, the seismic energy transmitted to a building will lead to column distortions and shear forces. The column shear forces from tests with and without foundation isolation are compared in Figure 12 to further evaluate the benefit of using UHMWPE/geotextile liner. The vertical axis in the figure defines the ratio of the column shear force in the building

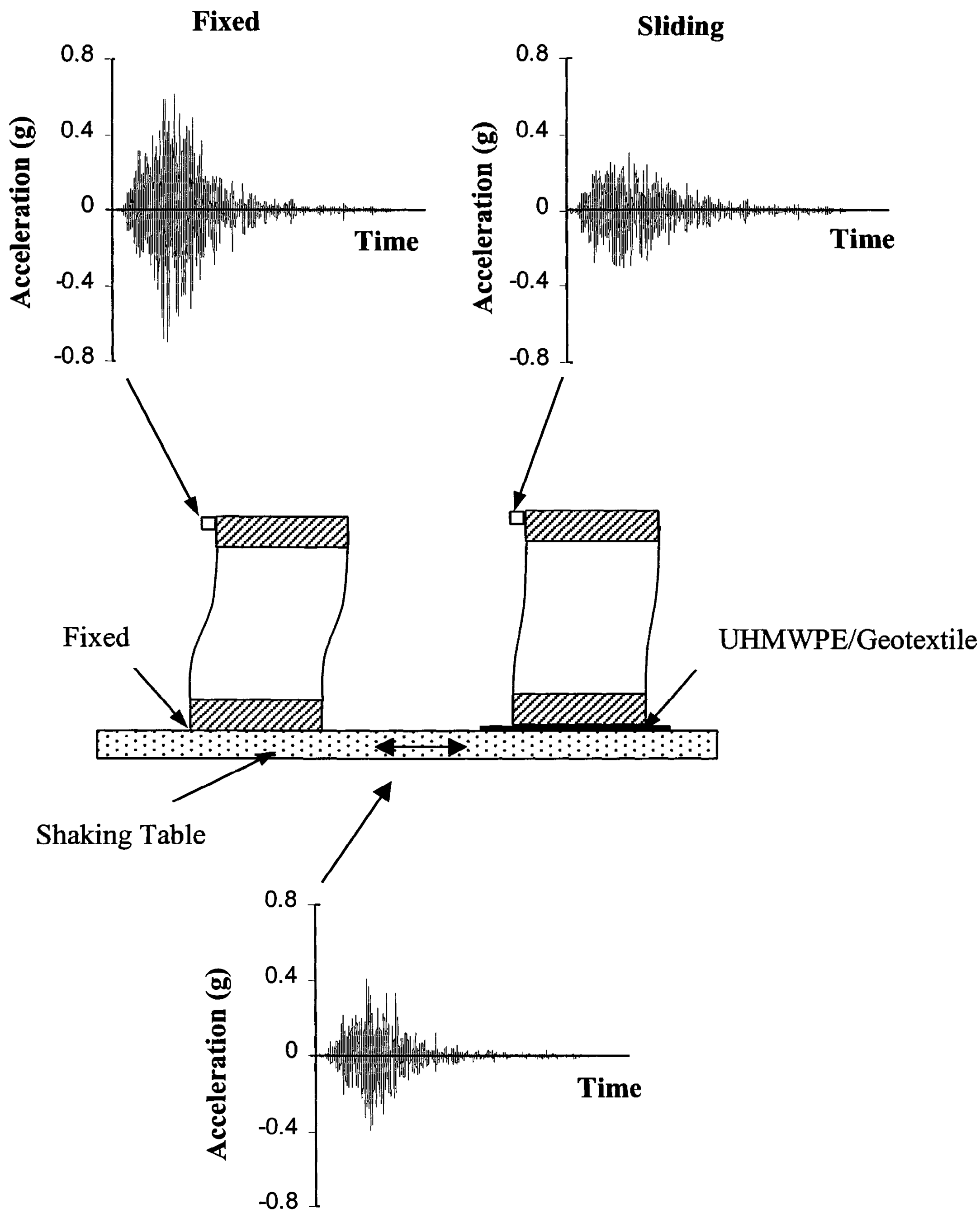

Figure 11. Comparison of model responses with and without geosynthetic foundation isolation.

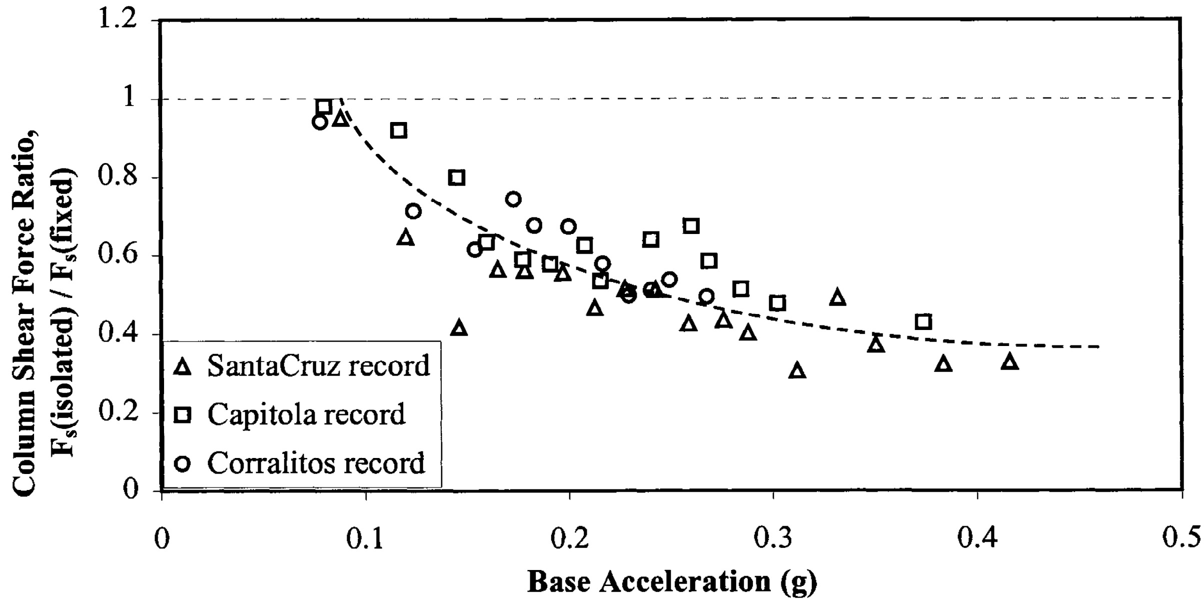

Figure 12. Reduction in column shear forces due to geosynthetic foundation isolation as a function of ground accelerations.

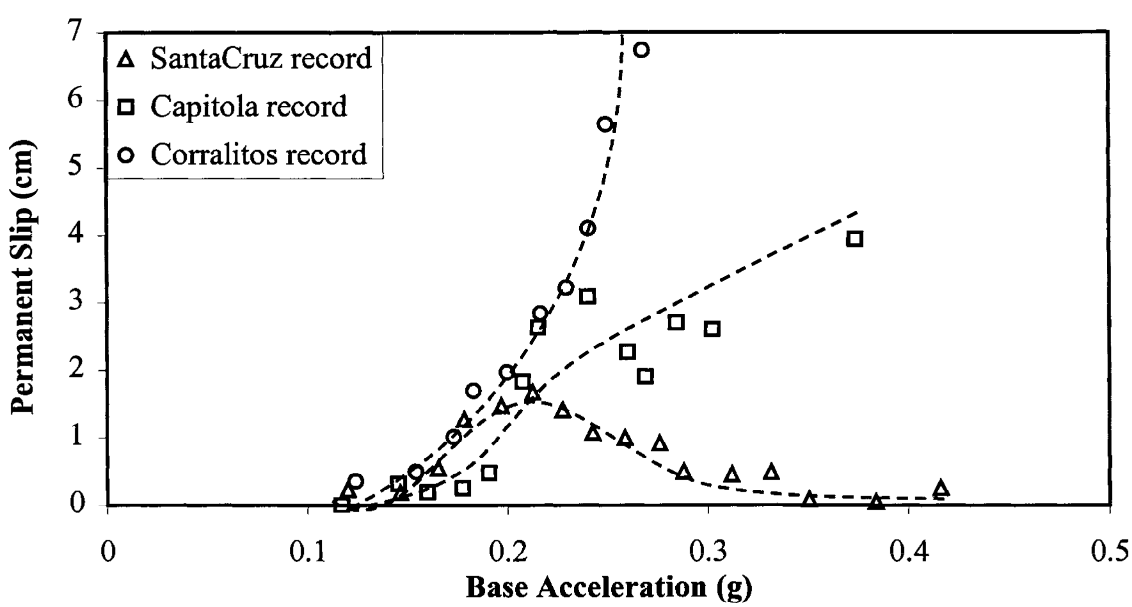

Figure 13. Slip deformations of the base as a function of ground accelerations.

model placed on the geosynthetic liner to the column shear force in the model that was fixed to the table. The horizontal axis defines the peak accelerations to which the three earthquake records were scaled. The results show that at a base acceleration greater than 0.07g the geosynthetic liner absorbs energy, and thus dramatically reduces the column shear forces in the building model. For example, at a base acceleration of 0.4g, the column shear force in the building model on foundation isolation is only 35% of that corresponding to the fixed case. This demonstrates the excellent energy absorption capacity of UHMWPE/geotextile interface.

Associated with this significant reduction in shear forces, as a result of foundation isolation, is the potential problem of slip deformations occurring along the geosynthetic interface. The measured slips from these shaking table tests are plotted in Figure 13. The results show that slip deformations typically are of the order of a few centimeters, and increase with increasing base accelerations.

At the present, the authors are continuing their research to evaluate the effect of various parameters that may influence the response of buildings on geosynthetic foundation isolators. Test results completed on UHMWPE/geotextile as foundation isolator have demonstrated a great potential for this interface to dramatically reduce the seismic loads on building structures.

CONCLUSIONS

Shaking table tests were carried out to demonstrate the effectiveness of using a smooth geosynthetic liner as foundation isolator that will reduce earthquake energy prior to being transmitted to a building structure. Various geosynthetic interfaces were investigated to identify a liner that is best suited for foundation isolation. Based on the experimental test results, Ultrahigh Molecular Weight Polyethylene (UHMWPE)/nonwoven geotextile interface was selected to be ideally suited for foundation isolation.

A model building structure was fabricated and tested on the shaking table to investigate the benefits of using UHMWPE/geotextile liner as foundation isolator. The column shears forces, the acceleration of the roof mass, and the slip along the liner interface were measured and analyzed under three earthquake excitations. The results show that through slip deformations the UHMWPE/geotextile liner reduces seismic energy, thus dramatically reducing the dynamic response of the building model. At a base acceleration of 0.40g, the column shear force in the building model on the UHMWPE/geotextile liner was 35% of that corresponding to a conventionally built, fixed base structure. Associated with this reduction in the column shear force was a permanent slip deformation measured to be about 4 cm for Capitola record.

In addition, using geosynthetics for foundation isolation to reduce seismic energy transmitted to buildings can be a very cost effective. It is also a simpler alternative to earthquake hazard mitigation measures conventionally used in current engineering practice.

ACKNOWLEDGMENTS

This research was initiated by a grant from the North American Geosynthetics Society, and the support of S.M. Bemben and D. A Schulze. This support is appreciated. The continuation of this research is being funded by a grant from the Earthquake Hazard Mitigation Program of the National Science Foundation. The authors express their appreciation to NSF and the program director Dr. Clifford Astill.

REFERENCES

Hussain, S.M. (1994) "Performance of base isolated buildings in the Northridge Earthquake", Proc. Seismic Base Isolation: State of the Practice, SEAOSC Seminar, CA.

Kavazanjian, E., Hushmand, B., and Martin, G. (1991) "Frictional base isolation using layered soil-geosynthetic liner system". Proceedings of the Third U.S. Conf. on Lifeline Earthquake Engineering, Los Angeles, CA, pp. 1140-1151.

Yegian, M.K., and Lahlaf, A.M. (1992) "Dynamic interface shear properties of geomembranes and geotextiles". Journal of the Geotechnical Engineering, ASCE, Vol. 118, No.5, pp. 760-779.

Zimmie, T.F., De, A., and Mahmud, M.B. (1994) "Centrifuge modeling to study dynamic friction at geosynthetic interfaces", Proc. Fifth International Conference on Geotextiles, Geomembranes and Related Products, Singapore, pp. 415-418.

GEOSYNTHETICS ENCASED STONE COLUMNS

J.N.MANDAL AND S.M.KAMBLE
DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, POWAI.
MUMBAI – 400076
INDIA

ABSTRACT

There are large tracks of soft marine clay deposits found along the major portion of the indian coast. Conventional type of shallow foundation could not be adopted in these areas because of high compressibility of these deposits resulting in excessive settlements and low bearing capacity. In the recent times, stone column technique has come into foundation practice to tackle such a difficult situations.Therefore an attempt has been made to carry out the experimental work of stone columns with and without geosynthetics,using centrifuge modelling technique. 40 successful experiments were conducted on Bombay marine clay for the various stone column configuration and various load intensities.It has been observed that the encased stone columns can sustain much larger bearing stresses without developing bearing capacity or excessive settlements and prevent shear failure. The load settlement and time settlement curves with and without geosynthetics are discussed and reported in the paper.

1 INTRODUCTION

A variety of methods of ground improvement techniques such as dynamic compaction, blasting, heating and freezing, consolidation preloading and vertical drains, electro-osmosis, lime-piles,jet grouting and granukar piles have been successfully applied in several cases.Granular piles also ,caleed stone columns,are becoming popular as technique of deep ground improvement not only in soft cohesive soils but also in loose cohesionless soils. These are composed of compacted sand or gravel inserted into the soft soil foundation by displacement method. The two essential requirements for the satisfactory performance of the foundation are that of foundation should be safe in shear and settlement should be within tolerable limits. A review of present state-of-the-art reveals that these approaches can easily be grouped in categories namely-[a] analytical approaches (Baumann and Bauer 1974, Pribe 1976,Balaam and Booker 1977, Aboshi 1979, Goughnar 1988, Rao and Ranjan 1985, Saha and De

1994);[b] empirical approches(Greenwood 1970, Thornburn 1975, Hughes and Withers 1974):[c] experimental approaches(Hughes et al. 1975).

2 EXPERIMENTAL DESIGN

A prototype fill marine clay of 5 m thickness has been modelled at a g-level of 100 as shown below,

- Height of protype(h_p) : 5 m
- Acceleration level (N) : 100
- Height of the model (h_m) : 5 cm
- Speed of centrifuge : 575 rpm

Experiments were conducted at various load intensities(0.3 kg/cm^2, 0.4 kg/cm^2, 0.6 kg/cm^2 and 0.8 kg/cm^2). To study the effect of peripheral geosynthetic reinforcenment on performance of stone column,the exoperiments were carried on both plain stone columns and reinforced stone columns.the stone columns were reinforced with geosynthetoics up to critical length 4D i.e. up to 4 times diameter of stone column.

3 EXPERIMENTAL SETUP

Model Container

A Perpex cylinder and Perpex base plate is used for fabrication of the model container
The details of the container are given below,

- Internal diameter : 125 mm
- Wall Thickness : 5 mm
- Base Plate : 150 mm × 150 mm
- Base Plate Thickness : 12 mm
- Clear height : 100 mm
- Cross-sectional area : 122.72 mm^2

To install the stone columns templates are fabricated using perpex sheet and stainless steel rods of external diameter 8 mm.The container and the templates are planned and fabricated indigenously as shown in Fig. 1 (a) to suit the requirments of the existing small geotechnical centrifuge installed in the soil engineering laboratory, Depeartment of civil Engineering,Indian Institute of Technology, Bombay. Configuration of stone column pattern is shown in Fig. 1 (b). The index properties of the soil are summarised in the Table. 1.

Five wide-width tensile strength test as per ASTM 4595-86 were performed on model geotextile, ultimate tensile strength of model geotextile is 0.85 N/mm at 2.7% elongation. The mass per unit area of model geotextile is 0.0015 gm/cm^2.

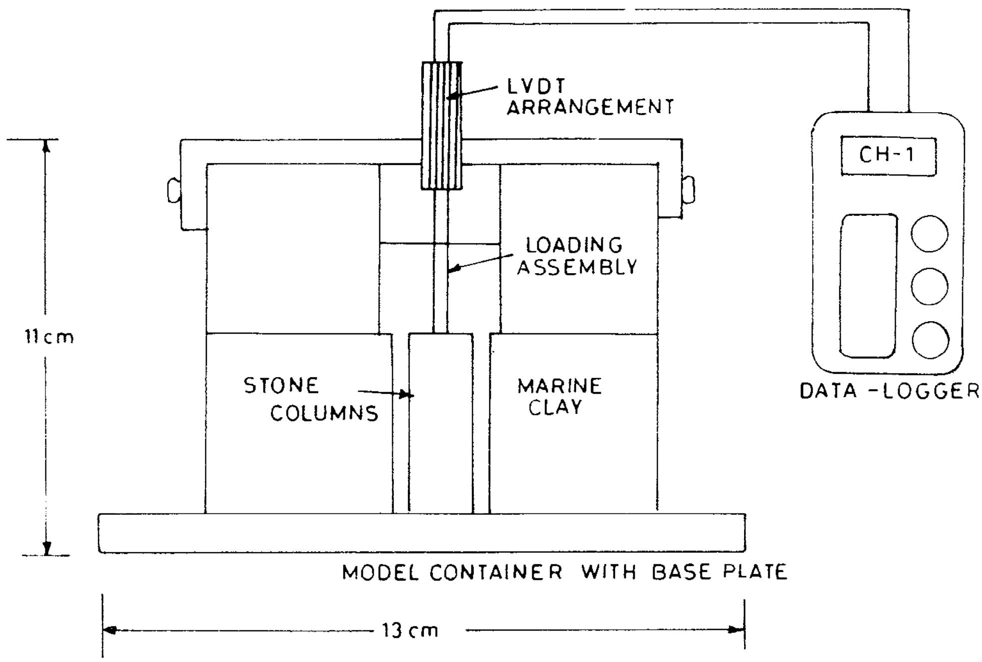

FIG. 1.(a) A VIEW OF THE MODEL CONTAINER WITH LOADING AND TRANSDUCER ASSEMBLY

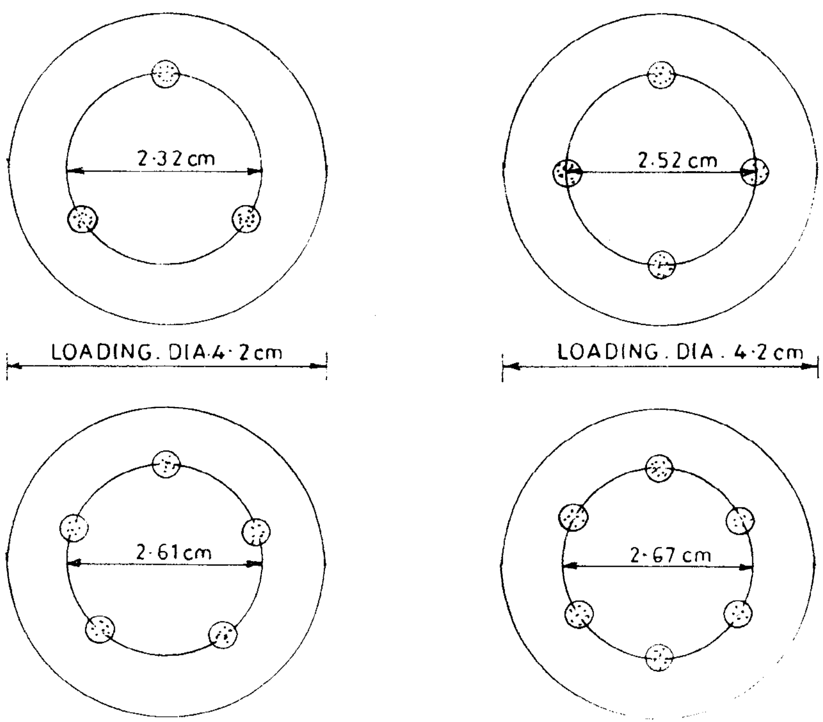

FIG. 1.(b) STONE COLUMN CONFIGURATIONS

Table 1. Properties of the soil used for experimental work.

Properties of marine clay used:	
Type of Clay	Bombay Marine Clay
Particle Size	Passing through 425 μ IS Sieve
Specific Gravity	2.74
Liquid Limit	79%
Plastic Limit	38%
Plasicity Index	41%
Flow Index	15%
Toughness Index	3
Coefficient of Consolidation	2.6×10^{-6} cm^2/sec
Submerged Unit Weight	0.46 t/m^3
Porperties of Stone Column Material:	
Type of Sand	Standard Sand
Specific Gravity	2.65
Properties of Sand Used as Surcharge:	
Type of Sand	Mumbra Sand
Specific Gravity	2.6
Submerged Unit Weight	0.855 t/m^2

Preparation of Model

Required quantity of soil passing through IS 425μ sieve,ovendried for 24 hours in a thermostat oven,then cooled in calcium dessicator is weigthted and the weight is recorded. Distilled water corresponding to appropriate initial water content is measured and then added to the soil. The contents of the soil and water are mixed thoroughly by using an electric stirrer,till a homogeneous soil paste is obtained. Then the slurry is placed in humidity chamber to prevent moisture loss.

The model container is cleaned and dried. A thin coating of silicon grease is applied on the boundaries, that are in contact with the deformation of the sample. Filter paper pieces are pasted in the inner walls of the perspex cylinder. A Saturated filter paper is placed on a thin sand bed at the bottom of perspex cylinder. This arrangement will faciliate the drainage at the top, botom and also radially, so the soil sample is normally consolidated(N.C.) at a quicker rate. A Satureted filter paper is palced on the top of slurry,beneath the sand layer and then the soil is normally consolidated by running the centrifuge for the required time. A template with required number of stone comlumns is gently pushed inside the N.C. clay sample. The clay in the steel molds is removed using a screw and template is gradulaly withdrawn. This makes the borehole in the N.C. clay. Non reinforced stone columns are prepared by filling the hole with oven dried standard sand passing through BS 18 and retained on BS 36, in 3-5 layers, each layer being tamped with the rod. For geosynthetic reinforced stone columns,the column is made in the borehole in usual way, leaving the top portion of the column to be restrained empty. The peripheral reinforcement is lowered upto 4D i.e. 4 times diameter of stone column and sand is compacted with rod. Transducer assembly is then fixed to the container such that the needle just touches the top of loading pad.

The details of small centrifuge in the soil engineering laboratory of Indian Institute of Technology are given in Table 2. The setup also includes the pocket LVDT which has a readibility of 0.01 mm. The data logger is confined to read the LVDT every 20 seconds.

Table 2. Details of the Centrifuge

Installation	Soil Engg. Labarotary,Dept of Civil Engg., Indian Institute of Technology Bombay
Type	Swinging Buckets On Both Sides Of The Arm
Arm Radius	20 cm
Maximum Outer Radius	31.5 cm
Centrifuge Range	250-1000 rpm
Maximum Acceleration	300g.
Capacity	0.72g tons.
Maximum Sample Depth	7 cm
Runup Time to 300g.	40 sec.
Rundown Time	2 min. 50 sec.
Motor	Universal (Capacity)
Equipments	Digistrob (3000 rpm)

4 RESULTS AND DISCUSSIONS

Observations made before the experiments consists of the time for which the centrifuge was run for each loading intensity for a particular configuration and loading specification area and intensity. The measurements made using the data logger during the experiments were pertaining to the settlements for each case. After 70 observations were made results were tabulated . For every case diameter of loading area, sample height and time for consolidation were identical.

The relative differences in the behavior of various sets of stone columns are mainly brought through load-settlements. A set of load settlement curves obtained from load intensity tests conducted on 8 mm diameter plain stone columns(PST) and reinforced stone columns (RST) using centrifuge modelling technique at 100 g-level. The stone columns are quite effective in reducing the vertical settlement beneath and adjacent to the loaded area, as compared to unreinforced ground.

Tables 3 and 4 give the settlement observations for 8 mm diameter plain stone columns(PST) and reinforced stone columns(RST) respectively. Table also shows that the virgin normally consolidated marine clay is failed at 0.6 and 0.8 kg/sq. cm load intensities when it is not treated. When the soil is reinforced with stone columns the results in Table 3 indicates that there is reduction in the settlement over a footing case on unreinforced soil, is about 20-25% for 3 stone columns, 30-35% for 4 stone columns, 35-40% for 5 stone columns and for 8 stone columns it is 50-55%. Thus using 6 PST in pile group we are getting almost half settlement compared to virgin soil. Now after examining Table 4 which indicates results for 6 mm RST, we can see the effect of geosynthetics encasing on the performance of stone column. There is reduction in the settlement about 40-45% for 3 stone columns, 50-55% for 4

Table 3. Settlement Observations for 8 mm Diameter Plain Stone Columns.

Stone Column Numbers	Loading Intensity (kg/cm^2)	Settlement Model(cm)	Settlement Prototype (cm)	Settlement Reduction Factor **
Zero Stone Column	0.30	4.22	42.2	-
	0.40	5.81	58.1	-
	0.60	*	*	-
	0.80	*	*	-
3 Stone Columns	0.30	3.02	30.2	0.71
	0.40	4.09	40.9	0.70
	0.60	9.59	95.9	-
	0.80	*	*	-
4 Stone Columns	0.30	2.49	24.9	0.59
	0.40	3.24	32.4	0.55
	0.60	8.46	84.6	-
	0.80	14.59	145.9	-
5 Stone Columns	0.30	2.24	22.4	0.53
	0.40	2.73	27.3	0.46
	0.60	7.22	72.2	-
	0.80	9.28	92.8	-
6 Stone Columns	0.30	1.91	19.1	0.45
	0.40	2.39	23.9	0.41
	0.60	4.47	44.7	-
	0.80	4.94	49.4	-

** Settlement reduction factor is the ratio of settlement of model with stone column to settlement of model without stone column.

stone columns, 60-65% for 5 stone columns and for 6 stone columns it is about 70-75%. These results show that when 6 mm diameter geosynthetics encased stone columns are used instead of plain stone columns, there is reduction of 30% for corresponding load intensity and number of stone columns in pile group.

Tables 5 and 6 show the settlement results for 6 mm diameter plain stone columns (PST) and reinforced stone columns (RST). After examining the data in Tables 3 and 4, We can conclude that the reduction in the settlement of treated marine clay over untreated is about 30 % for 3 stone columns, 40-45% for 4 stone columns, 50-55% for 5 stone columns and about 60% for 8 stone columns. Table 4 indicates the readings for 8 mm diameter geosynthetic encased stone columns in marine clay. This case

Table 4. Settlement Observations for 8 mm Diameter Reinforced Stone Columns

Stone Column Numbers	Loading Intensity (kg/cm^2)	Settlement Model(cm)	Settlement Prototype (cm)	Settlement Reduction Factor
Zero Stone Column	0.30	4.22	42.2	-
	0.40	5.81	58.1	-
	0.60	*	*	-
	0.80	*	*	-
3 Stone Columns	0.30	2.12	21.2	0.50
	0.40	2.83	28.3	0.49
	0.60	5.56	55.6	-
	0.80	8.64	86.4	-
4 Stone Columns	0.30	1.44	14.4	0.34
	0.40	2.33	23.3	0.40
	0.60	3.71	37.1	-
	0.80	5.39	53.9	-
5 Stone Columns	0.30	1.04	10.4	0.25
	0.40	1.42	14.2	0.26
	0.60	2.74	27.4	-
	0.80	3.94	39.4	-
6 Stone Columns	0.30	0.41	04.1	0.10
	0.40	0.72	07.2	0.11
	0.60	1.28	12.8	-
	0.80	1.62	16.2	-

* indicates failure of the foundation of system.

shows the lower settlement, for corresponding load intensity and stone column configurations with plain stone columns. It indicates that the improved performance i.e. decrease in the settlement over unreinforced soil is about 50-55% for 3 stone columns, 60-65% for 4 stone columns, 70-75% for 5 stone columns and about 90% for 6 stone columns. If we compare both Tables 3 and 4, we can observe that for same diameter stone columns i. e. 8 mm geosynthetic encased stone columns gives about 30% less settlement compared to plain stone column for corresponding load intensity and stone columns configuration.

Table 5. Settlement Observations for 6 mm Diameter Reinforced Stone Columns.

Stone Column Numbers	Loading Intensity (kg/cm^2)	Settlement Model(cm)	Settlement Prototype (cm)	Settlement Reduction Factor
Zero Stone Column	0.30	4.22	42.2	-
	0.40	5.81	58.1	-
	0.60	*	*	-
	0.80	*	*	-
3 Stone Columns	0.30	2.59	25.90	0.61
	0.40	3.32	333.20	0.57
	0.60	7.09	70.90	-
	0.80	11.69	116.90	-
4 Stone Columns	0.30	2.08	20.80	0.49
	0.40	2.73	27.330	0.46
	0.60	5.49	54.90	-
	0.80	8.77	87.80	-
5 Stone Columns	0.30	1.67	16.70	0.39
	0.40	2.11	21.10	0.36
	0.60	4.53	45.30	-
	0.80	5.91	59.10	-
6 Stone Columns	0.30	1.22	12.20	0.30
	0.40	1.76	17.60	0.28
	0.60	2.83	28.30	-
	0.80	3.51	35.10	-

* indicates failure of the foundation of system.

Further load settlement curves plotted in Figs. 2 and 3 show the effect of number of stone columns in pile group, effect of diameter of stone column and effect of geosynthetic encasement on the load bearing capacity and settlemnt reduction of the stone column-ground system. From load settlement curves, ultimate load carrying capacity of stone column can be estimated as load corresponding to a total settlement equal to 10 % of the diameter of the column. A set of these load settlement curves indicate that there is improvement in the performance of stone column system. Such systems increase load bearing capacity and reduce settlement where more number of stone columns in pile group used keeping all other conditions same. It is also observed that when 8 mm diameter stone columns used instead of 6 mm diameter there is 10 to 30 % reduction in settlement.

Table 6. Settlement Observations for 6 mm Diameter plain Stone Columns

Stone Column Numbers	Loading Intensity (kg/cm^2)	Settlement Model(cm)	Settlement Prototype (cm)	Settlement Reduction Factor
Zero Stone Column	0.30	4.22	42.2	-
	0.40	5.81	58.1	-
	0.60	*	*	-
	0.80	*	*	-
3 Stone Columns	0.30	3.29	32.9	0.78
	0.40	4.54	45.4	0.79
	0.60	11.08	110.8	-
	0.80	*	*	-
4 Stone Columns	0.30	2.93	29.3	0.69
	0.40	3.71	37.1	0.64
	0.60	9.15	91.5	-
	0.80	*	*	-
5 Stone Columns	0.30	2.55	25.5	0.60
	0.40	3.21	32.1	0.56
	0.60	7.45	74.5	-
	0.80	11.95	119.5	-
6 Stone Columns	0.30	2.29	22.9	0.54
	0.40	2.91	29.1	0.50
	0.60	5.89	58.9	-
	0.80	7.73	77.3	-

* indicates failure of the foundation of system.

The broad objective of this paper i.e. effect of geosynthetics encasing,(peripheral restrainment) of stone columns on the performance of stone-column-soil system can be clearly studied from load settlement curves in Figs. 2 and 3. It indicates that the further improvement in performance of stone columns can achieved by reinforced stone columns over plain stone columns.It is obvserved that mere provision of reinforced ground with reinforced stone columns may help in reducing the settlements between 50 to 90%.In present cases,the settlement reduction due to ground reinforced with reinforced stone columns is found to be increased by 30% over plain stone columns. This is due to tendancy for buldging of granular pile is restricted by the confinement effect of geosynthetic reinforcement as it mobilizes the tensile resistance.It is also observed that while constructin of stone columns significant saving in stone column material can be achieved using reinforced stone columns, the loss of costly stone column material while compaction of stone column in surrounding soil is restricted due to peripheral geosynthetic casing.

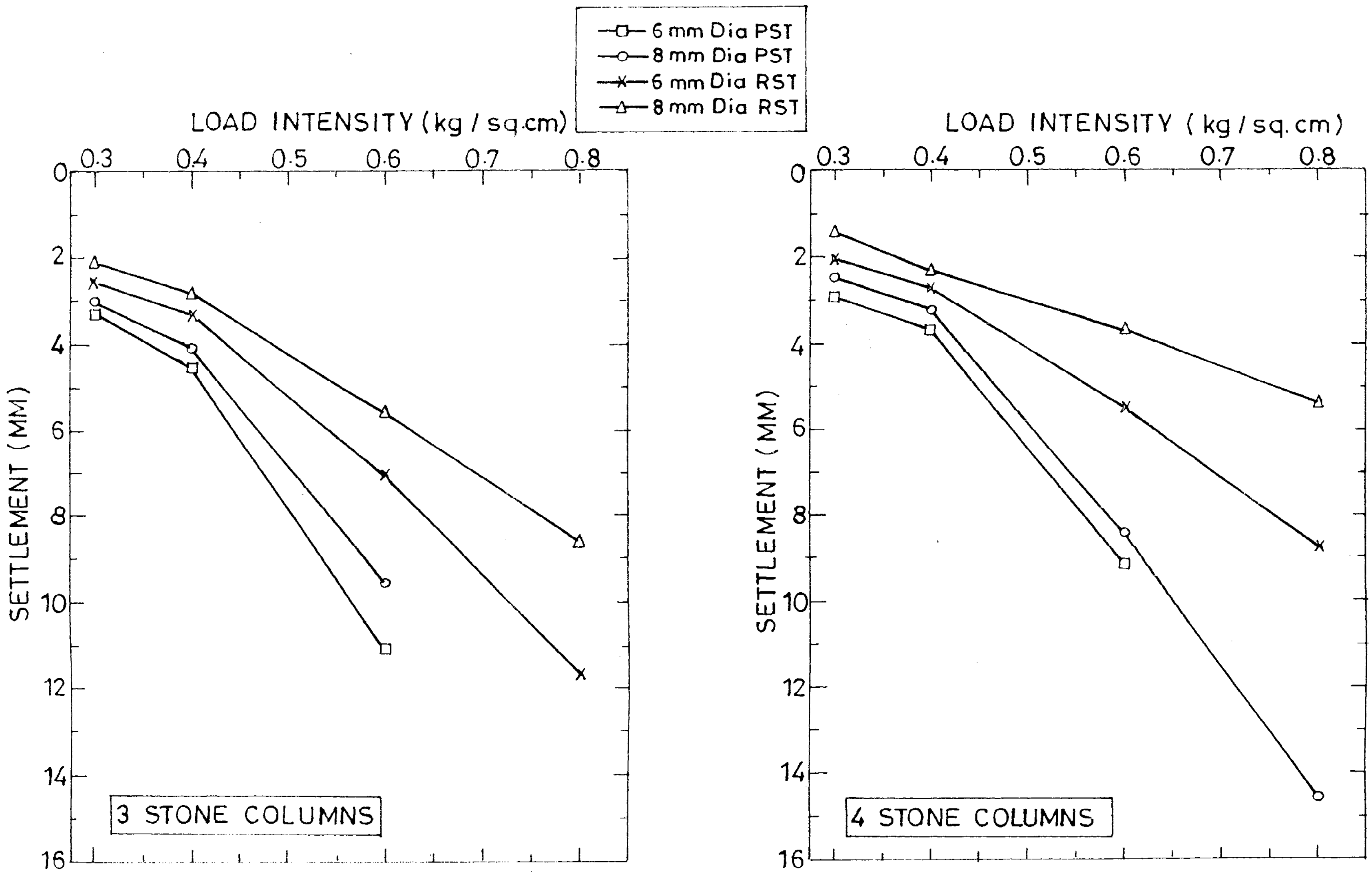

FIG. 2 Load settlement curves for plane stone columns (PST) & reinforced stone columns (RST)

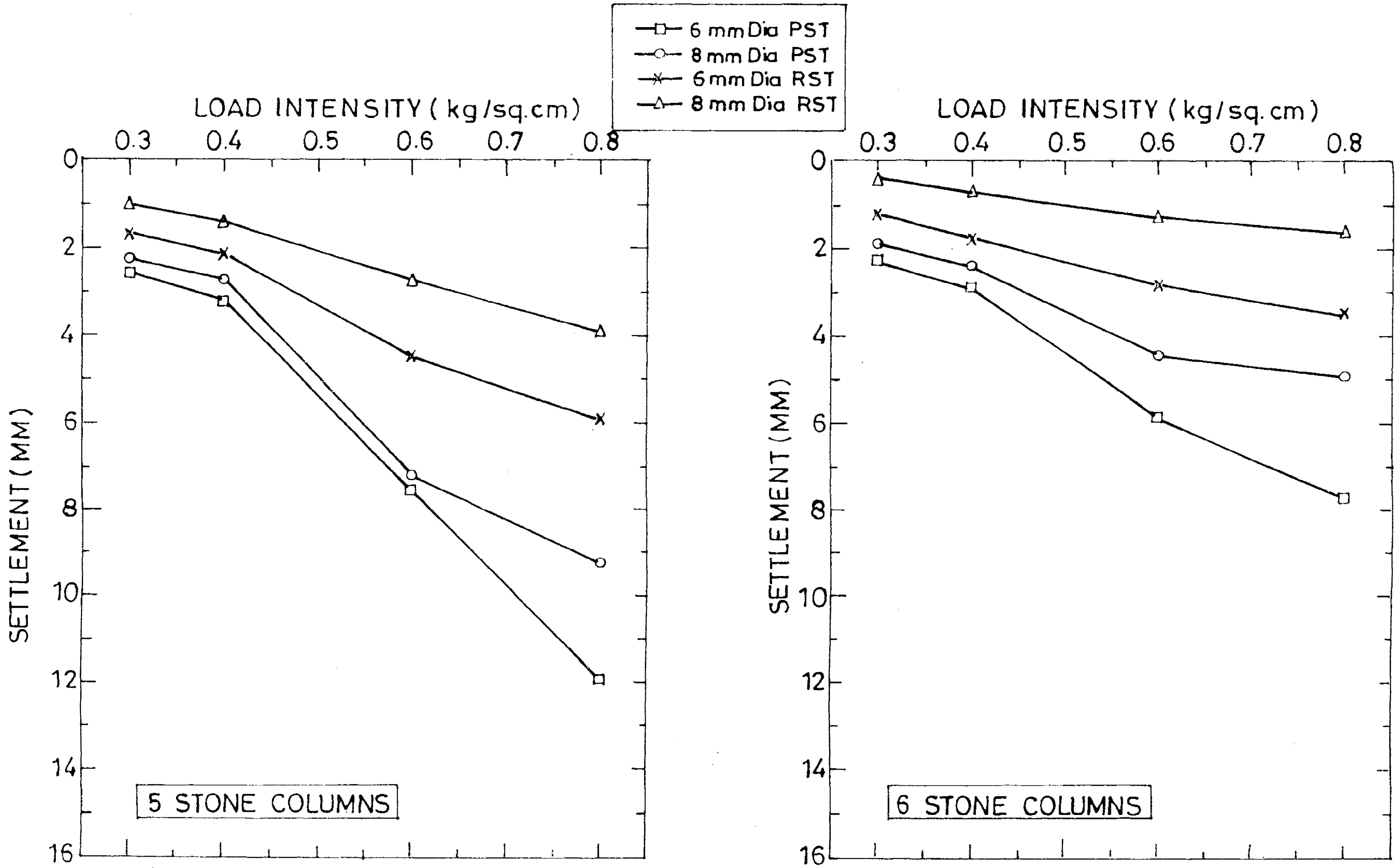

FIG.3. Load settlement curves for plane stone columns (PST) & reinforced stone columns (RST)

Post-test dessection indicated that the stone columns did not undergo shear failure,but purely buldging.The increase in shear resistance of soft clay layer due to presence of stone columns can be substantial depends on the magnitude of surface loading and stress concentration on stone columns.

5 CONCLUSIONS

The following conclusions were drawn based on the results obtained from centrifuge modelling experiments conducted on plain and geosynthetics reinforced stone columns:
The stone columns are quite effective in reducing vertical deformations beneath and adjacent to loaded area. The bearing capacity of soft marine clays can be increased and compressibility can be reduced significantly upto 80-90% by reinforcing it with stone columns. The increase in shearing resistance of soft clay layer due to presence of stone columns can be substantial, although depends upon magnitude of surface loading and stress concentration on stone column.

The number of stone columns and the diameter of stone columns in pile group affect the total performance of stone column-soil system. Greater number of stone columns in the pile group gives more load bearing capacity and reduction in the settlement. The performance of stone columns also can be increased by using large stone column diameter in pile group.

It is possible that plain granular piles occur buldging failure in the upper portion. So if buldging is restricted by providing geosynthetics reinforcement up to the critical length, where the vertical shear stress developed along the pile surface-clay interface is equal to average shear strength of clay, higher bearing capacity of stone columns can be obtained. The settlemnt reduction due to ground reinforced with geosynthetics encased stone column may be increased by 25 to 30 % over plain stone column. While construction of stone columns, due to compaction the loss of stone column material reduced due to peripheral reinforcement of stone column.

6 REFERENCES:

Aboshi, H., (1979) "The composer- a method of improvement characteristcs of soft clay by inclusion of large diameter sand columns", Collque. Inter. Sur Le Reinforcement De Sols E.N.P.C., Paris, France, pp. 211-216.

Ballam, N. P. And Booker, J. R.,(1977) "Settlement characteristic of soft clays reinforcement with granular piles", Proc.5th Asian Regional Conference, Vol.I, Bangkok.

Baumann, V. And Baur, G. E. A.,(1974) "The performance of foundation on various soils stabilised by vibrocompaction method", Canadian Geotechnical Journal,Vol.II,pp.509-530.

Greenwood,D. A.,(1970) "Mechanical improvement of soils below ground surface", Proc. Ground Engg.Conference, Inst.of Civil Engg., London.

Goughnar, R .R.,(1988) "Discussion on the paper-settlement analysis of skirted granular piles", Journal of Geotechnical Engg. Div. ASCE, Vol. 114, No.6, pp. 727-728.

Huges, J. M. O., and Withers, N. J., (1974) “Reinforcing of soft cohesive soils with stone columns”, Ground Engg, London, Vol.17,No.3,pp. 42-49.

Huges, J. M. O.,Withers, N. J. and Greenwood, D. A.(1975) “A field trial of the reinforced effect of stone column in soil”, Geotechnique. London, Vol.25, No.1, pp.31-44.

Priebe, A.,(1976) “Abschalzung des setzung sverha mens eines duren stopverditchtung verbesserten”, Baugrundes die Bautechnik.H.5.1946.

Rao, B. G. and Ranjan, G.,(1985) “Settlement analysis of skirted granular piles”, Journal of Geotechnical Engineering, Vol.111.No.11, pp.1264-1283.

Saha, A. and De, P. K., (1994) “Field and analytical study of stone columns by non-linear strain”, XIII ICMSMFE, New Delhi, pp. 1211-1214.

Thorburn, S.,(1975) “Building structures supported by stabilized ground”,Geotechnique,London, Vol.25, pp.83-94.

MISAPPLICATION OF A GEOGRID RESULTING IN FAILURE OF A CULVERT

RAYMOND D. D'HOLLANDER, P.E.
BLASLAND, BOUCK & LEE, INC., U.S.A.

ABSTRACT

A water control structure, consisting of a pair of corrugated metal pipe culverts, was constructed over soft, marshy soils. The structure included burial of the pipes through two berms and a pipe bridge over a canal. Part of the structure was constructed over a geogrid reinforced granular base. Settlement of the underlying soft soils caused abrupt differential settlements at the pipe joints. The geogrid was ineffective at preventing the total or differential settlements and the joint bands ruptured. The ruptured joints caused a serviceability failure of the water control structure due to massive leakage, requiring complete reconstruction.

DESIGN

A water control structure, consisting of a pair of 46-m long, 1.5-m diameter corrugated metal pipe (CMP) culverts with underlying geogrid support, was constructed in November and December, 1991 in a wildlife refuge. The purpose of the water control structure is to aid in maintaining consistent water levels in an area by providing a means of drainage that can be opened during periods of flooding and closed when water storage is desired. The water control structure that is the subject of this paper is part of a series of 1.2-m and 1.5-m water control structures at the refuge. The following discussion of the water control structure design is based upon review of available design plans, specifications, boring logs, and post-construction correspondence between the owner and contractor.

The design locations of the intake and discharge points of the water control structure required the culverts to cross a canal and two berms. A plan and profile of the water control structure is shown in Figure 1. The water control structure was designed and constructed as a pair of essentially identical parallel culverts.

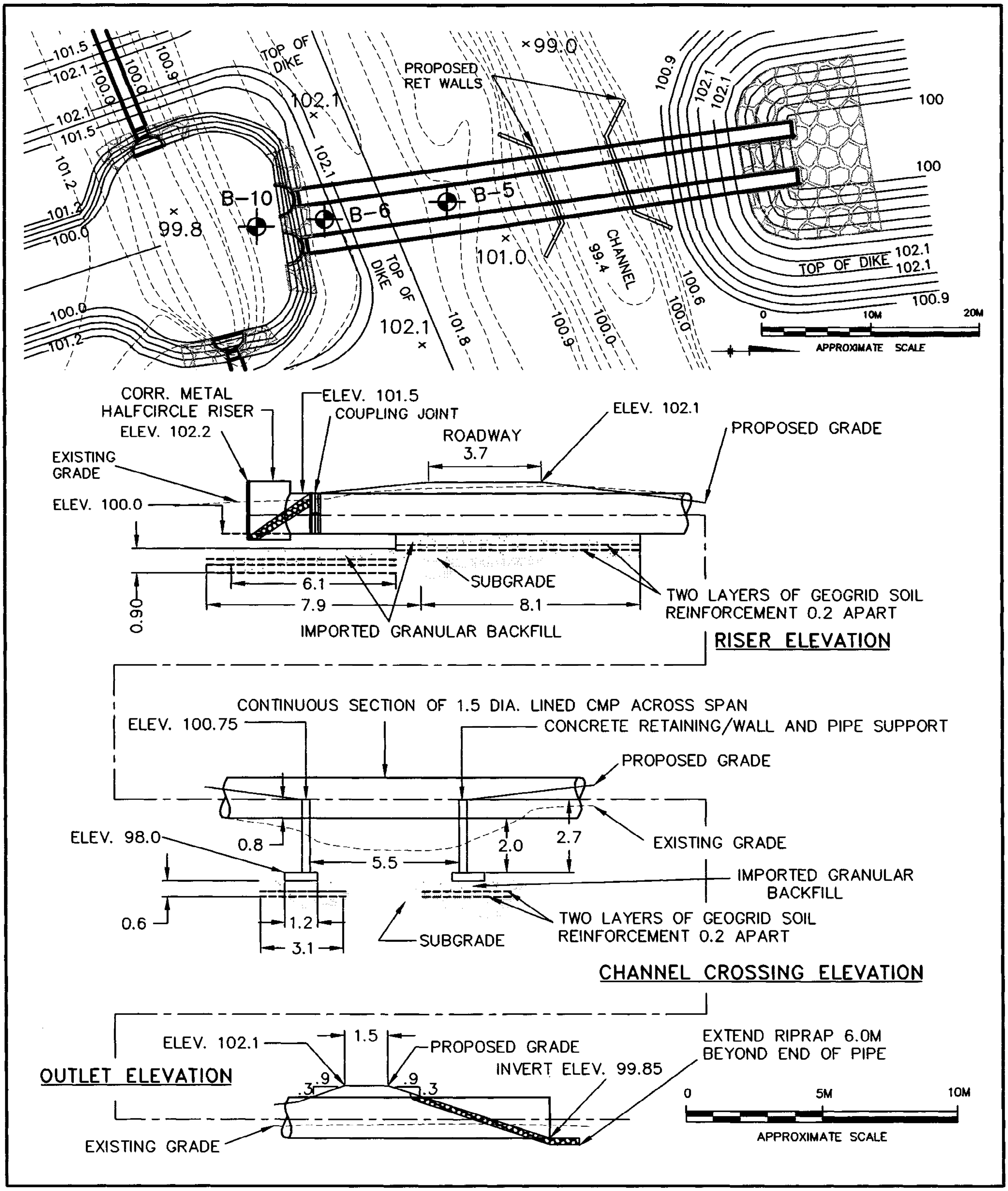

Figure 1. Water Control Structure Plan and Profile

Each culvert begins at the south end with a half-circle riser intake structure that can have boards inserted into channels to control the entry elevation of water into the pipes. The intake structure is directly connected to a short (approximately 1 m) long pipe section. The 46-m long culvert connected to this riser structure comprised a series of 3-m to 9-m long CMP sections. The culvert pipes were AASHTO M36, Type IA 60 inch (1.5-m) diameter helically made steel-lined smooth-bore 14-gauge corrugated metal pipe with rerolled ends. The 16-gauge connecting bands were designed to be at least 0.33 m wide with a continuous corrugation and neoprene O-ring gasket on each side of the band (Figure 2).

The design intake invert of the culverts was elevation 100.0 m. Immediately north of the intake riser structure, the culverts penetrate through a berm with a design crest elevation of 102.1 m with a 3.7-m wide roadway forming a broad flat top to the berm. The culverts through this berm are made up of two 6-m long sections, followed by a 3-m section and a subsequent 6-m section. At the north side of the berm, the culverts become a pipe bridge spanning a small canal. Each of the paired culverts uses a 9-m long CMP section supported on concrete retaining walls on both sides of the canal to form this pipe bridge. The actual free span length over the canal is approximately

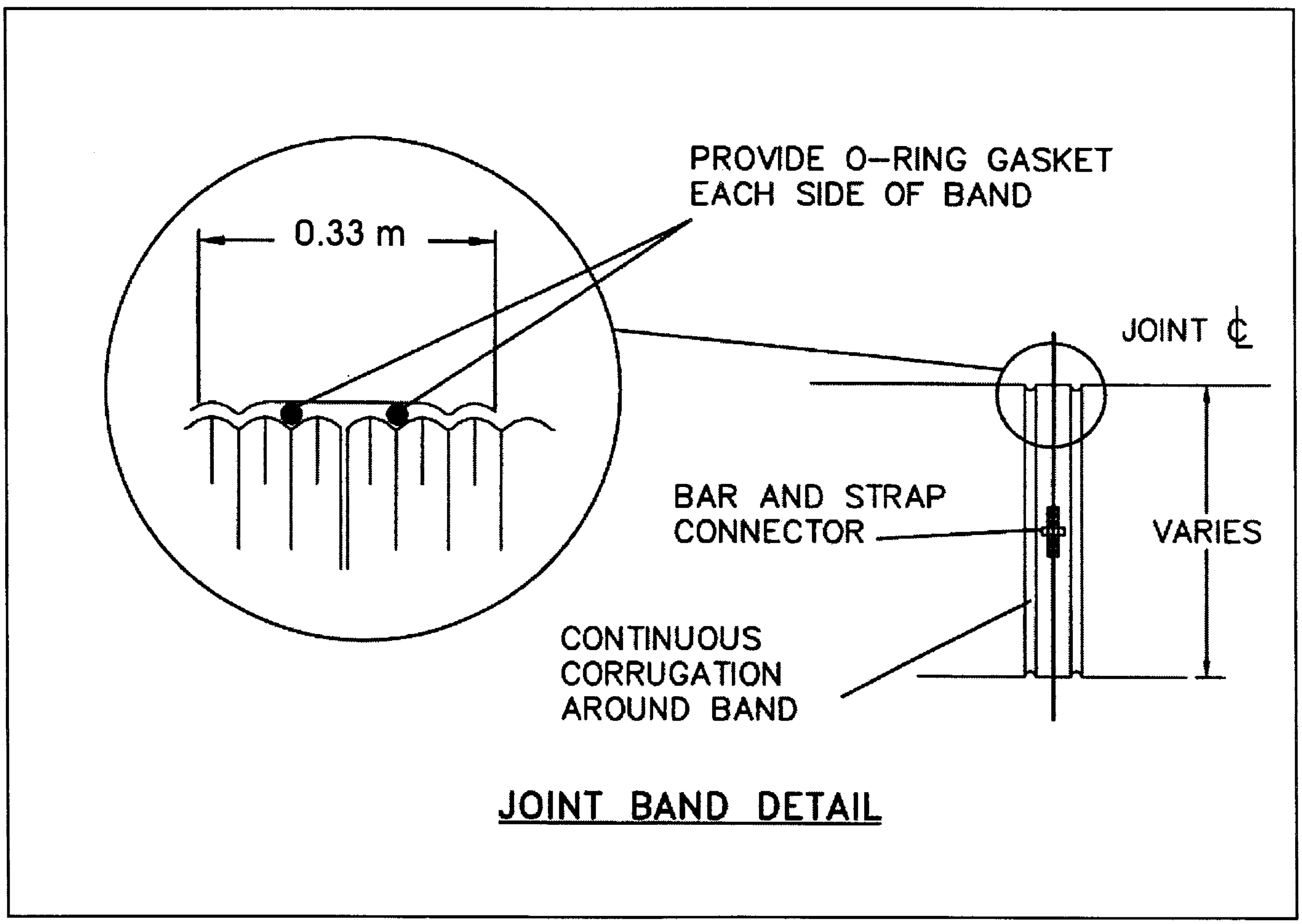

Figure 2. Joint Band Detail

5.5 m. After the pipe bridge, the culverts penetrate through another berm, also with a design crest elevation of 102.1 m, prior to reaching the outlet point. This berm was not designed as a roadway and therefore the crest was 1.5-m wide. Each of the culverts was constructed with a pair of 6-m long sections immediately north of the pipe bridge followed by a 3-m long section that formed the outlet. The design outlet invert was elevation 99.85 m.

The available information indicates that three soil borings were advanced at the water control structure location. The approximate locations of these borings, based on available information, are shown on Figure 1. The boring log information is summarized on Figure 3, below. The depth of the borings ranged from 3 to 6 m. Once the embankment materials were penetrated, the boring logs indicated the presence of very soft peats, silts, and marls for the remainder of the boring. The peats are typically at least 1.5 m thick. Standard Penetration Test (SPT) N-values were less than 10 for all samples and some samples could be pushed under the weight of the hammer.

A geogrid-reinforced granular base course was provided for the culvert support and pipe bridge footings, apparently to mitigate the influence of settlements of these soft and organic soils. The design documents called for a 6-m by 9-m, 0.5 m thick reinforced concrete slab below the riser intake structure and 3 m of culvert after the first banded joint. The next 8 m of pipe was designed with a double layer of geogrid embedded in a coarse granular bedding. The footings for the retaining wall/pipe support structures were constructed over double layers of geogrid in a coarse

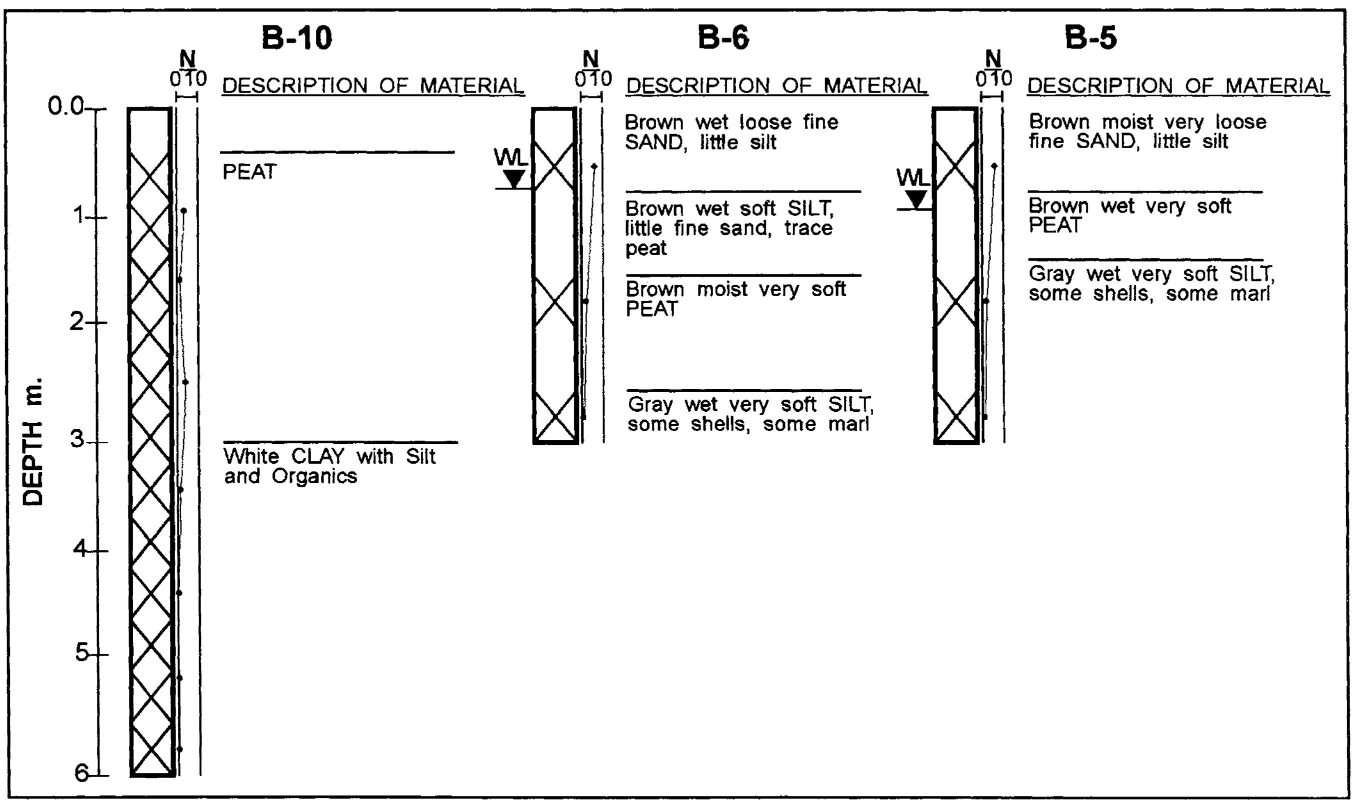

Figure 3. Boring Log

granular bedding. The footings for the retaining walls were designed to be 1.2 m wide, 0.3 m thick, constructed of reinforced concrete, and to have a bearing elevation of 98.3 m. The 2.7 m high walls were 7.6 m long with 3.7 m long wingwalls oriented back at 45 degrees. The culverts were designed to be supported on saddles cast into the concrete. The pipes penetrate through the wall at a skew of 10 to 15 degrees. The remainder of the pipe sections were designed to be supported over unreinforced granular bedding. Figure 4 shows the pipe bridge and retaining walls.

Figure 4. Pipe Bridge and Retaining Walls

CONSTRUCTION DESCRIPTION

The water control structure was constructed in November and December, 1991. The following discussion of the actual construction is based on the owner's inspector's daily logs, photocopies of the inspector's photographs, post-construction correspondence between the owner and contractor, post-construction survey, and a post-construction site visit by the author.

The construction of this water control structure was performed concurrently with other water control structures at the refuge. The other structures were less complex and of shorter length. They penetrated through single berms and did not involve retaining walls with pipe bridges.

The records indicate that the construction of the water control structure proceeded relatively uneventfully with three exceptions:

- The stone bedding in the north berm settled and a new layer of stone with an additional layer of geogrid was placed prior to setting the culverts.

- One of the pipe saddles in the retaining walls did not line up properly and had to be cut out to allow the pipe to be placed in a straight line. Grout was used to provide a new seat for the pipe once it was aligned.

- The owner had the general contractor raise the dike elevation to 102.4 m to accommodate 0.3 m of anticipated settlement.

The daily logs indicate that the inspector was checking numerous construction details, including as-built pipe invert elevations, tightness of banded pipe joints, rip-rap thickness, concrete placement, and pipe bedding conditions. It is of interest that the only noted problems with pipe joints in the 1991 daily logs refer to loose bands at another water control structure being built concurrently. The water control structure pipe invert elevations were also specifically checked and found to match the design elevations.

POST-CONSTRUCTION PROBLEMS

Leakage from behind the wingwalls was noted in the spring of 1992 during the first use of the culverts. Sandbags were used in an attempt to staunch the flow of water. It was found that the leakage had washed out much of the soil immediately behind the wingwalls. The contractor constructed a temporary cofferdam to retain water in the main pond north of the water control structure.

In early June 1992, water levels were controlled so that the owner and contractor could conduct an inspection of the water control structure. This inspection showed that the joints at each end of both the 9-m long pipe bridge culverts had separated to the point that daylight could be seen through the joints due to the amount of soil that washed away, especially behind the south wall. A tunnel scoured along the west pipe up to 4.6 m behind the south wall was reported. Figure 5 shows the joint deformation observed at the pipe bridge after the backfill was removed.

Several other joints were noted to have separated as well. Some of the connecting bands and O-ring gaskets were noted to be out of position as the band corrugation that should have been located at the second corrugation from the end of the pipe were instead located over the joint or the first corrugation. O-ring gaskets were noted to be in a variety of positions, ranging from hanging down through the joints to remaining in the second corrugation outside of the band which had slipped off. Variations in the space between pipes were noted. Generally the space was noted to be greater at the bottom of the joint than at the top except at the joints to the pipe bridge. Figure 6 shows the typical deformation pattern observed inside the culvert and Figure 7 shows the typical separation between the pipe and band.

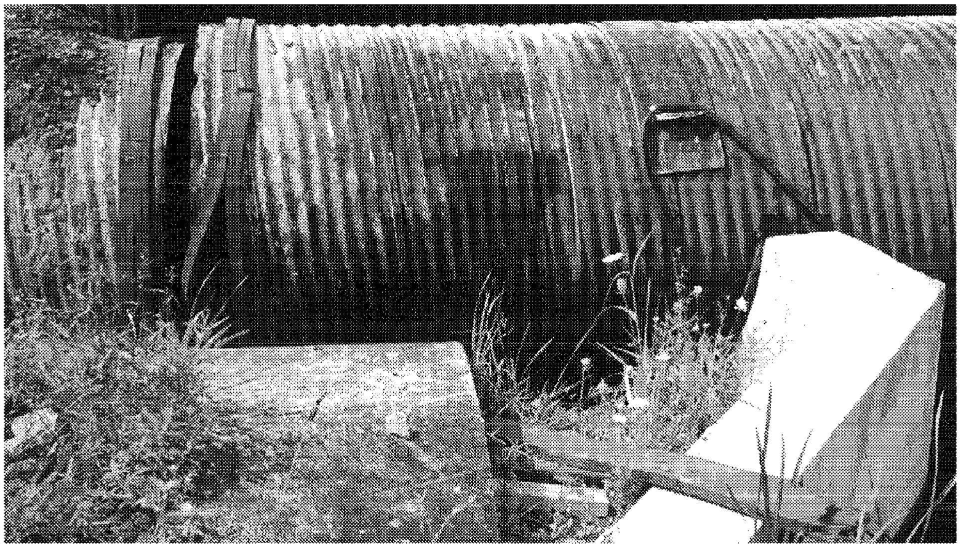

Figure 5. Typical Deformation of Pipe Bridge Joint

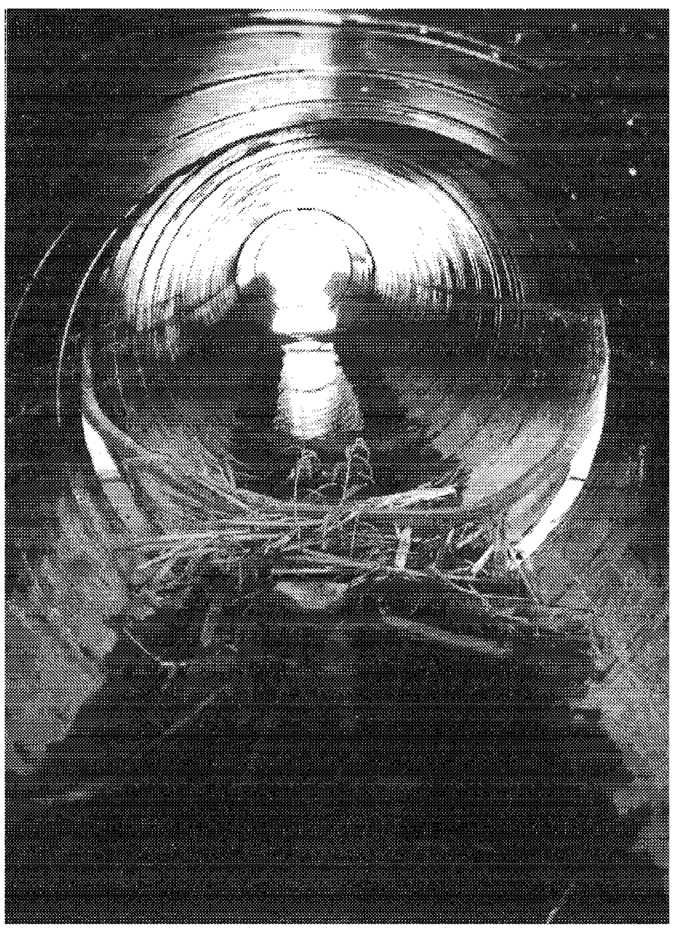

Figure 6. Typical Deformation of Pipe Joints

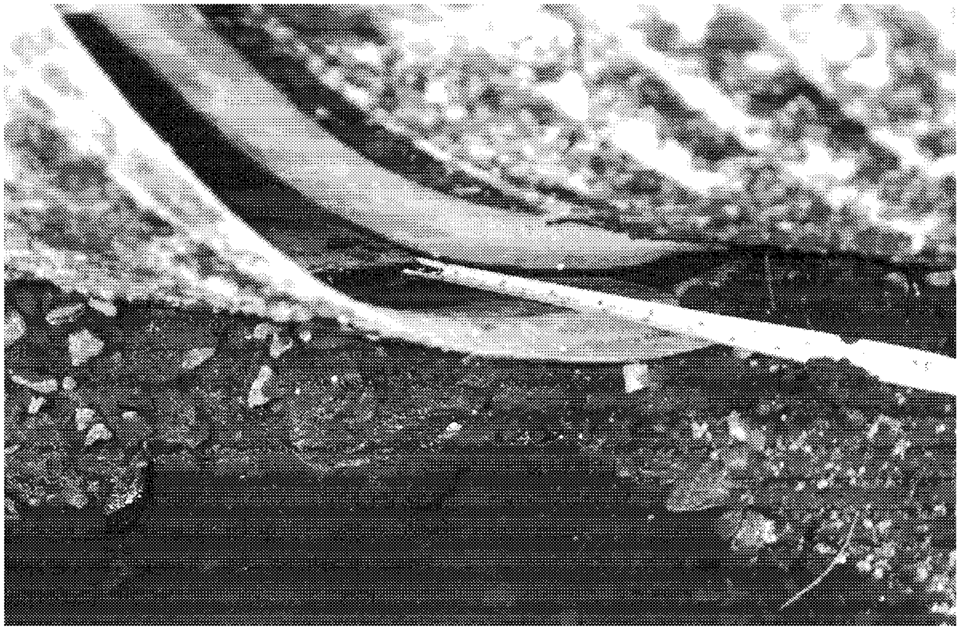

Figure 7. Typical Separation Between Pipe and Band

Other types of distress noted included: indentations, 0.2 to 0.3 m in diameter, that reflected through both layers of the steel pipe; and slightly out of round sections with the ends of the pipe bridge culverts being the most prominent. The author understands that the first few sections in the main berm south of the pipe bridge were set and backfilled first to allow trucks to haul fill across them which may have stressed the pipes.

This water control structure had failed to the point of being unusable for its design purpose and required near-total reconstruction. The lack of serviceability was caused primarily by the leakage from the joints immediately behind the retaining walls eroding away soils and causing large losses of water from the water control system. Some problems with the other water control structures were also noted, including loose bands and outlet pipes apparently banged out of alignment, but these defects did not appear to have significantly impacted the serviceability of these structures.

At this point, the contractor uncovered the pipes north of the pipe bridge and attempted to put new bands around some of the joints. These bands were 0.60 m wide and smooth, without corrugations to lock into the pipe. Apparently these bands were too loose and did not fit around the pipe. The vertical and horizontal pipe diameters were measured to be 1.46 to 1.49 m. Based on this information, the conclusion was drawn by the owner and contractor that the failure was caused by loose bands due to the culvert pipes diameters being less than specified.

The author became involved as an expert on behalf of the pipe supplier and visited the site at the end of August, 1992. The pipes were largely exposed but apparently otherwise undisturbed, with the exception of the west pipe bridge which had been removed from its bed and was resting on the top of the retaining wall.

A survey team accompanied the author on this site visit. The pipe invert and top of retaining wall elevations were surveyed to compare with the design and surveyed end-of-construction elevations. As discussed above, the construction records indicate that the design and as-built elevations were the same. The survey data indicate that the pipe inlet and outlet settled 70 to 110 mm during the eight months following construction. The survey data also showed that the north retaining wall also settled 90 mm. The elevation of the top of the south retaining wall was not surveyed due to time and access difficulties.

During this visit, water was flowing very slowly along the bottom of the pipes. The author noticed that the water depth varied significantly as it flowed through the pipe, and that the water depths could be used to measure the profile along the pipe invert. The design pipe invert profile and the pipe invert profiles measured 8 months after construction are compared on Figure 8. The survey data and water depth data indicate that the pipe inverts, when compared at the north and south ends, approximately maintained the 150 mm elevation drop from south to north as designed even though the actual pipe ends have settled. The wall settlement and ends of the exposed west pipe where the pipe bridge had been removed indicate that this design gradient was also maintained at the pipe bridge location. However, assuming that the pipe had been laid with a smooth invert, the water depths along the pipes indicate that significant (120 to 270 mm) differential settlements occurred in the middle of the berms. Abrupt water depth changes as great as 100 mm were measured across pipe joints indicating that pipes had moved vertically with respect to each other.

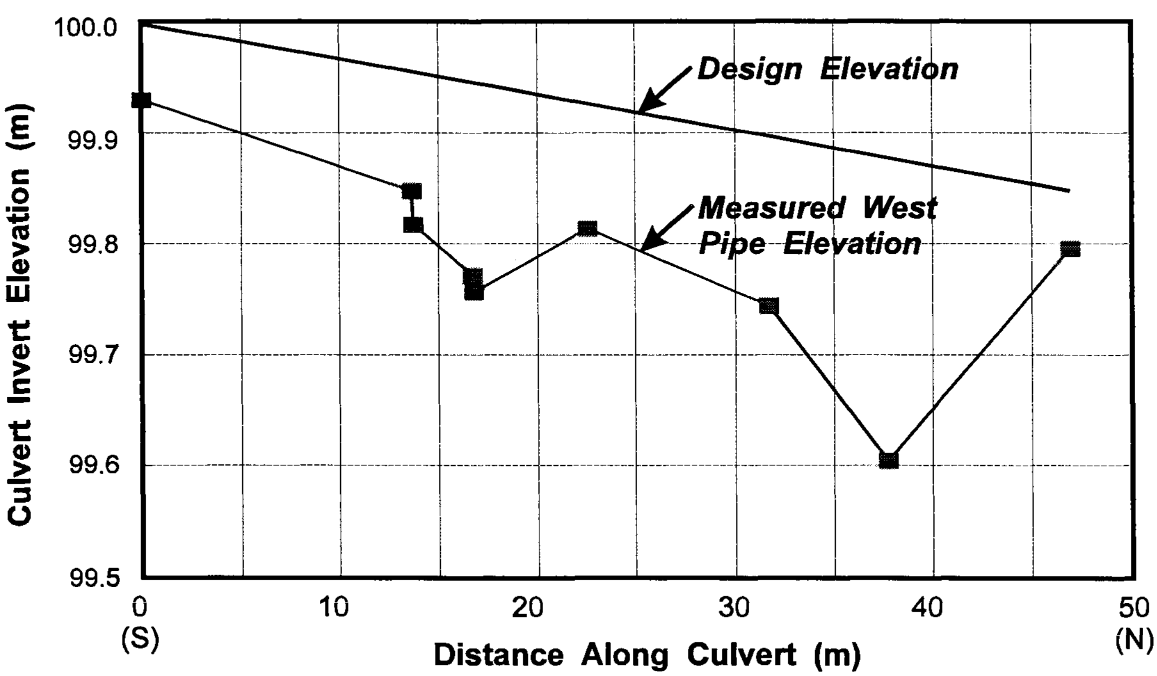

Figure 8. Design and Actual Invert Elevations

The author noted that the joints had typically deformed with a wider space at the bottom than at the top which was the same pattern noted by the owner's inspector. At some joints, it appeared that the top of one pipe was pushed into the other pipe while the bottoms separated, creating a hinge effect. This deformation pattern is consistent with the general settlement pattern that was clearly convex with greater settlement towards the middle of the berm instead of at the footings.

ANALYSIS AND CONCLUSIONS

Three basic failure modes were deemed possible. These modes were:

- Seepage through loose bands causing erosion and pipe settlement;
- Joint separation due to thermal contraction of the joints; and
- Differential settlement of the pipes causing rupture of the joints.

Loose Bands

The owner had concluded that loose bands due to smaller than specified pipe diameters caused seepage from the joints which then eroded the soil out from behind the retaining walls. The author reviewed the available construction inspection information to evaluate the likelihood that identical defects would have occurred unnoticed on the pipe bridge and several other joints.

The owner's inspector was onsite through nearly all of the construction. The author believes that the inspector was looking for loose bands because one of the inspector's daily logs noted that he observed loose bands observed on another water control structure being constructed concurrently and directed the contractor to remedy the joints. No such notes appear to have been made regarding the subject water control structure. Photocopies of construction photographs were made available to the author. These photographs clearly show that the inspector saw the completed pipes and joints prior to burial. The bands visible in the photographs appeared to be sufficiently tight for the neoprene O-ring gaskets to contact both the pipe and band which should have limited seepage to nominal amounts.

In addition to the available evidence indicating that the bands were installed properly during construction, there were difficulties in ascribing all of the observed damage to the loose band theory. While loose bands could have been a possible explanation for damage in the vicinity of the retaining wall where the soil was washed away, it is unclear how loose bands with some seepage could have caused the significant and consistent joint ruptures observed in the middle of the berms where no erosion was observed.

Thermal Contraction

The concept of thermal contraction was discussed between the pipe manufacturer, contractor, and owner after failure. The pipe manufacturer suggested replacing the 0.33 m bands with 0.6 m bands if thermal contraction was a concern or determined to be the cause of the failure. However, the author does not believe that thermal contraction would have been a primary cause of the joint failure for the following reasons.

The pipes were placed and connected in late November and December with typical temperatures ranging between -7 and 5 degrees Celsius as recorded by the inspector in his daily logs. Since problems were noted in early spring, it is unlikely that the installed pipe saw atmospheric temperature differences greater than 30 degrees Celsius between the end of construction and failure. The maximum atmospheric temperature differences at this location are typically 55 to 65 degrees Celsius between winter and summer. Since similar designs have performed adequately at this and nearby locations over extended time periods, it is unlikely that the relatively small actual temperature differences would have significant effects on the culvert performance. Temperature differentials would also not explain the observed differential settlements.

Differential Settlement

The documentation and author's observations indicate that the lack of serviceability was caused by specific joint failures behind the retaining walls. In the east pipe, the 10-m long pipe bridge had 60 mm of water in it while the pipe immediately south of it had 160 to 180 mm of water in it. There was a distinct and measurable step of 100 mm between the two pipes at this joint indicating that the pipe in the berm had moved down relative to the pipe bridge.

In the west pipe, notable settlement patterns were visible even though the pipe bridge had been removed. The water depths in the pipes north and south of the pipe bridge indicated that about 80 to 190 mm of differential settlement occurred with the greatest settlements occurring in the centers of the berm and not at the pipe bridges. The pipe section just north of the pipe bridge had 150 to 300 mm of water in it compared to 90 mm on the south side and 60 mm in the east pipe at the same joint. The 150 mm of water at the north joint of the west pipe bridge is similar to the 170 mm at the south joint of the east pipe bridge where a 100 mm vertical offset was noted at the joint.

At both of these joints where large shear movements appear to be a factor, the greatest settlements appear to be at the opposite end from the failed joint. This is inconsistent with the hypothesis that erosion of the soils by the retaining wall due to leaking bands caused the differential settlement, in which case the pipe would have settled more at the failed joint than at the opposite end.

The observed settlement patterns indicate that the pipes appear to settle more in the center of the berms than at the edges near the inlets, outlets, and pipe bridge retaining walls. The observed joint deformations of larger gaps at the bottom than at the tops are additional evidence of such a sagging settlement pattern. Since the pipe bridges are supported on rigid retaining walls at both ends of the pipe, the pipe bridges are very stiff and unable to rotate or deflect to accommodate the settlements. The 0.33-m band joints are not designed to accommodate large moment and shear conditions and would be prone to failure at these locations.

The inspector's daily logs indicate that during forming of the walls in mid-November, the "east structure" appeared to be 100 to 120 mm low and that the crushed stone base north of the retaining walls had settled sufficiently to require a new lift of geogrid and stone placed prior to final setting of the pipes in early December. According to the inspector's daily logs, the final as-built elevations of the pipe matched the design elevations. The post-failure survey data and water depth observations indicate that total and differential settlement of the pipe base continued after construction.

The author evaluated potential settlements of the soils using the available boring logs, design drawings and published compressibility correlations. Most of the soils were somewhat preloaded due to the existing dikes. However, new load was placed in several areas, particularly north of the retaining walls where the drawings indicate up to 1.5 m of new material was placed and just south of the retaining walls where the dike slope was widened by placing 0.3 to 0.6 m of new fill. These locations coincide with the largest noted differential settlements of the pipes.

Settlement calculations were performed using assumed parameters to evaluate if settlements similar to those measured could be predicted. Relatively conservative soil parameters were used to simulate the parameters that might typically be used in design considering the lack of site specific compressibility testing. As will be shown below, the actual measured settlements were less than might have been predicted during the design stages. As shown in Table 1, the following parameters were assumed based on the descriptions in the boring logs and published correlations (Macfarlane, 1969):

Table 1
Estimated Organic Soil Compression Parameters

Soil Type	Organic Con tent (%)	Water Content (%)	Bulk Density (kg/m^3)	Compression Index (C_c)	Secondary Compression Index (C_α)
Peat	10% to 50%	100% to 400%	1120 to 1440	1 to 3.5	0.035 to 0.21
Organic Silt	5% to 10%	50%	1440	0.6	0.02

The north berm was assumed to have 1.2 m of additional fill with a 1.5 m wide crest and 3H:1V sideslopes. Assuming the peats and underlying soils were normally consolidated at the time of berm construction, 0.6 to 0.9 m of primary consolidation was estimated. It is likely that loading prior to and during construction reduced the actual primary consolidation after pipe placement to a fraction of this amount. The reported settlement of the pipe bedding course confirms this observation.

Elastic theory indicates that the change in stress under the center of the embankment is approximately double the stress change at the edge of the embankment. The corresponding primary consolidation settlements in the center of the embankment would also be approximately double the settlements that would occur at the edge.

The likely secondary compression of the soils was computed assuming a compressible layer thickness of 6 m. The time for primary consolidation was assumed to be completed within days based on the field observations made during construction. The estimated secondary compressions ranged from 0.1 to 0.3 m per logarithmic cycle of time. Since massive leakage was reported in the spring of 1992, the pipes had been in place only one or two logarithmic cycles of time prior to failure.

The inclusion of the geogrid in the pipe and footing base course and the request to raise the dikes by 0.3 m indicates that settlement was expected by the designer. However, the geogrid is substantially more flexible than the pipe, particularly the very stiff retaining wall and pipe bridge system. It is the author's opinion that the inclusion of the very stiff pipe bridge structure created relatively large rotations and shears at the pipe bridge joints that were not accounted for in the band design. The geogrids below the footings were probably a major assistance in providing a firm construction platform but would not have significantly modified the settlement pattern of the underlying soils loaded by the footings or the berm.

The relatively large deformations that occurred in the middle of the berms, while causing large gaps to open up at the pipe joints, did not appear to have been a serious short-term serviceability problem, although continued secondary compression could have ultimately caused unacceptable leakages and erosion from those joints. The primary serviceability issue appeared to be due to the large breaks that occurred close to the retaining wall allowing water to flow at a relatively steep gradient into the channel between the retaining walls. This steep gradient, combined with the high flow rate, caused the erosion to occur behind the walls, ultimately permitting nearly unabated drainage from the pipe.

CONCLUSION

It is the author's opinion that mitigation of the differential settlement potential at the pipe joints needed to be considered during design. Traditional heavy construction using soil fills and concrete over organic soils is well known to potentially cause large settlements. Although they were probably a useful construction aid, the geogrids incorporated into the design were too flexible relative to the stiff culvert pipe and retaining wall system to be the primary settlement mitigation method. We note that the reconstructed water control structure has apparently performed adequately indicating that a simple preloading program could have been sufficient to prevent the serviceability failure. Alternative measures could have included flexible couplings at the joints or continuation of the reinforced concrete footings along the entire pipe length instead of just below the retaining walls and inlet structure. The use of lightweight fills might also have reduced the differential settlements sufficiently to prevent excessive joint deformation. A less rigid wall system, such as a geosynthetic reinforced berm, may have avoided some of the abrupt differential settlements that caused the joints by the walls to fail.

REFERENCES

Macfarlane, I.C. (1969) "Muskeg Engineering Handbook," University of Toronto Press, Toronto, Ontario.

GEOSYNTHETIC CONSTRUCTION ESTIMATING - HOW THE QUALITY OF THE BID PACKAGE AFFECTS THE QUALITY OF THE PROPOSAL

CURTIS SPENCER
SERROT CORPORATION, U.S.A.

ABSTRACT

In today's competitive market, the geosynthetic installation contractor must contend with satisfying the increasing requirements of the owner, engineer, and regulatory agencies. This environment places increased demands on the estimator to keep costs as low as possible and still meet the specifiers minimum requirements. The level of accuracy required in the preparation of the cost estimate is greater than ever before. However, the accuracy and detail of the RFP (Request For Proposal) package greatly affects the material selection, construction techniques, production rates, and ultimately the price quoted. This paper addresses the basic process involved in creating a cost estimate, the design, constructibility, and material issues associated with a project, and those sections of the bid package critical for the creation of an accurate, cost competitive estimate, free of exceptions and ambiguities.

CREATION OF AN ESTIMATE

The procedure for creating a geosynthetic estimate is essentially the same as any other type of cost estimating. The estimator will review the specifications to identify the contractors scope of work, what materials are required, how the project is to be bid, and any contract conditions which effects the cost of the job. Next, the drawings are reviewed for conformity to the scope of work. There are times when the drawings and specifications are not consistent and judgment call is necessary. The drawings are also reviewed to determine the constructibility of the project, difficulty factors associated with installation such as the steepness of slopes, access to the area to receive geosynthetic installation, and distance from the stockpile to the work area. Next an area takeoff is performed. The final step of the process is the creation of the actual cost estimate. This includes the costing of materials and freight, assigning production rates to the various work items to determine a labor cost, calculating equipment cost, etc. Once the cost estimate is completed the project is ready to price.

BASIC REQUIRED INFORMATION

The content and quality of bid packages range from a hand sketch drawing on a scrap piece of paper to a full specification and drawing package. Since the estimate can only be as accurate as the information from which it is derived. The more accurate and complete the bid package, the more accurate the resulting cost estimate and proposal. As a minimum, the following information is required:

Method of Payment - Lump Sum, Unit Price, Time and Materials, or Cost Plus

Wage Status and Rates - Union, Non-Union, Prevailing Wage, or Davis - Bacon

Tax Status - Taxable or Exempt

Material Requirements - Properties and testing frequencies

Drawings - Plan view, cross-sections, and details

Projected Start Date - Time of year installation will occur

Project Location - Detailed site location or map

Special Provisions - Any special requirements which would effect the cost of the project

METHOD OF PAYMENT

Geosynthetic projects are usually bid on a lump sum or unit price basis. Time and material or cost plus bids are usually limited to repairs or special projects where the total scope of work cannot be clearly defined at the bid stage. Traditionally, the majority of geosynthetic projects have been bid on a unit price basis. However, over the past several years we have seen more projects being bid on a lump sum basis. Both types of bid have advantages and disadvantages. Many owners who request that a project be bid on a lump sum basis do so to prevent area disputes at the end of the project. Most geosynthetic contractors understand this and have no problem bidding a lump sum project, provided that the drawings are accurate with a clearly defined limit of work. If the work limits are not clearly defined, the estimator is likely to be more conservative in calculating his areas and a higher bid price will be the result. In a unit price contract the geosynthetic contractor is paid for the total amount of material installed. Because of this the contractor is less concerned about pay limits being clearly defined on the drawings. However, it is important for the contractor and owner's representative establish a clear and consistent method for measuring and agreeing on the installed quantities to prevent area disputes at the end of the project.

PAY AREAS

For projects bid on a unit price basis it is very important to both the owner and contractor to clearly define how the pay area will be measured. There are three basic methods for measuring the pay area:

Net Lined Area - All materials installed as measured along the plane of the liner, including the material buried in the anchor trench.

Plan View Area - All materials installed as measured along the plan view surface with no adjustment for slope or other vertical surfaces.

Plan View or Net Lined Area Excluding Anchor Trench - No payment for material buried in the anchor trench. Areas are usually measured to the upper inside corner of the anchor trench.

If rubsheets or rain diversion flaps are required it is important to specify whether these areas will be incidental or be included in the measured area. It is also important to identify the limit of geosynthetics on both the plan view and details to avoid confusion. If pay areas are not clearly defined, the owner is faced with the possibility of receiving unbalanced bids up front and a major area dispute at some point during the construction of the project.

OVERALL SITE LAYOUT AND ENVIRONMENTAL CONSIDERATIONS

The effect that site conditions have on job cost is often overlooked by most people and probably contributes to as many claims by geosynthetic contractors as area disputes. The first issue is access to the area to receive geosynthetic installation. Typically, most contractors would like a twenty foot wide path around the perimeter of the area from which to deploy materials. Most liners and geosynthetic clay liners (GCL's) are deployed using a front-end loader, reach-lift, or other heavy equipment. Most of this equipment requires a minimum width of fifteen feet behind the anchor trench to turn parallel to the slope. If materials have to be deployed at an angle to the slope, additional handling is required and productivity is lowered. Likewise, if the material stockpile is located an excessive distance from the work area, additional equipment will be required to "feed" the deployment equipment or productivity is reduced due to longer transit time for the deployment equipment. Easy access into the floor of the work area is also important to decrease downtime on the part of the deployment crew. Based on the drawings, the geosynthetic installer will take these access concerns into account when assigning production rates to a project. If differing access conditions are encountered when the installation crew arrives on site, a dispute or claim will follow shortly. Environmental conditions also play a large role in determining the productivity of a given project. For example, high wind velocities will necessitate the placement of additional temporary ballasting of geosynthetics or stop geosynthetic installation all together. High or low ambient temperatures can also effect production rates. High temperatures may force the crew to work only during the cooler part of the day or even at night. Cold temperatures may force the preheating of liners to achieve acceptable welds or seaming in portable shelters to prevent snow from blowing into the seam area.

DESIGN OPTIMIZATION VS. MATERIAL AND INSTALLATION LIMITATIONS

Design considerations such as slope lengths, appurtenances, and the shape of the area to be lined also effect the price of the project. There are practical maximum lengths to which geosynthetics can be manufactured or handled in the field. There are roll diameter and weight limitations that the winding equipment utilized by most manufacturers can handle. Larger diameter geosynthetic rolls will also reduce the amount of material that can be shipped on a truck, thus increasing freight cost. In the field, larger and or heavier rolls may require special deployment equipment and reduced production rates, which also will increase the cost of the project. The design engineer needs to take these maximum panel lengths into account when determining the length of the slopes. Intermediate benches may be required for extremely long slopes to accommodate these roll lengths and so that the geosynthetics can be seamed without concern for stresses in the seam area. The shape of the work area is also reviewed to calculate the scrap allowance required for a project. A typical rectangular landfill cell with moderate 3:1 slopes will require a scrap allowance of six to seven percent. However, the same landfill cell with long, bowl shaped corners may have a scrap allowance of ten to twelve percent, and some landfills with very irregular shapes will result in a calculated scrap allowance up to twenty percent. Details such as liner attachments, tie-ins, and penetrations are also reviewed for constructibility as well as cost.

MATERIAL SPECIFICATIONS

Approximately ninety percent of these projects estimated have material specifications to which exceptions have to be taken. This is because the property values specified are written around a specific manufacturer's material or the engineer has seemingly based the material properties on design requirements without regard to what is commercially available in the marketplace.

While all geosynthetics may look the same, the various manufacturing processes by which these materials are created can give them a wide range of property values. Because of this, it is prudent when writing a specification to compare the published property values from several different manufacturers to ensure that the specification can be met. When reviewing the published property values, it is important to note if they are listed as typical, minimum, or minimum average roll values. Many times a particular manufacturer's product is specified in a bid request along with the manufacturers typical property values, only the typical property values are specified as a minimum or minimum average roll value. This forces the manufacturer to take exception to their own specification. The typical and minimum average roll values for an 8 ounce geotextile are listed in Table 1.

Table 1. Typical and M.A.R.V values of 8 ounce geotextile.

Property	Typical Value	M.A.R.V. Value
Grab Tensile	1110 N	975 N
Grab Elongation	60%	50%
Mullen Burst	3240 kPa	2895 kPa
Puncture	685 N	600 N
Trapezoid Tear	465 N	420 N
Apparent Opening Size	.150 US Sieve mm	.180 US Sieve mm
Permittivity	1.8 sec-1	1.5 sec-1
Permeability	.48 cm/sec	.38 cm/sec

Fortunately, steps are being taken to help the specifier with this problem. The Geosynthetic Research Institute (GRI) has issued GRI standard GM13, which is a standard specification for smooth and textured HDPE liners. The significance of this specification is that the property values and testing frequencies published have been approved by all of the material manufacturers. The committee that developed Standard GM13 was comprised of engineers, testing agencies, resin suppliers, and regulators as well as the material manufacturers. Similar standard specifications for geotextiles and geonets are planned for the future.

A great amount of confusion involves the use of the words typical or nominal, minimum, minimum coupon, minimum average, and minimum average roll value (MARV) when associated with property values. Engineers, manufacturers, and testing laboratories interpret these terms differently. It is very important that the specifying engineer clearly state if the property values are typical, minimum, or MARV and give a definition of what the term means.

Another area of confusion is determining and defining the difference between performance and index properties. As an example, geotextiles are primarily used as a cushion or filter material. If the material is to be used as a cushion, the properties such as grab strength, grab elongation, mullen burst, and puncture resistance are the performance properties. If the geotextile is used as a filter material, properties such as A.O.S., permittivity, permeability, and water flow are important. In either case, thickness and weight are index properties and should not be used as a basis for material specification.

One final area of concern regarding material specifications is friction angle interface values. No material manufacturer can certify to a friction angle value without first performing shear box testing in accordance with the parameters specified. The engineer should perform shear box testing with several different types of material prior to writing the specification to determine what values are achievable with an acceptable factor of safety. Because of difficulties associated with repeatability of test results, several tests may have to be performed on each interface to develop an average value the engineer can have confidence in. Studies performed by Rivette, Spikula and Nava (1993) as well as Criley and Saint John (1997) have documented the difficulty of repeatability with interface shear testing.

QUALITY CONTROL AND QUALITY ASSURANCE TESTING

Many times it seems the maxim "more is better" has been applied to the frequency of manufacturer and conformance testing frequencies specified. The fact of the matter is that, depending on the property, a higher test frequency does not guarantee a higher quality product. When you are talking about geosynthetics, many of the properties of the finished product are dependent on the base resin used. For example, the physical properties most often listed for HDPE geomembranes are thickness, density, tensile properties, tear resistance, puncture resistance, stress crack resistance, carbon black content and dispersion, and melt flow index. Of these properties, the base resin used determines the density, stress crack resistance, and melt flow index. Therefore, all of the material produced from a given resin batch or lot will have the same property values and should be tested on a resin batch basis. The other properties can be effected by the additive package used or the manufacturing process and should be tested at a higher frequency to ensure performance. The same concept holds true for other geosynthetic materials as well. It is a good idea to talk to the manufacturers representatives to determine which properties are resin dependent or properties which historically have little or no variance from roll to roll.

PROJECT DURATION AND SCHEDULING

Geosynthetic installation rates are usually calculated in square feet, square yards, or square meters per man-hour. By estimating the project in this manner it is simple to calculate the size of the crew required to install a given quantity of material on a daily basis in order to meet the owner's schedule. There are also practical minimum and maximum crew sizes that can be utilized for efficient geosynthetic installation. Since the rate at which geosynthetics can be installed is largely dependent on subgrade preparation and backfilling of geosynthetics, it is important for the specifier to take these critical path items into consideration when determining the project duration.

SUMMARY AND CONCLUSIONS

There is virtually no area of the specifications or drawings of a bid package that does not have some effect on the cost estimate. Because of this, it is paramount that the specifier be as complete and accurate as possible in the preparation of these documents. The specifier should review the bid package from the perspective of the bidder, to assure that the project can be efficiently constructed. If it is not complete enough to construct, then it is not complete enough to bid. Only by being thorough can the specifier be assured of receiving quotations that are all based on the same scope of work and quality of work, without conditions or exceptions.

REFERENCES

GRI Standard Specification, GRI Standard GM13 for "Test Properties, Testing Frequency and Recommended Warrant for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes"

Criley, K. R., and Saint John, D. (1997) "Variability Analysis of Soil VS. Geosynthetic Interface Friction Characteristics by Multiple Direct Shear Testing", Proceedings Geosynthetics '97: Interface Friction Testing, Industrial Fabrics Association International, North American Geosynthetics Society, International Geosynthetics Society, Long Beach, CA. Pp. 885-897

Rivette, Chuck A., Spikula, Dan R., and Nava, Robin C. (1993) "Concerns and Experiences With Textured Geomembranes," Proceedings 7th GRI Conference on Liner Systems: Innovations, Concerns, and Designs, Industrial Fabrics Association International, St. Paul, MN. Pp. 131-142.

TWO FREQUENTLY ASKED QUESTIONS ABOUT GEOSYNTHETIC CANAL LININGS "DO THEY WORK?" AND "HOW MUCH DO THEY COST?"

JACK HAYNES
DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, USA

JAY SWIHART
DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, USA

ABSTRACT

The Bureau of Reclamation (Reclamation) is collaborating with several irrigation districts in central Oregon to demonstrate and evaluate various canal linings under actual field conditions. This paper shows the results of more than 6 years of field testing and includes a series of photographs showing the subgrades, construction, and required maintenance. This paper also examines the effectiveness of the various geosynthetics and other materials to control seepage and compares construction costs.

Uncontrolled field testing of 23 types of geosynthetics canal liners exposed the various materials to very harsh conditions including freeze/thaw, wet/dry, direct and indirect sunlight, extreme rocky subgrades, wildlife (elk, deer, rodents, cattle, etc.) and man. The test sections typically range in size from 150 to 300 meters long with surface areas between 1,400 and 2,800 m^2. Canals in this study had fractured basalt bottoms and typically lost 35 to 50 percent of the flow to seepage. Pre-construction seepage rates as determined by full-scale ponding tests ranged from 0.43 to 1.6 m^3/m^2-day. Following installation of geosynthetics linings, average seepage rates were reduced to less than 0.03 m^3/m^2-day.

BACKGROUND

This paper describes the Deschutes Canal Lining Demonstration Project. To date, 23 test sections have been constructed on five irrigation districts. Three of the districts are near Bend, Oregon, one near Altus, Oklahoma, and one near Maupin, Oregon (see figures 1, 2, and 3). The lining materials include combinations of geosynthetics, concrete grout, shotcrete, elastomeric coatings, and sprayed-in-place foam (SPUF). The test sections are being evaluated for durability and effectiveness in reducing seepage. The test sections now range in age from 6 months to 6½ years, and the differences in performance are becoming apparent, see table 1. Over the first 6 years of testing, three interim reports have documented construction, short-term effectiveness, and maintenance requirements.

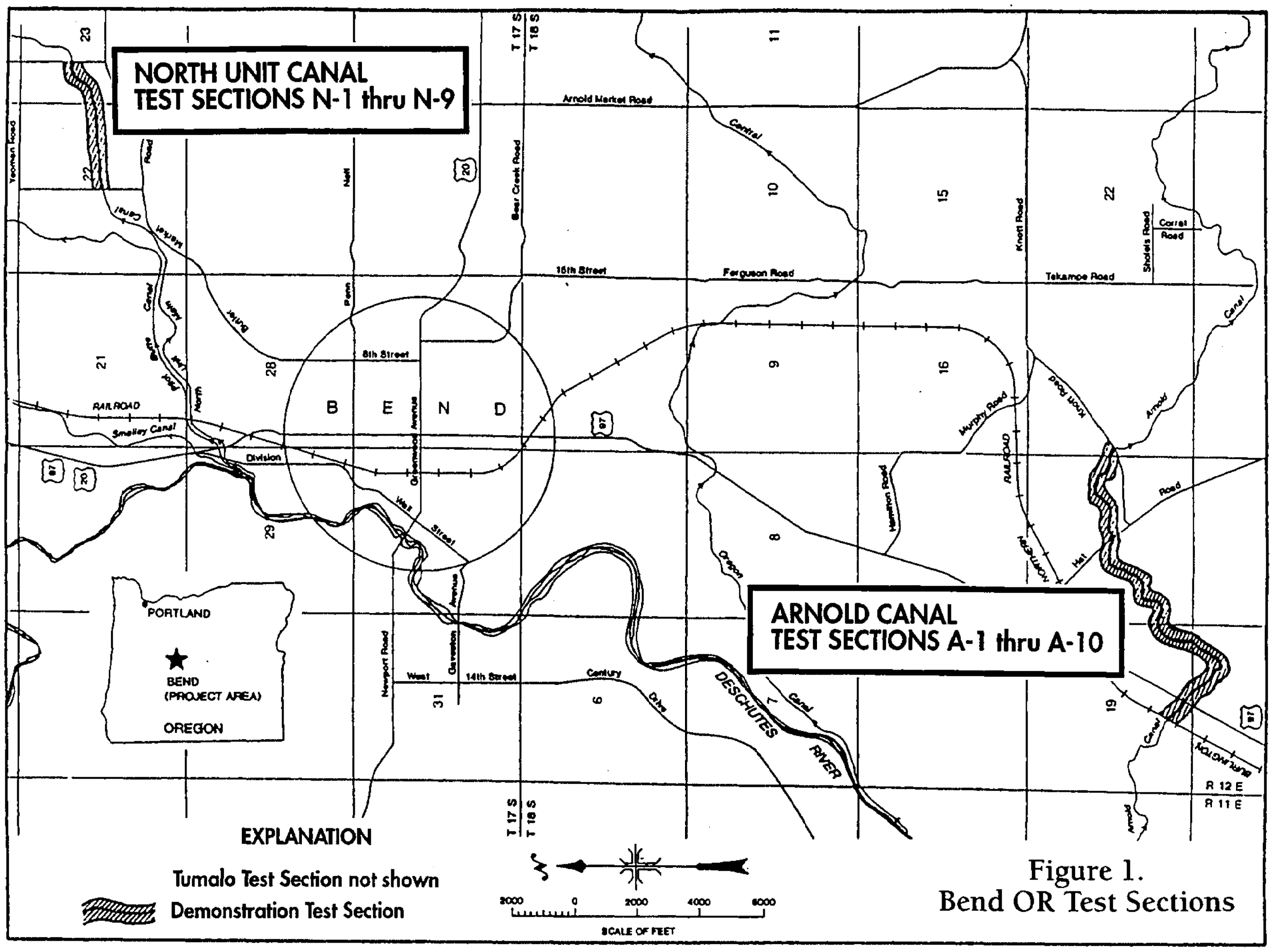

Figure 1.
Bend OR Test Sections

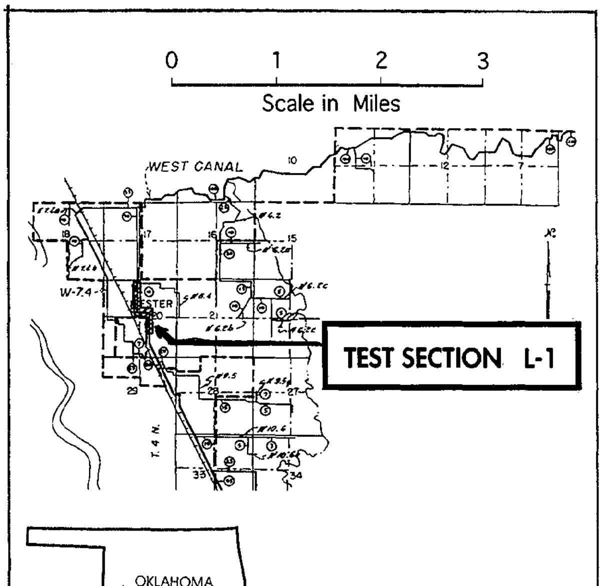

Figure 2.
Altus OK Test Section

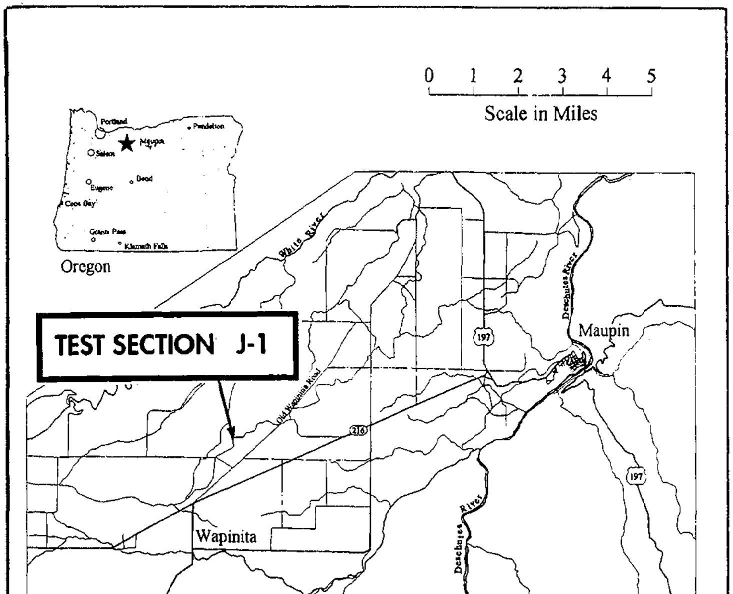

Figure 3.
Juniper Flat Test Section near Maupin OR

Table 1. - 6½ Year Condition Assessment and Construction Costs

	Test Section Description	Cost ($/m²)	Condition (age)	Comments
A-1	Polyethylene Geocomposite with 75-mm Shotcrete cover	$22.20	Excellent (6 years)	No problems
A-2	0.75-mm VLDPE with 75-mm Shotcrete cover	$23.00	Excellent (5½ years)	No problems
A-3	Exposed 2-mm HDPE	$14.80	Very good (5½ years)	Several small tears and cuts
A-4	Exposed PVC Geocomposite	$11.30	Very good (6 years)	Several small tears and cuts Unbonded geotextile seams
A-5	Exposed 1.1-mm CSPE	$11.90	Very good (6 years)	Several small tears and cuts
A-6	Exposed 0.9-mm CSPE Geocomposite	$11.10	Very good (6 years)	Several small tears and cuts
A-7	1-mm PVC with 75-mm Grout-filled mattress	$25.40	Excellent (6½ years)	No problems
A-8	75-mm Grout-filled Mattress	$20.00	Excellent (6 years)	No problems
A-9	Exposed VLDPE with Grout-filled Mattress on Side Slopes only	$19.30	Removed from Study after 28 months	Liner "whales" were impeding flow
A-10	Exposed HDPE with Grout-filled Mattress on Side Slopes only	$19.30	Removed from Study after 28 months	Liner "whales" were impeding flow
N-1	SPUF with Urethane Protective Coating	$46.40	Poor (5 years)	Partial Foam wash-out, Invert replaced with RCC
N-2	SPUF with modified Urethane Protective Coating	$42.20	Poor (5½ years)	Partial Foam wash-out, Invert replaced with RCC
N-3	Woven Geotextile with modified Urethane Coating	$28.40	Failed (1st day)	Complete Failure (May 1993)
N-4	Needle-punched Geotextile with modified Urethane Coating	$28.40	Failed (1st day)	Complete Failure (May 1993)
N-6	75-mm Shotcrete with steel fibers	$17.10	Excellent (6 years)	No problems
N-7	75-mm Shotcrete with Polyfibers	$15.80	Excellent (6 years)	No problems
N-8	75-mm Shotcrete with fibrillated Polyfibers	$15.80	Excellent (6 years)	No problems
N-9	75-mm Unreinforced Shotcrete	$14.30	Excellent (6 years)	No problems
T-1	Neoprene-Asphalt Emulsion over an Existing Concrete Flume	$18.30	Poor (4 years)	Disbonded from Invert
T-2	Neoprene-Asphalt Emulsion over a Sandblasted Steel Flume	$23.20	Very Good (4 years)	40-50 blisters in the Invert
T-3	Neoprene-Asphalt Emulsion over a Broomed Steel Flume	$15.10	Very Good (3 years)	About 40 blisters in the Invert
L-1	Exposed 4-mm Bituminous Geomembrane	$15.00	Very Good (4 years)	Partial wash-out has been repaired
J-1	Exposed 4-mm Bituminous Geomembrane	$15.00	Excellent (½ year)	

GEOLOGY

Oregon's volcanic geology contributes to high seepage rates (Gilbert, 1991), and canals in the area typically lose 35 to 50 percent of their water to seepage because they have fractured basalt bottoms (Figure 4) and/or sides of highly porous soil, or soil and rock (Figure 5). The fractured basalt subgrade also hinders excavation in the canal prism. Therefore, specialized lining technologies are needed to reduce seepage in these areas. Subgrade conditions for the one test section in Oklahoma were mostly fine sands with some gravel.

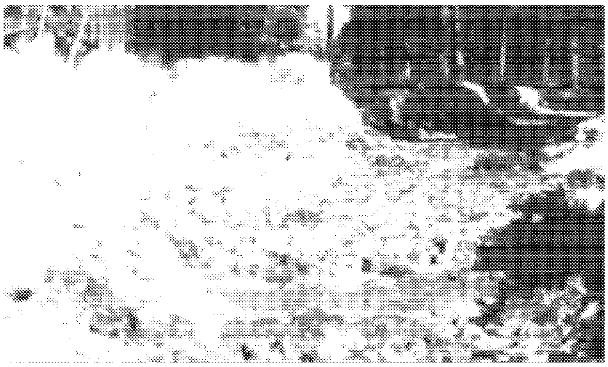

Figure 4- Fractured Basalt Subgrade

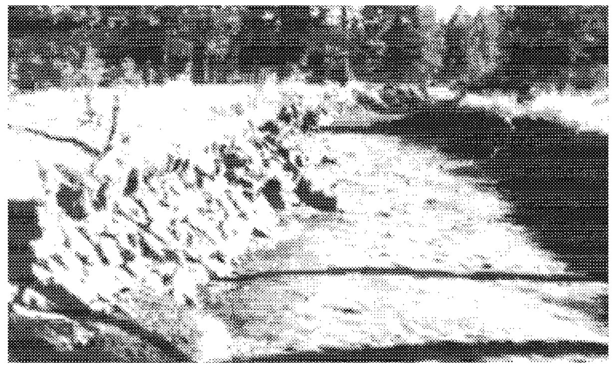

Figure 5- Invert Sediment with rocky sideslopes

PONDING TEST RESULTS

Both pre- and post-construction ponding tests were conducted to determine seepage rates on the Arnold and on the North Unit test sections (Tables 2 and 3). Additional tests are planned for inclusion in the final report.

Table 2. - Arnold Canal Ponding Tests

Test Section	Pre-construction Swihart 1994 $m^3/m^2/day$	Post-construction Swihart 1994 $m^3/m^2/day$	Post-construction Burnett 1997 $m^3/m^2/day$
A1		0.02	0.03-0.09
A2	0.43	0.03	
A3		0.00	0.00-0.03
A4		0.00	0.03-0.06
A5		0.03	0.00-0.15
A6		0.04	
A7		0.03	0.00-0.12
A8		0.06	0.09-0.15
A9	0.20	0.02	
A10		0.02	

Table 3. North Unit Canal Ponding Tests

Test Section	Pre-construction Swihart 1994 $m^3/m^2/day$	Pre-construction conditions After lining removal Burnett 1996 $m^3/m^2/day$	Post-Construction Swihart 1997 $m^3/m^2/day$
N-1			
N-2	1.0-1.6	1.0-1.7	
N-3			
N-4			
N-5		0.7-1.1	
N-6			.13
N-7			
N-8			
N-9			

Test Section A-1 - Polyethylene Geocomposite with 75-mm Shotcrete Cover

Construction Cost = \$22.20/m^2; Dimensions: Length = 300 m; Area = 2,800 m^2

Condition: Excellent - After almost 6 years service, the shotcrete lining is in excellent condition, completely protecting the underlying Petromat geosynthetic liner from weathering and mechanical damage (Figure 6). The only significant damage is that the shotcrete cover is showing extensive cracking over the anchor trench where the shotcrete was tapered-down to a thickness of less than 25 mm (Figure 7). Tapering of the shotcrete over the anchor trench is not recommended for future installations; instead the shotcrete should maintain a minimum thickness of 50 mm over the anchor trench.

Maintenance: Minimal maintenance required to date.

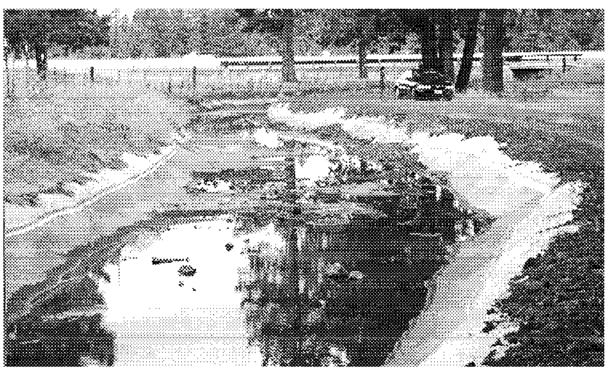

Figure 6- Overview of both A-1 and A-2

Figure 7- Cracking over anchor trench at top

Test Section A-2 - 0.75-mm textured VLDPE with geotextile cushion & 75-mm Shotcrete Cover

Construction Cost: \$23.00/m^2; Dimensions: Length = 150 m; Area = 1,400 m^2

Condition: Excellent - After 5½ years, the shotcrete lining is in excellent condition, completely protecting the underlying VLDPE geosynthetic liner (Figure 6). Dozens of transverse contraction cracks have developed on each bank. Some new cracks appear every year, and many of the old cracks grow in length, but do not widen significantly. Cracking in the thin, tapered shotcrete over the anchor trench is moderate to severe (Figure 7). Again, tapering of the shotcrete over the anchor trench is not recommended for future installations, instead the shotcrete should maintain a minimum thickness of 50 mm over the anchor trench.

Maintenance: Minimal maintenance requirements to date.

Test Section A-3 - Exposed 2-mm textured HDPE

Construction Cost: $14.80/m^2$; Dimensions: Length: 150 m; Area: 1,400 m^2

Condition: Very Good - After 5½ years of service (Figure 8), the exposed HDPE liner is performing well, with only a few small tears over sharp subgrade rocks (Figure 9).

Maintenance: Minimal maintenance required to date.

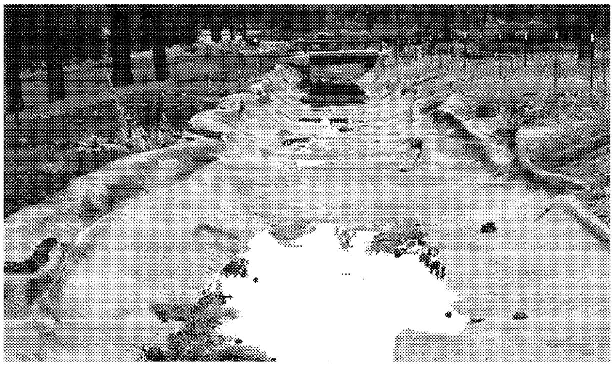

Figure 8- Overview of A-3

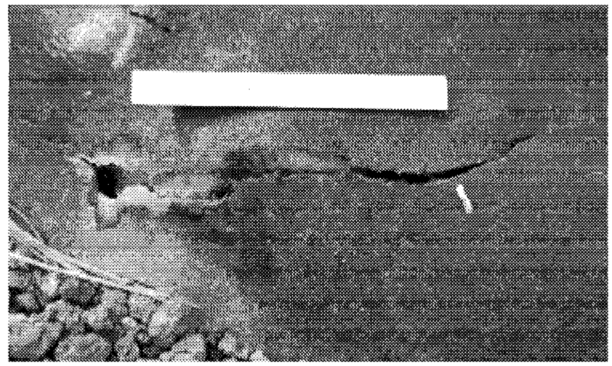

Figure 9- Tear in side wall above water surface

Test Section A-4 - Inverted PVC Geocomposite with geotextile cushion

Construction Cost: $11.30/m^2$; Dimensions: Length = 300 m; Area = 2,800 m^2

Condition: Very Good - After 6 years (Figure 10), the PVC is holding up well with no visible deterioration or stiffening, even where exposed. The geotextile cover is slowly weathering away (especially where unbonded at seams). Sediment (up to 300 mm deep) has collected in the invert providing additional UV protection. Some aquatic growth is impeding flow.

Maintenance: Minor maintenance required to date to repair damage (Figure 11).

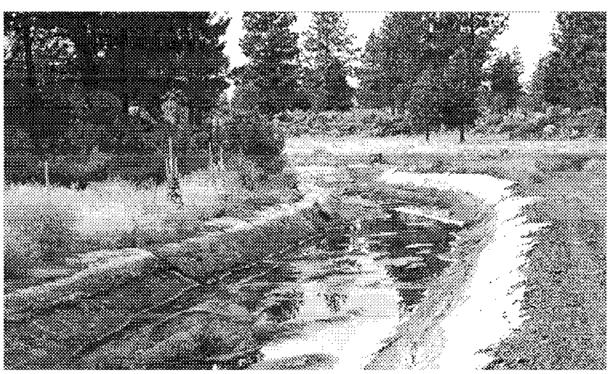

Figure 10- Overview of A-4

Figure 11- Tree fell onto liner during storm

Test Section A-5 - Exposed 1.1-mm CSPE with geotextile cushion

Construction Cost: $11.90/m^2$;   Dimensions: Length = 150 m; Area = 1,400 m^2

Condition: Very Good - After 6 years, the exposed CSPE geomembrane (Figure 12) is holding up well. Standing water and a layer of sediment covers almost the entire invert, typically 0.15 to 0.30 m deep. Some vegetation is growing but has little effect on flow to date. A couple of small tears have developed at the anchor trench (Figure 13) and a sharp subgrade rock has punctured the liner at the waterline.

Maintenance: Minor maintenance required.

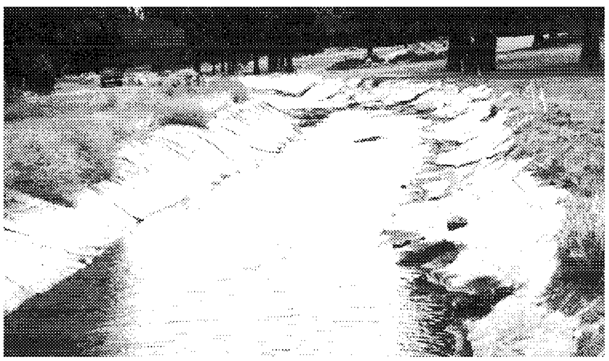

Figure 12- Overview of both A-5 and A-6

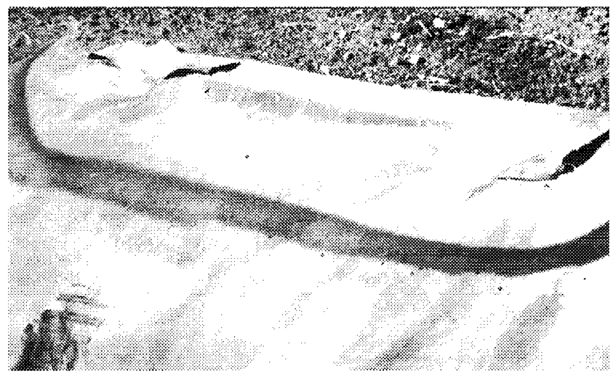

Figure 13- Tears at anchor trench

Test Section A-6 - Exposed 0.9 mm CSPE geocomposite

Construction Cost: $11.10/m^2$;   Dimensions: Length = 150 m; Area = 1,400 m^2

Condition: Very Good - After 6 years, the exposed CSPE geomembrane (Figure 12) is holding up well. The upstream transition between Test Sections 5 and 6 has a transverse adhesive-bonded seam which is working well. Inexperienced backhoe operators have caused more damage to the exposed linings to date than any other element. A few small tears near the anchor trench need to be repaired.

Maintenance: Minor maintenance required to date.

Test Section A-7 - 1-mm PVC with 75-mm grout-filled mattress

Construction Cost: $25.40/m^2$;   Dimensions: Length = 240 m; Area = 2,200 m^2

Condition: Excellent - After 6½ years, the grout-filled mattress is in excellent condition, completely protecting the underlying PVC geomembrane. The mattress is fairly uniformly grouted in spite of the uneven rocky subgrade (Figure 14). The outer fabric is beginning to deteriorate (Figure 15), especially where subjected to abrasion. When the water is turned off, this test section holds water all winter, while the adjacent Test Section A-8 holds water for only a couple of weeks. This side-by-side comparison demonstrates the difference in seepage rates due to the geomembrane underliner.

Maintenance: Minor maintenance required to date.

Figure 14- Overview of both A-7 and A-8

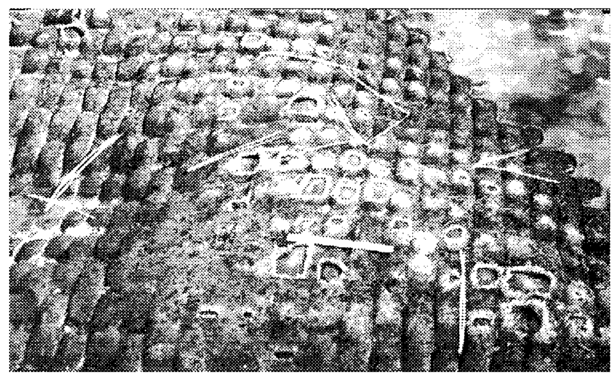

Figure 15- Mattress fabric is wareing away

Test Section A-8 - 75-mm grout-filled mattress

Construction Cost: $20.00/m^2$;   Dimensions: Length = 210 m; Area = 2,000 m^2

Condition: Excellent - After 6 years, the grout-filled mattress is in excellent condition with no freeze/thaw damage (Figure 14). The first 60 m with zippered seams has a much neater appearance than the second 150 m with sewn seams. The grout-filled mattress is well tied-in to the bridge, with no gaps that would allow seepage. The outer fabric of the grout mattress is in good condition, with little deterioration, except for one location on the left bank where the geotextile has worn away, and several concrete "bricks" are missing.

Maintenance: Minor maintenance required to date.

Test Section A-9 - 1.5-mm VLDPE with geotextile cushion and 75-mm grout-filled mattress on side slopes only

Construction Cost: $19.30/m^2$;   Dimensions: Length = 300 m; Area = 2,800 m^2

Condition: Removed from study after 2 ½ years - Liner "whales" were restricting flow (Figure 16). Attempts to deflate the "whales" with knife-cuts, and attempts to ballast with concrete blocks were largely unsuccessful as the "whales" tended re-appear elsewhere. Figure 17 shows contractor patching numerous holes, tears, and rips from sharp subgrade rocks. Eventually, the invert liner was removed with the grout-filled mattress left in place on the sideslopes. The cause of the "whales" in Test Sections A-9 and A-10 was never resolved. Volcanic gases are suspected to be the cause.

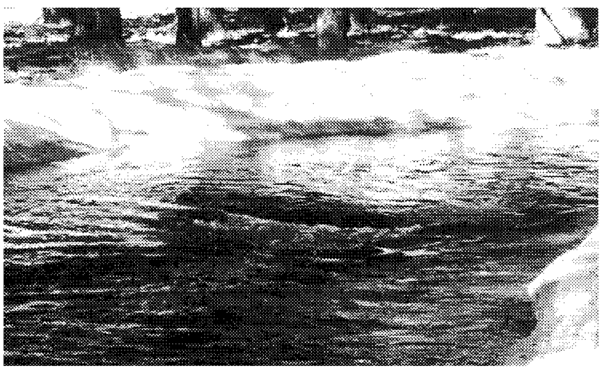

Figure 16- Liner "whale" impeding flow

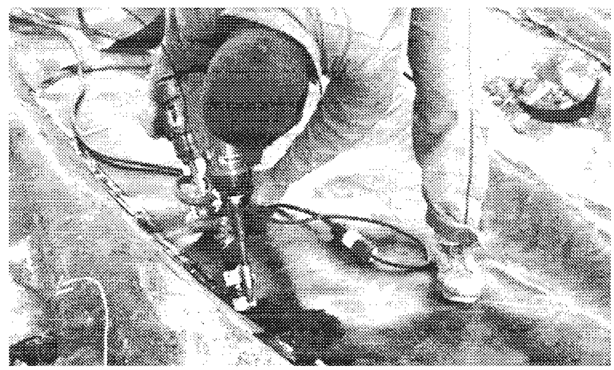

Figure 17- Patching of tears in geomembrane

Test Section A-10 - 1.5-mm HDPE with geotextile cushion and 75-mm grout-filled mattress on side slopes only

Construction Cost: $19.30/m^2$;   Dimensions: Length = 300 m; Area = 2,800 m^2

Condition: Removed from study after 2 ½ years - This test section experienced the same problems with liner "whales" as Test Section A-9. The exposed HDPE was removed in March 1995, and this test section was abandoned. The grout-filled mattress on the side-slopes will be left in place. In many locations, the imported sand bedding had completely washed away, indicating there may have been significant flow beneath the liner.

Test Section N-1 - SPUF with urethane protective coating
Test Section N-2 - SPUF with modified urethane protective coating

Construction Cost N-1: \$ 46.60/m²; Dimensions: Length = 100 m; Area = 1,700 m²
Construction Cost N-2: \$ 46.20/m²; Dimensions: Length = 100 m; Area = 1,700 m²

Condition: Partially failed - After 5 years, most of the invert foam has washed out of these test sections (figure 18). The washout initiated in the first few weeks of service just below the drop at the start of Test Section N-1 where loose sand and gravel deposits offered little uplift resistance to the buoyant foam (figure 19). The high velocities then undercut large, loose subgrade rocks, allowing more foam to break free. The failure then propagated downstream washing out the invert foam in Test Section N-2.

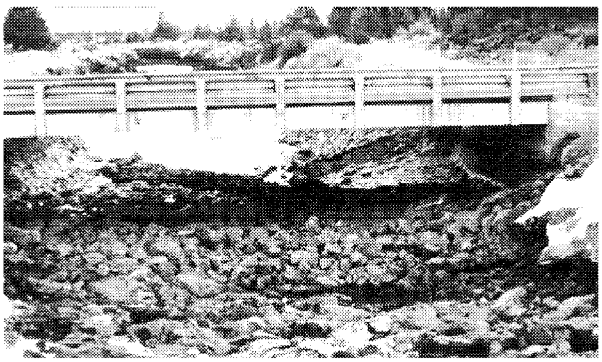

Figure 18- View after fifth year

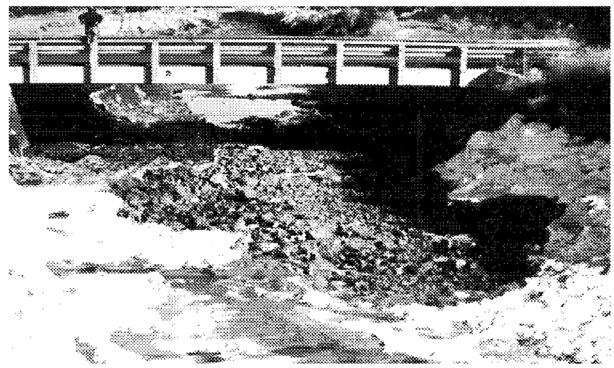

Figure 19- View after first year

Test Section N-3 - Woven geotextile with spray-applied modified urethane coating
Test Section N-4 - Needle-punched geotextile with spray-applied modified urethane coating.

Construction Cost (both): \$28.40/m²; Dimensions (both): Length = 100 m; Area = 1,700 m²

Condition: Complete failure - On the first day of service, large sections of the geotextile liners washed out resulting in complete failure (Figures 20 and 21).

Figure 20- View of foam anchoring system

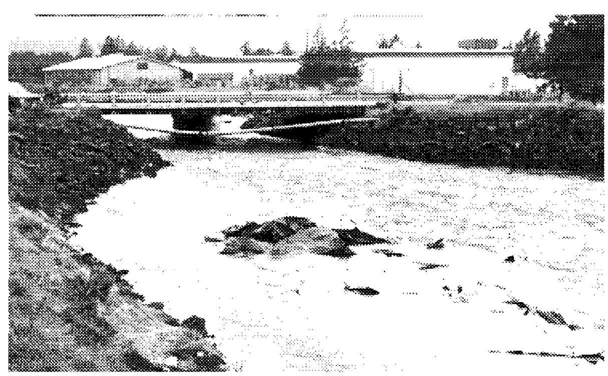

Figure 21- Lining floating downstream

Test Section N-6 through N-9 - General comments apply to all 75-mm shotcrete sections:

Condition: Excellent - After 6 years all the shotcrete is in excellent condition. No visible differences exist in the performance of the four shotcrete test sections. No freeze/thaw damage is evident. Small pools (Figure 22) are present on all four test sections, even several weeks after water turn off, indicating low seepage rates. Contraction cracks on the side walls (Figure 23) have developed every 30 to 60 meters. Crack width varies from hairline to 3 mm. The thickness of the shotcrete is highly variable (ranging from 1 to 6 inches thick) because of the uneven subgrade conditions, and normal problems with field installation quality control. Many large rocks (up to 300 mm diameter) are collecting in the canal invert (perhaps rolled in by local youths). Vegetation is growing out of cracks in the shotcrete near the top of side slopes.

Table 4 - N-6 through N-9 Summary of Basic Data

Section	Discription	Cost ($\$/m^2$)	Length (m)	Area (m^2)
N-6	shotcrete reinforced with steel fibers	$17.10	150	2,800
N-7	shotcrete reinforced with polypropylene fibers	$15.80	150	2,800
N-8	shotcrete reinforced with fibrillated polypropylene fibers	$15.80	150	2,800
N-9	unreinforced shotcrete	$14.30	150	2,800

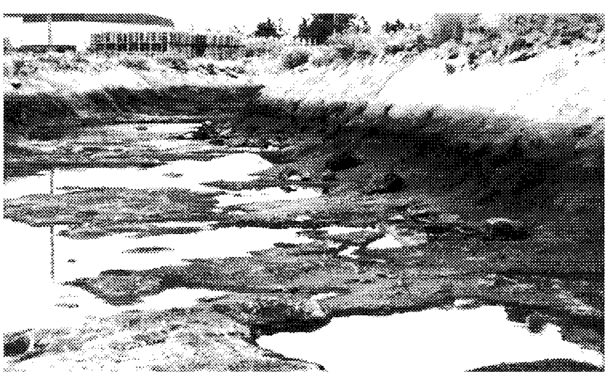

Figure 22- Small pools holding water in winter

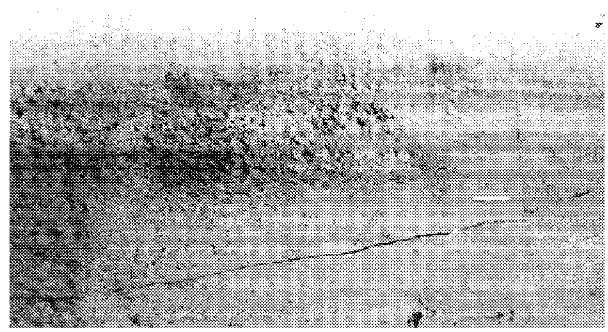

Figure 23- Contraction cracks in side walls

Test Section T-1 - Neoprene asphalt over an existing concrete flume

Construction Cost: $18.30/m^2$; Dimensions: Length = 23 m; Area = 150 m^2

Condition: Poor - After 4 years, the membrane is completely disbonded (due to high volicities) in the invert and has rolled up into the corners against the side walls (Figure 24). Material on the vertical side walls has lots of small tears and pinholes (Figure 25).

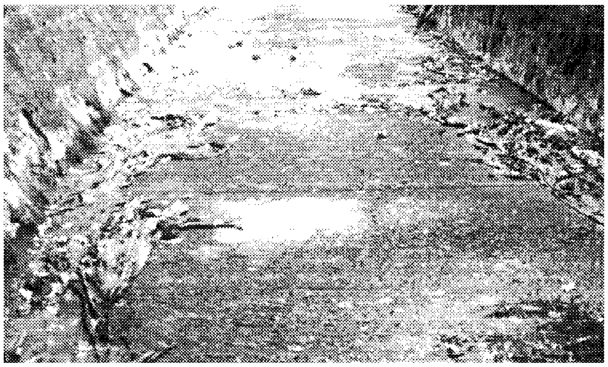

Figure 24- Material rolled up into cornors

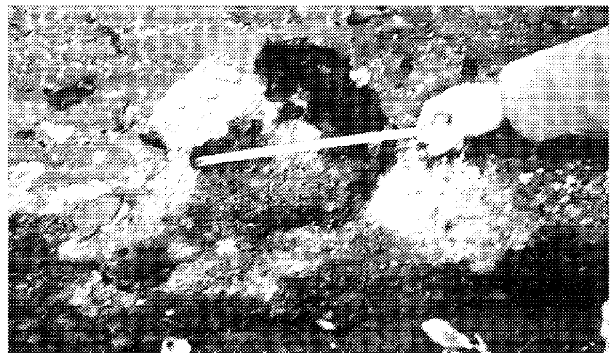

Figure 25- Small holes on side walls

Test Section T-2 - Neoprene asphalt over a sandblasted steel flume
Test Section T-3 - Neoprene asphalt over a broomed steel flume

Construction Cost T-2: $23.20/m^2$; Dimensions: Length = 140 m; Area = 730 m^2
Construction Cost T-3: $15.10/m^2$; Dimensions: Length = 80 m; Area = 420 m^2

Condition: Very Good - After 3 to 4 years the membrane is well bonded to 99 percent of the steel flume (Figure 26). No leakage is evident. Numerous blisters (Figure 27) have developed where the membrane is poorly bonded to underlying old tar material.

Maintenance: Minor maintenance required to repair blisters.

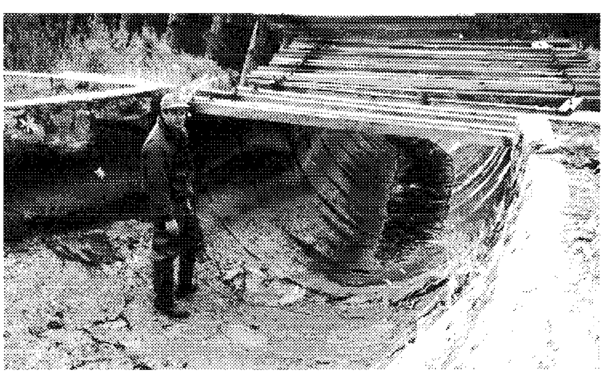

Figure 26- View of coated steel flume

Figure 27- Blisters formed on old tar

Test Section L-1 - Exposed 4-mm bituminous geomembrane

Construction Cost: $15.00/m^2$;   Dimensions: Length = 730 m; Area = 6,500 m^2

Condition: Very Good - After 4 years of service (Figure 28), the geomembrane is in very good condition. Figure 29 shows a piece of new material on top of the 4-year-old material. The alligator cracking began to appear after about one year, but the material remains quite flexible.

Maintenance: Flood waters damaged the anchor berm, requiring minor repairs.

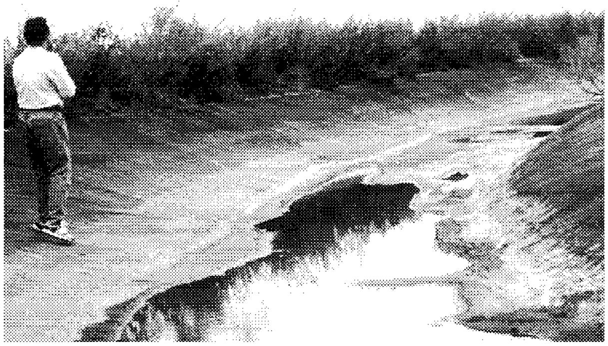

Figure 28- View of canal after 4 years

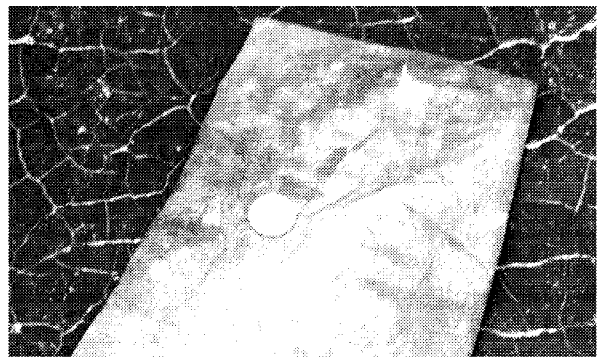

Figure 29- Comparison of liner (new versus old)

Test Section J-1. - Exposed 4-mm bituminous geomembrane

Construction Cost: $15.00/m^2$;   Dimensions: Length = 270 m; Area = 2,200 m^2

Condition: This new installation (only 6 months old) over fractured basalt (Figure 30) has not yet gone through a full irrigation season (Figure 31).

Maintenance: None to date.

Figure 30- Trackhoe trying to excavate basalt

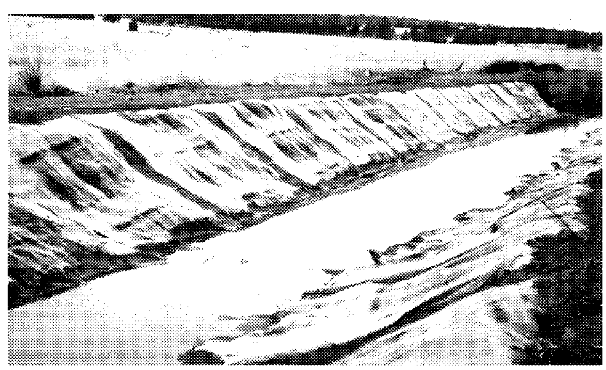

Figure 31- View of completed canal

CONCLUSIONS

How Much Do They Cost?

- This study has identified several effective lining technologies with construction costs between \$11.00 to \$23.00/m^2.

Exposed geomembrane	\$11.00 - \$23.00/m^2
Concrete alone	\$14.00 - \$20.00/m^2
Geomembrane with concrete cover	\$19.00 - \$23.00/m^2

Do They Work?

- Seepage reduction - Post-construction ponding tests showed that seepage had been reduced by 90 to 99 percent depending on the lining material. As expected, the 5-year ponding tests show some increase in seepage; however, seepage rates have still been reduced by 80 to 95 percent depending on the lining material. Geomembranes with concrete cover appear to provide the greatest long-term effectiveness.

- Maintenance - Test sections with exposed geomembranes are subject to mechanical damage and will probably require more maintenance than either concrete linings or geomembranes with concrete cover.

- Durability - Concrete linings have a proven life expectancy of 30 to 50 years. Since geomembranes are relatively new material, exposed geomembranes only have a proven life expectancy of about 20 years at this time.

- Future studies - The long-term effectiveness and durability of these 23 test sections will be addressed in a series of "Durability Reports." Life-cycle costs will include initial construction costs, maintenance costs, and design life (durability). Future ponding tests will be used in Cost/Benefit analysis to calculate the cost of conserved water (\$/hectare).

REFERENCES

Burnett R. (1996) "North Unit Main Canal - Seepage Loss Analysis Ponding Tests", U.S. Bureau of Reclamation, Denver CO

Burnett R. (1997) "Arnold Canal - Canal-Lining Test Sections 5-Year Seepage Evaluation", U.S. Bureau of Reclamation, Denver CO

Gilbert J.D. (1991) "Geology Report", Central Oregon Canal system, Upper Deschutes River Basin Water Conservation Project OR, U.S. Bureau of Reclamation, PN Region, Boise ID

Swihart J.J., Haynes J.A. (1994) "Deschutes Canal-Lining Demonstration Project - Construction Report", Report No. R-94-06, U.S. Bureau of Reclamation, Denver CO

Swihart J.J., Haynes J.A. (1997) "Deschutes Canal-Lining Demonstration Project - Year 5 Durability Report", Report No. R-97-01, U.S. Bureau of Reclamation, Denver CO

CAHOKIA MOUNDS: HISTORY PRESERVED

PETER M. HANRAHAN
NORTH AMERICAN GREEN
UNITED STATES OF AMERICA

ABSTRACT

Serious erosion problems were addressed recently at the Cakokia Mounds in southern Illinois. Monk's Mound, the largest of the Native American ceremonial mounds at the site, was threatened by severe slumping. Several geosynthetic products were used, in combination with other products and techniques, to deal with these problems. Monk's Mound, built in stages from approximately 900 A. D. though 1200 A. D., has survived to date due to the sound construction techniques employed by its builders. With selective use of modern geosynthetic solutions, the State of Illinois has taken steps to preserve this important site for generations to come.

HISTORY PRESERVED

When the Western Hemisphere's largest prehistoric earthen structure was confronted with serious erosion problems, the stakes were very high when possible solutions were considered.

Monk's Mound, the largest of the Cakokia Mounds, was the focus of the restoration efforts. Cahokia, which is the largest prehistoric city in the now United States, is located in southwestern Illinois, about 13 kilometers east of St. Louis. At its peak, around 1150 A. D., Cakokia contained more than 120 mounds. Today, due to a combination of factors, including erosion and human intervention, fewer than 80 remain.

Administered by the Illinois Historic Preservation Agency, the Cakokia Mounds State Historic Site is committed to the preservation and protection of the remaining mounds. Over the past decade of so, slumping on the east and west sides of Monk's Mound has been occurring at an alarming rate. In 1997, funding was released which allowed preservation efforts and corrective measures to be implemented.

Around 1150 A. D., Cahokia was one of the world's great urban centers. It was larger than London at the time, and no city in the United States surpassed it in population until the early 1800's. The city was home to some 20,000 residents at its peak.

Settled and developed by native Americans now called Mississippians, the Cahokia site was occupied from approximately 700 to 1400 A. D. The city flourished from around 1000 to 1200, then began a period of decline.

Most of the mounds at Cakokia were not burial mounds, but were built as platforms as sites for important buildings, temples or lodges. Monk's Mounds is the largest of these platform mounds, and the focus of recent preservation efforts. Monk's Mound was built with earth dug with stone tools and carried in baskets. It covers approximately six hectares and is 30 meters tall. It contains an estimated 2240 cubic meters of earth.

Construction of Monk's Mound began around 900 A. D. It was then enlarged in stages through about 1200 A. D. Since the Mississippian period it hosted several other human settlements, including a chapel built by French priests in the early 1700's, and a farmhouse built in 1831. The State of Illinois acquired 57 hectares at the site in 1925, creating Cahokia Mounds State Park, and setting the stage for more recent preservation efforts.

William R. Iseminger, archaeologist at the site, and also Director of Public Relations at Cahokia Mounds, described some serious slumping which was first observed at Monk's Mound in the mid 1980's. Three particularly wet years had definitely contributed to the problem. At that time, archaeological investigation revealed that previous slumping had occurred, and had been dealt with over time. In search of possible solutions for future slumping, core samples were taken, and monitoring equipment was installed. The cracks and crevices were then filled in, vegetation was reestablished, and the site remained free of serious problems until about a decade later.

More movement occurred in 1994-95. Serious slumping and cracking was observed, particularly on the west slope, which faces the city of St. Louis. Human intervention definitely played a role in this problem. A 19th century farmhouse had been erected on the mound, and, for example, a well had been dug. This was just one of the cracks in the mound surface that needed to be repaired. The discovery was also made that there was an elevated water table in the mound. This condition required the installation of horizontal drains into the west side of the mound, in order to relieve hydraulic pressure beneath the soil surface. These drains were installed in the winter of 1997-98.

The major cracks were filled with sand, and then sealed with geosynthetic clay lining, which was secured with wooden stakes. On top of the lining material, a layer of granular clay was then applied. Finally, a thin layer of fine-grained topsoil was installed. After seeding and fertilizing, single netted straw erosion control blankets were installed to help minimize soil loss and to assist with vegetation establishment.

Each step in this crack filling process was critical to its projected success. In developing the design for this portion of the project, and in choosing the appropriate materials, the consulting engineers involved had one major objective, and that was to prevent water from penetrating the mound surface through the detected cracks.

First of all, the cracks had to be filled. Filling the cracks with sand allowed for maximum void filling and compaction. After crack filling, the geosynthetic clay lining material, secured by wooden stakes, was used to seal the crack openings. This material, which consists of a layer of granular sodium bentonite, bound between two geotextile layers, is designed for low permeability. Here the designers sought to prevent excess moisture from penetrating the original cracks.

Geosynthetic clay liners have been used successfully in a wide variety of field applications requiring enhanced resistance to water infiltration. The products are used extensively in landfill lining, surface containment, surface impoundment and in landfill covers. The sodium bentonite molecules are comprised of stacked platelets that can absorb water and expand, creating a dense yet flexible hydraulic barrier. The use of needlepunched geotextiles on both sides of this material provides high shear strength, allowing the composite material, with a thickness of less than three centimeters, to provide superior hydraulic performance to several feet of compacted clay.

The designers sought further to assure low permeability by applying a layer of loose granular bentonite over the geosynthetic clay lining material. Finally, a layer of fine-grained topsoil was installed to serve as host to the revegetation process.

After seeding and fertilizing, single netted straw erosion control blankets were installed to help minimize soil loss and to assist with vegetation establishment. This final step was taken to assure that all the critical soil layers stayed in place while the revegetation process was occurring.

The single netted straw erosion control blankets chosen consist of wheat straw covered by a single photodegradable synthetic netting. The wheat straw, applied evenly over the entire area of the mat, is then sewn into the netting every 0.59 centimeters. The blankets chosen included netting fortified with photo accelerators, giving the netting a breakdown time of approximately 45 days. Due to the fertile soil conditions at Cahokia, the design engineers were confident of attaining a strong stand of vegetation within that period.

Critical steps to the successful installation of straw erosion control blankets include, most importantly, properly securing them to the ground. In this case, as with the geosynthetic clay liners, wooden stakes were used.

In addition, low profile earthen berms were constructed above the slump area, to divert water from the upper reaches of the mound. This portion of the design package aimed at keeping excess rainwater away from the repaired areas and moving it efficiently to the base of the mound. These diversion channels were lined with composite turf reinforcement mats.

The selected turf reinforcement mats, used in the diversion channels, were chosen to perform several functions. First of all, the mats provide immediate erosion protection, and secondly, they allowed for the establishment of a permanent stand of vegetation. Most importantly, their successful installation assured the establishment of a permanently reinforcement grass-lined drainage swale, with the ability to withstand the wearing effects of storm run-off far in excess of unreinforced turf. Also, the use of these mats allowed the designers the benefit of maintaining vegetative cover over the entire slope profile.

The turf mats selected featured a three dimensional polypropylene netting structure, and included a layer of 100% coconut fiber. The netting, which meets or exceeds all Federal Highway Administration standards for turf reinforcement mats, is designed to anchor the root and stem structure of the vegetation. The coconut fiber is present to add temporary erosion protection and mulch during the revegetation process.

Independent tests conducted by the Texas Transportation Institute have concluded that the selected turf mats successfully withstood flow events of up to 383 Pascal. By contrast, unreinforced turf will fail when subjected to flows of about 176 Pascal.

Like the temporary erosion blankets, the turf mats were installed after grading, seeding and fertilizing. They were also secured with wooden stakes.

Although the west side of Monk's Mound has been slightly altered by the slumping, the decision was made not to attempt to return the mound to its previous profile. It was feared the added weight of the new material necessary to accomplish this might make the slumping problem even worse.

The survival of Monk's Mound to this day suggests that the native Americans who built the structure knew a great deal about erosion control. Archaeologists have discovered, for instance, ancient clay berms at the base of some sloping areas, suggesting early efforts to control slumping. Also, core sampling has revealed carefully placed soil layers, suggesting a clear understanding of soil properties by the builders.

One of the challenges faced at the site was the construction of a service road, to the top of the mound, to be used during construction. So as not to disturb the mound structure, cellular confinement grid, 1.58 centimeters deep, was placed over the layer of non-woven geotextile. The grid cells were then filled with crushed gravel, creating a rugged temporary roadway.

A non-woven geotextile was chosen as the base for the service road. The non-woven material was selected to provide a base to the temporary roadway, and to protect the mound surface from road use damage.

The cellular confinement grid was then placed over the geotextile to provide load support. The three-dimensional cells of the material prevent shear and lateral movement of the infill material. Then, the load is distributed to surrounding filled cells, creating a flexible bridging action. Similar material was used to serve as the base of temporary roadways used for heavy equipment movement over soft sand during Operation Desert Storm.

As the erosion control project was being completed, a second project involved the replacement of a worn wooden stairway to the top of the mound. The wooden stairs, installed in 1980-81, were badly worn and had shifted in many locations. Precast concrete stairs greeted visitors when the mound was reopened to visitors in the summer of 1998.

Prior to the installation of the wooden stairs, climbers of the mound literally converged on the mound from all directions. The wear and tear caused by this random foot traffic caused the creation of gullies up to five feet deep. While the new wooden stairway did channel foot traffic to one area, the steps themselves brought one their own set of problems. Over time, the steps shifted and deteriorated. Despite the fact that the steps were constructed of treated wood, and even though they had been repaired a couple of times in recent years, problems had continued. Water seeping behind the steps only hastened their deterioration.

This project illustrates several key points. Obviously, the use of modern geosynthetic materials can be helpful in dealing with hydraulic and erosion problems at important historic sites. We need to continue to find ways to use modern technology to help us preserve important links to our past. Most importantly, however, the success of this projected involved many different stakeholders in the development of a plan carefully balanced to meet varying needs.

The State of Illinois, charged with the responsibility of preserving the Cakokia site, has more than a half century of experience in dealing with preservation efforts at Monk's Mound. The manufacturers and suppliers of the geosynthetics and erosion control products recommended the use of selected products to deal with the very specific problems encountered at the site. And, finally, the design engineers found the necessary links between the specific needs of the site and the appropriate products and techniques now available.

CONCLUSIONS

Recent preservation efforts at Monk's Mound are part of a continuing process. The administrators of the site have accumulated a great deal of experience in dealing with hydraulic and erosion control problems at the site. Additionally, rapidly emerging technology and design techniques have increased our ability to effectively deal with such problems. As a result, the solutions developed represented a marriage between the site needs and the specific products and solutions needed to effectively deal with those problems.

The geosythetic clay lining material was used to form a barrier to further water infiltration into surface crack areas on the mound.

A non-woven geotextile was used as a base material for the temporary service road, which provided access to the top of Monk's Mound.

Cellular confinement grid was used, in conjunction with crushed gravel, to build the weight bearing service road.

Temporary erosion control blankets were used to protect seeded areas and facilitate a strong stand of native vegetation.

Permanent turf reinforcement mats were installed to permanently reinforce high flow vegetated diversion channels.

Working as a team, the administrators at Cahokia, along with the designers, suppliers and contractors involved, collaborated in an effort to provide solutions to the unique problems associated with this important historic site.

ACKNKOWLEDGEMENTS

The author would like to thank William R. Iseminger, archeologist at the Cahokia Mounds State Historic Site, for his encouragement and assistance in the preparation of this paper. Kevin Van Tuyl of A. S. P. Enterprises of St. Louis was also extremely helpful. Mr. Van Tuyl provided invaluable information regarding the products and techniques employed on the project. Claudia Gellman Mink's excellent book, "Cahokia: City of the Sun," provided important historical details.

REFERENCES

Iseminger, William R., (1997) "Monks Mound: A Moving Monument", Historic Illinois, August, 1997, pp. 9-12.

Mink, Claudia Gellman, (1992) "Cahokia: City of the Sun", Cahokia Mounds Museum Society, Collinsville, Illinois.

Northcutt, Paul, and McFalls, Jett, (1997) "Field Performance Testing of Selected Erosion Control Products, Texas Transportation Institute, Austin, Texas, page 128.

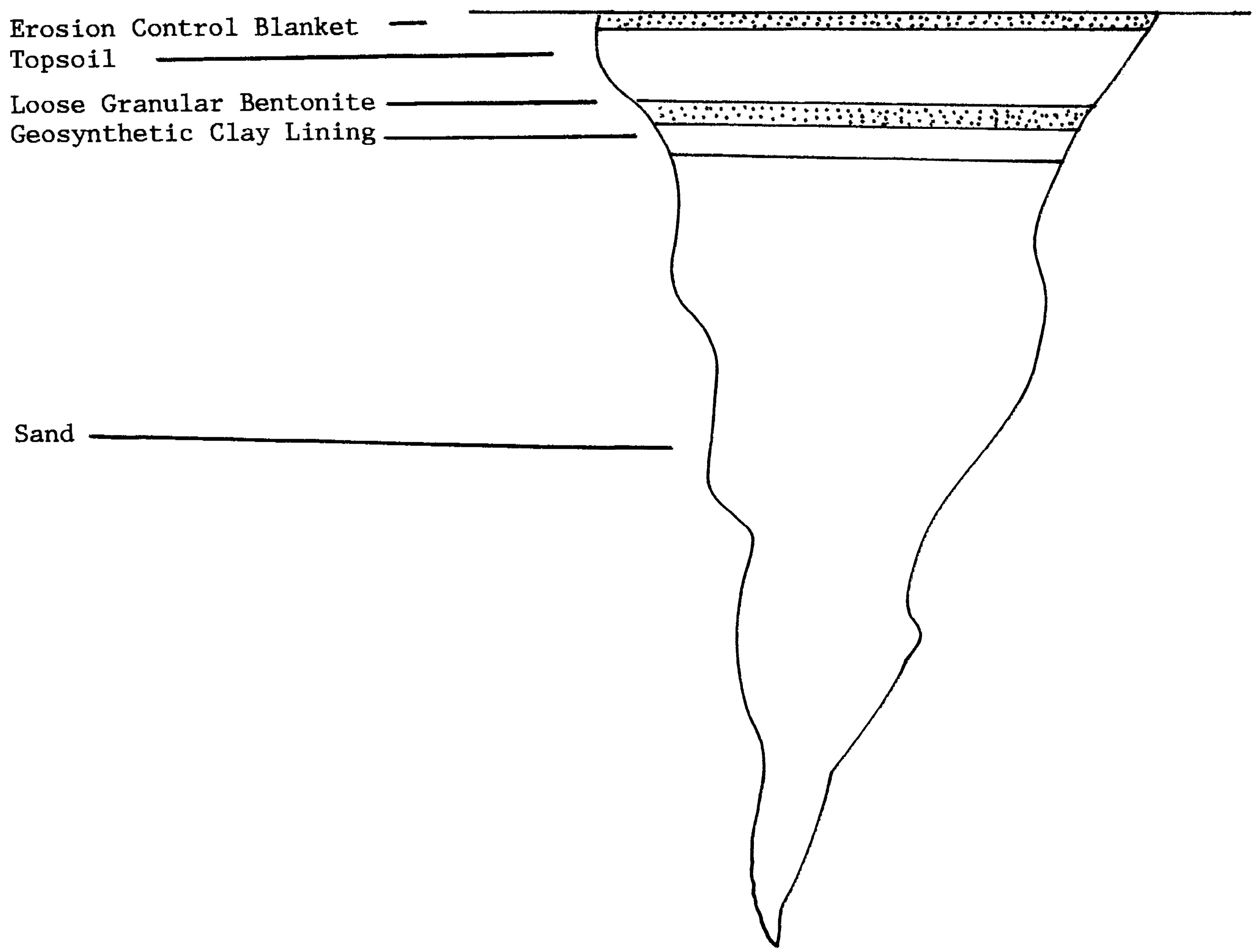

Figure 1. Cross Section of Filled Surface Crack

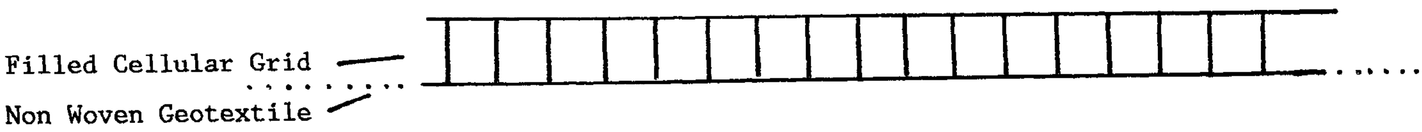

Figure 2. Cross Section of Temporary Roadway

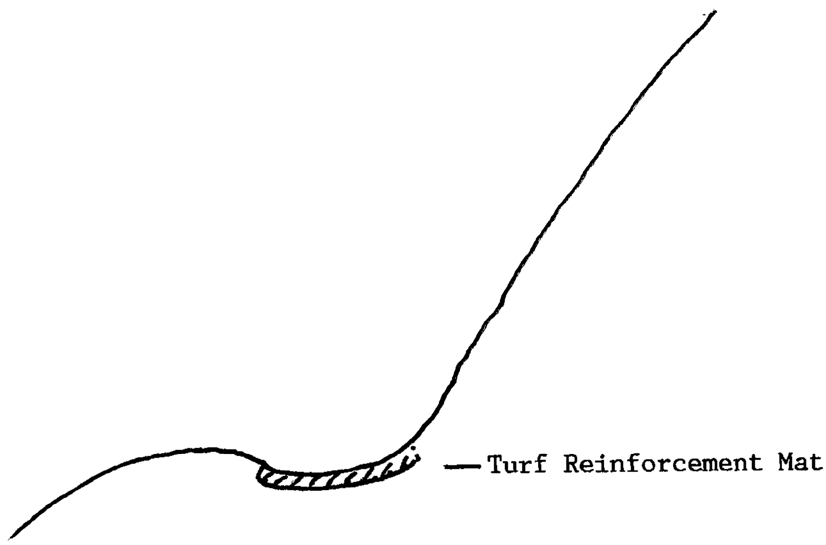

Figure 3. Berm Area Turf Reinforcement Mat Installation

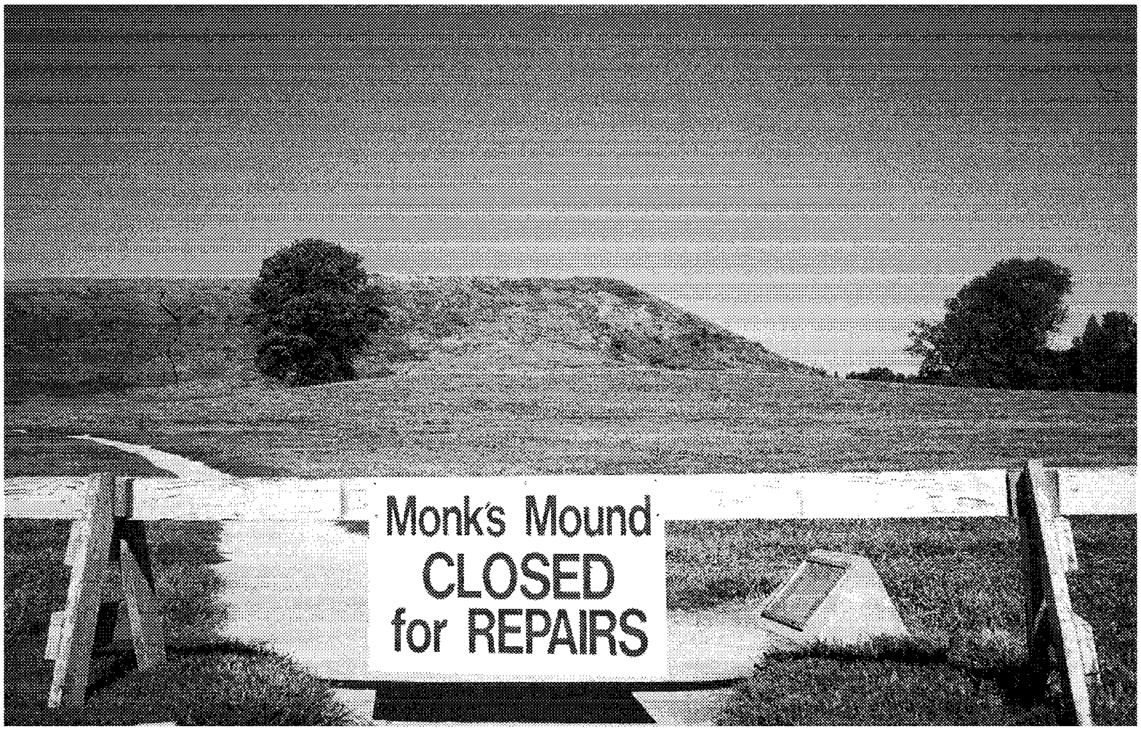

Figure 4. Access to Monk's Mound Suspended

Figure 5. Serious Surface Cracking Prior to Repairs

Figure 6. Earthen Berm Construction

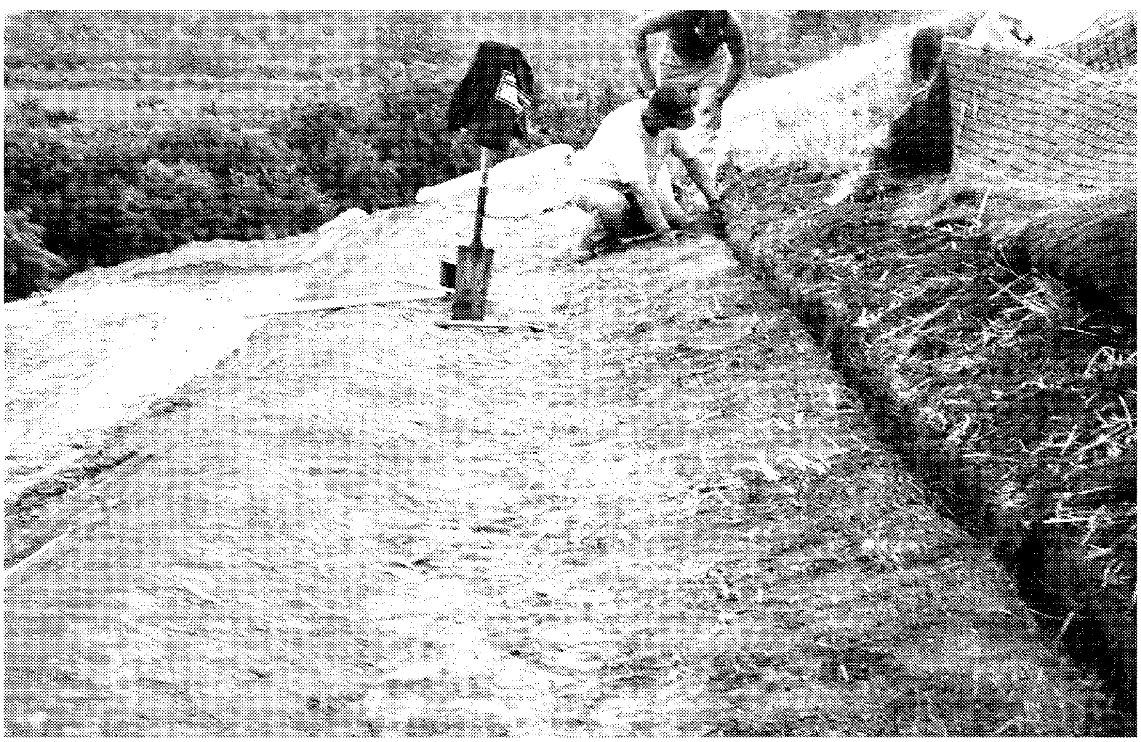

Figure 7. Turf Reinforcement Mat Installation

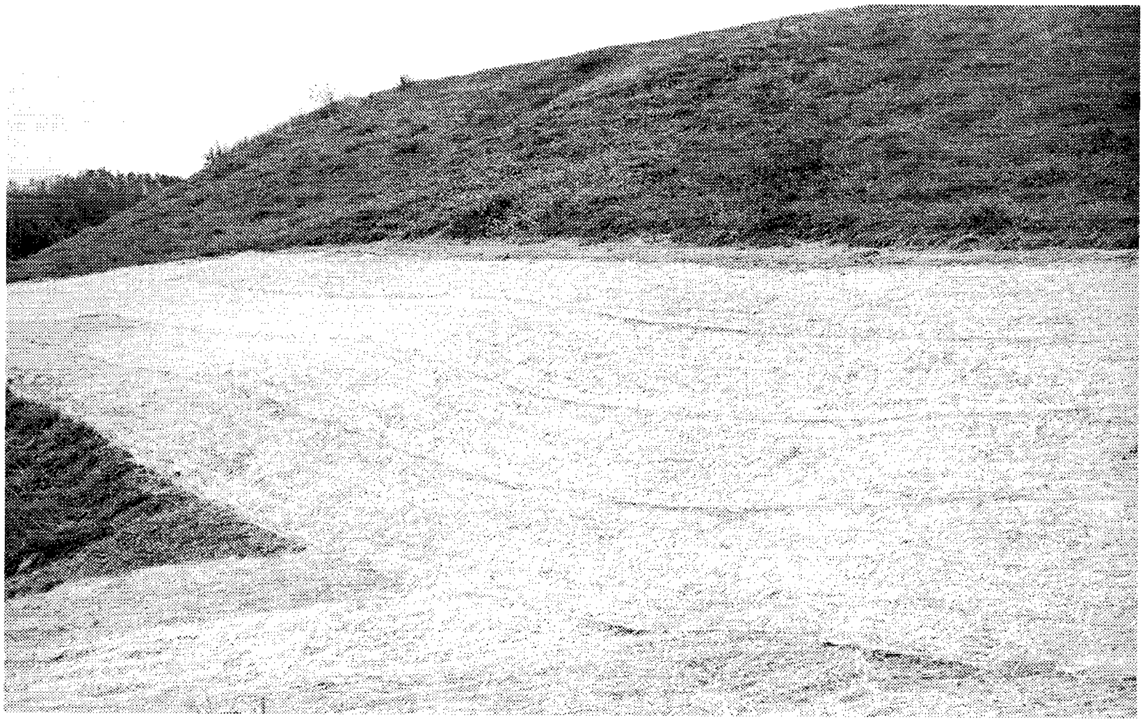

Figure 8. Erosion Control Blanket Installation

Figure 9. Construction of Temporary Roadway

CONTAINMENT OF OILS WITH GEOSYNTHETIC CLAY LINERS USING NATURAL SOIL-MOISTURE FOR BENTONITE HYDRATION

ASHRAF JAHANGIR
TRINTY ENGINEERING TESTING CORP.
DALLAS, TEXAS USA

DAVID E. DANIEL
UNIVERSITY OF ILLINOIS
URBANA, ILLINOIS USA

ABSTRACT

Geosynthetic clay liners (GCLs) are frequently used as a lining material for secondary containment areas that surround storage tanks used to contain petroleum hydrocarbons, chemicals, oils, and other liquids. The hydraulic conductivity of the GCL to these types of liquids depends upon whether the bentonite is hydrated with water at the time that the liquid comes into contact with the GCL. Hydrated bentonite is relatively impermeable to water-immiscible liquids such as petroleum products and oils, but dry bentonite is highly permeable to these types of liquids. This study was conducted to determine if the moisture in subgrade soils can provide sufficient water of hydration to enable the GCL to achieve a low permeability to oils. Tests were performed using two GCLs (a geotextile-encased GCL and a geomembrane-supported GCL) and one liquid (mineral oil). Individual and overlapped GCL panels were placed on moist sand for 8 to 10 weeks, and then were permeated with the oil. Subgrade moisture was drawn into the overlapped portion of the geomembrane-supported GCL, causing the overlap to seal against oil flow. The permeability of the geotextile-encased GCL was sensitive to the thickness of overburden material. It was found in these tests that natural subgrade moisture was adequate to hydrate GCLs and their overlaps sufficiently for the GCL to achieve a low permeability to the oil.

INTRODUCTION

Geosynthetic clay liners (GCLs) are used for a range of sealing applications, including lining of secondary containment areas around liquid storage tanks. In such applications, the GCL is typically buried beneath 150 to 450 mm of soil. GCLs are popular for secondary containment linings because of the low hydraulic conductivity of GCLs, the ease with which the GCL can be installed, and the favorable cost compared with alternative materials.

Secondary containment with GCLs is effective only if the low hydraulic conductivity of the GCL is maintained when the GCL is exposed to the spilled liquid that is to be contained. One concern is the potential for ion exchange to occur if polyvalent cations (e.g., calcium, magnesium, or aluminum) are leached from the cover soil and allowed to permeate through the sodium bentonite in the GCL. Sodium bentonites have very low hydraulic conductivity, but bentonites containing polyvalent cations in their pore waters have significantly higher hydraulic conductivity (e.g., Gleason et al., 1997). For example, Dobras and Elzea (1993) describe a case in which crushed limestone was placed above a GCL that was used as a secondary containment lining around fuel oil storage tanks. Over a period of several years, calcium was leached from the limestone, which caused ion exchange in the bentonite and an increase in hydraulic conductivity of the GCL. The problem was corrected by removing the limestone, applying soda ash to the GCL (which converted the bentonite in the GCL back to a sodium bentonite), and replacing the limestone with a non-calcium-bearing cover material. Other problems with ion exchange have been reported for GCLs covered with soils containing leachable calcium (e.g., James et al., 1997).

A second concern is with regard to the chemical compatibility between bentonite and the liquid to be retained. Petroleum hydrocarbons, oils, and other types of water-immiscible liquids have a very low dielectric constant. Bentonites that are placed in contact with low-dielectric-constant liquids do not swell, as shown, for example, by Leisher (1992). Bentonites only maintain a low hydraulic conductivity in liquids that tend to cause swelling. Daniel et al. (1993) permeated the bentonite component of a GCL with several petroleum hydrocarbons, using different degrees of water hydration prior to permeation with hydrocarbons. Typical results are shown in Fig. 1. It was found that the water-saturated bentonite was essentially impermeable to the petroleum hydrocarbons, and that dry bentonite was highly permeable to hydrocarbon liquids. It appeared that so long as the water content of the bentonite was greater than 50% to 100% at the time that the bentonite was exposed to the hydrocarbon, the GCL maintained a very low hydraulic conductivity for several months of testing employed in the study. Long-term tests were not performed because the study was directed toward short-term retention of spills. Thus, for GCLs to be effective in retaining petroleum hydrocarbons and oils, it appears that the GCL must be adequately hydrated with water at the time that the GCL is challenged to retain the water-immiscible liquid.

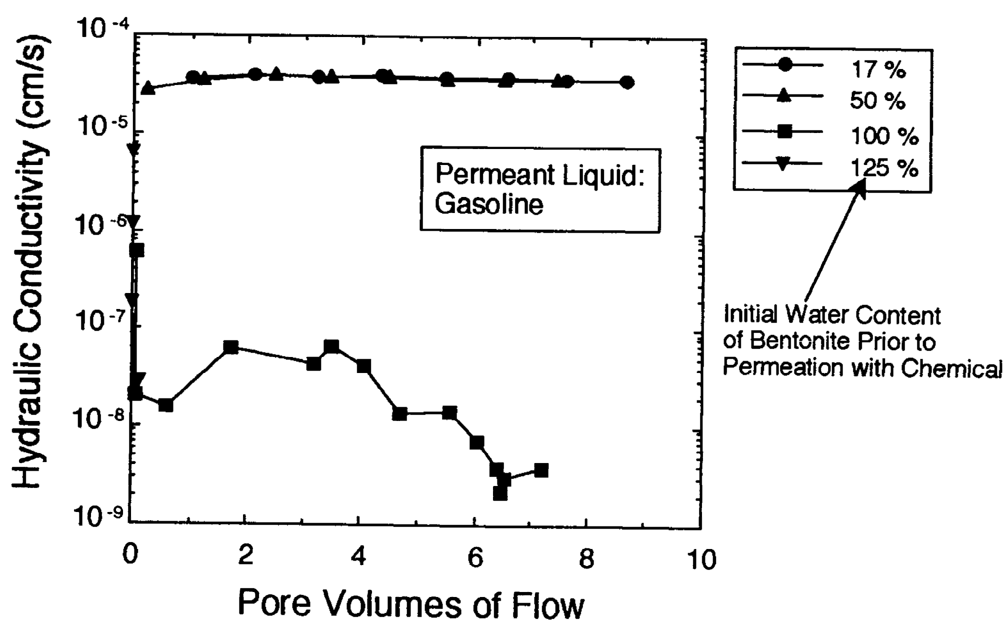

Figure 1. Effect of the Initial Moisture Content of Bentonite on the Hydraulic Conductivity of the Bentonite Component of a GCL to Gasoline (from Daniel et al., 1993).

This investigation was conducted to determine if GCLs hydrate enough to achieve a low hydraulic conductivity to water-immiscible liquids by absorbing natural moisture from subgrade soil. Dry bentonite has an extremely low water potential (high suction), and will tend to draw water from nearly all subgrade soils (Daniel et al., 1993). For the tests described herein, GCLs were placed on top of moist sand, allowed to suck moisture from underlying sand for several weeks, and then were permeated with oil. The hypothesis was that the GCL would be adequately hydrated from natural subgrade moisture to maintain a low hydraulic conductivity to hydrocarbons, even in overlapped panels. If this is the case, then the GCL used for short-term retention of water-immiscible liquids need only be placed on adequately moist subgrade soils and protected from excessive desiccation in order be sufficiently hydrated for hydrocarbon retention (e.g., Geoservices, Inc., 1989). Long-term retention involves a number of other issues that go beyond the scope of this study.

TESTING PROGRAM

Materials

Because of the long period of time required for each test (typically 8 months), the investigation was restricted to two GCLs: a geotextile-encased GCL (Claymax® 500SP) and a geomembrane-supported GCL (Gundseal®). These two GCLs were selected because the geomembrane-supported GCL should be the most difficult to hydrate in overlap areas (the geomembrane component blocks moisture absorption), and because this particular geotextile-encased GCL tends to be the most permeable type of GCL at the low overburden stress that typically exists in secondary containment applications. It was believed that if these two GCLs hydrate adequately, the others likely will hydrate adequately, as well.

Both GCLs contained the standard grade of bentonite for the product. The initial water content of both GCLs was approximately 20%. The geomembrane-supported GCL was tested with the geomembrane component facing upward. The geotextile-encased GCL was installed with the side containing the manufacturer's printed information facing upward (the two geotextiles on the upper and lower surfaces appeared to be identical).

The water used for permeation and for moistening the subgrade soil was ground water pumped from a well. Several hydrocarbon liquids were considered for use, but the overriding concern was laboratory safety. Large quantities of the liquid were needed; volatility and flammability of liquids were major concerns. Mineral oil was selected because it is safe, non-water-soluble, and produces the same permeability as petroleum hydrocarbons (confirmed with comparative tests using gasoline and mineral oil). Neither mineral oil nor petroleum hydrocarbons produce swelling of bentonite. The mineral oil had a density of 862 kg/m^3, a dynamic viscosity of 93 centipoise, and an interfacial tension with tap water of 0.047 N/m, all at 25°C (Jahangir, 1994).

The subgrade soil upon which the GCLs were placed consisted of a coarse, clean sand (D_{10}

= 0.5 mm, D_{50} = 0.85 mm) with less than 1 percent finer than 0.075 mm. The GCLs were covered with washed and screened quartz gravel having an approximate particle size of 10 mm.

Equipment and Procedures

Bench-scale hydraulic conductivity tests were carried out in stainless steel tanks (Figs. 2 and 3). The tanks were designed to accommodate intact and overlapped GCL panels, with an overlap width of 150 mm (typically, the minimum used in the field). The GCL panels were cut slightly larger than 600 mm by 600 mm to provide material for sealing in the flange area. Flanges between the upper and lower halves of the cells eliminated any possibility of sidewall leakage affecting the rate of flow measured in the lower half of the tank. A silicone edge sealant was used to seal the flanges (Fig. 4) and worked very well. Further information is provided by Jahangir (1994).

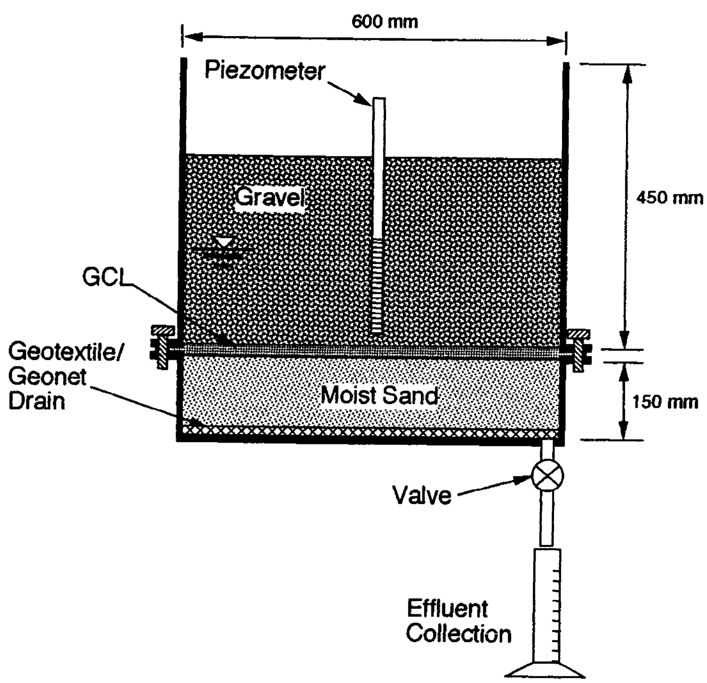

Figure 2. Cross-Section of Stainless Steel Tanks Used to Test GCLs on Subgrade Soil.

Tests were performed on both intact and overlapped panels of the geotextile-encased GCL. Only overlapped panels of the geomembrane-supported GCL were tested because it was pointless to attempt to permeate an intact panel (the geomembrane component would obviously result in no flow through the intact panel). Thus, for the geomembrane-supported GCL, the tests were designed to challenge the overlap area, where the bentonite should seal the overlap. The overlap width was 150 mm for all overlaps of both types of GCLs.

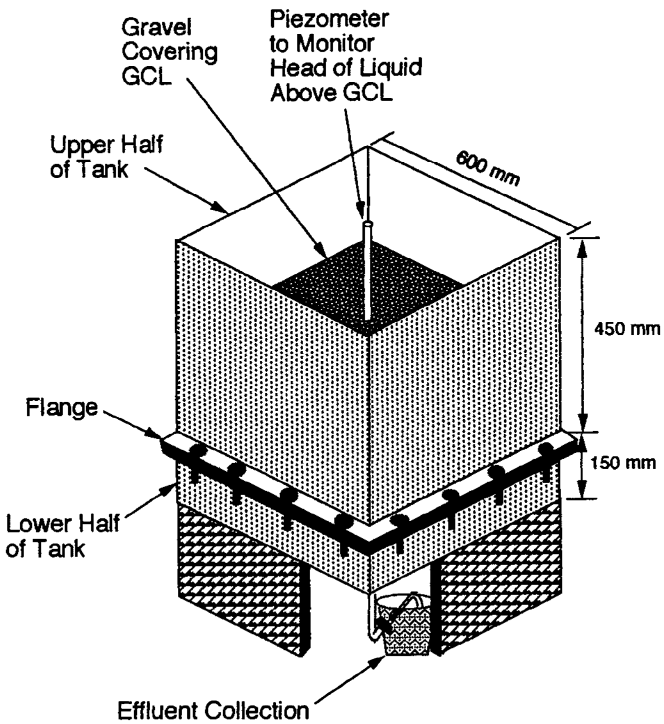

Figure 3. Set Up of Tank Assembly.

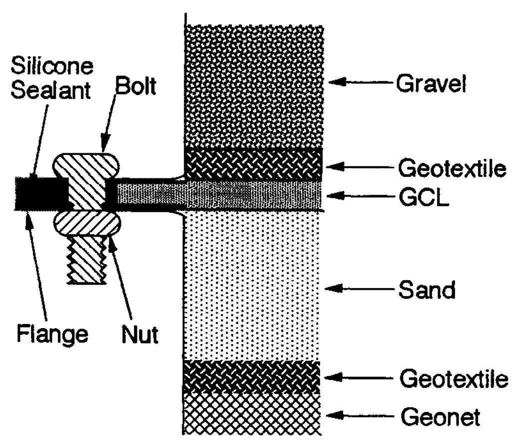

Figure 4. Edge Seal at Flange of Stainless Steel Tanks.

Three types of permeation procedures were used. First, for control purposes, GCL panels (placed on dry sand) were permeated directly with water. In addition, the dry GCLs (placed on dry sand) were permeated directly with mineral oil to confirm that dry GCL panels would be highly permeable to the oil. Finally, GCL panels were placed on moist sand and kept there for 8 to 10 weeks under a gravel overburden. The tanks were covered to prevent evaporation of water. Then mineral oil was introduced into the tanks and allowed to permeate the partially hydrated GCLs. For all tests on partly hydrated GCLs, two identical tanks were set up at the same time. At the end of the 8 to 10 week hydration period, one of the tanks was dismantled so that the water content of the GCL could be determines, and oil was introduced into the second tank.

The moist sand upon which GCLs were placed was moistened to the desired water content, placed inside the bottom part of the tank, and compacted. The moisture content of the sand varied from 4.5% to 8%. It was found that the degree of hydration of the GCLs was not significantly affected by the water content of the sand, within the range used. In all the cases, the water content of the bentonite was greater than 100% after natural hydration from the underlying sand. A valve at the bottom of the tank was kept open to maintain atmospheric pressure in the sand.

For each test involving a partially hydrated GCL, two tanks were set up under identical conditions. After 8 to 10 weeks of hydration, one of the tanks was disassembled. The thickness of the GCL after swelling was measured. Water content samples were taken at various locations.

The thickness of gravel placed on top of the GCL was 150 mm to simulate the minimum thickness typically used in field applications. However, for the geotextile-encased GCL, additional tests were performed with 300 mm and with 450 mm of overburden material to evaluate the influence of thickness of overburden material. The hydraulic gradient was 150 to 300 for the geotextile-encased GCLs (permeation across the entire area) and 15 to 20 for the overlap in the geomembrane supported GCL (permeation only in the overlap zone).

The GCLs were permeated for about 8 months. No attempt was made to achieve full breakthrough of the mineral oil through the GCLs (e.g., per criteria of Daniel, 1994) because to do so might have required several years of testing. The tests were intended to represent relatively short-term (several months) permeation periods.

Additional details about the testing equipment and procedures are given by Jahangir (1994).

Hydraulic Conductivity

Hydraulic conductivity was determined using the falling-head method. The actual measured thickness was used. For the geotextile-encased GCL, the thickness included the thickness of the two thin, woven geotextiles. For the overlapped panels of the geotextile-encased GCL, an average thickness was used because about one-fourth of the area permeated was two panels thick, creating a greater thickness in the overlapped area. For the overlapped geomembrane-supported GCL, flow only took place through the bentonite in the overlap because of the geomembrane component.. The hydraulic conductivity was computed based on the volume of bentonite in the overlap area only. Thus, the hydraulic conductivity of the geomembrane-supported GCL is not the overall conductivity (just the value in the overlap area). This approach was taken because the essential issue under investigation for the geomembrane-supported GCL was whether the overlaps self-seal from natural subgrade moisture.

Hydraulic conductivity was converted to intrinsic permeability using the equation:

$$K = k\,(\mu/\rho g) \tag{1}$$

where K is the intrinsic permeability, k is hydraulic conductivity, μ is the dynamic viscosity of the permeant liquid, ρ is the density of the permeant liquid, and g is the acceleration due to gravity. Intrinsic permeability as calculated to isolate the response of the bentonite. The hydraulic conductivity of a GCL is a function of the density and viscosity of the permeating liquid, but the intrinsic permeability is a function only of the properties of the bentonite.

Tests Performed

Geomembrane-Supported GCL. Three tests were performed on overlapped panels for the geomembrane-supported GCL. In all three tests, 150 mm of overburden gravel was used. In two tests, the subgrade was dry sand, and the permeant liquid was water in one test and oil in the second test. For the third test, the GCL was placed on moist sand for 9 weeks, allowed to hydrate naturally, and then permeated with oil.

Geotextile-Encased GCL. For the geotextile-encased GCL, tests were performed as indicated in Table 1.

Table 1. Bench-Scale Hydraulic Conductivity Tests on Geotextile-Encased GCL.

Thickness of Overburden Gravel (mm)	Intact GCL Panel			Overlapped GCL Panels
	Air-dry GCL	Air-dry GCL	Moist GCL	Moist GCL
	Water	Mineral Oil	Mineral Oil	Mineral Oil
150	x	x	x	x
300	x		x	
450	x		x	x

For the GCL panels permeated with mineral oil under an overburden of 450 mm, water was poured in after the hydraulic conductivity test with the mineral oil was completed. Then, hydraulic conductivity of the GCL panels to water was monitored. Once the GCLs exhibited a steady hydraulic conductivity to water, they were again permeated with oil.

Tests with Flexible-Wall Permeameters

Additional tests were performed on 100-mm-diameter samples using a flexible-wall permeameter but no back-pressure. Only air-dry or water-saturated specimens were tested. With the geomembrane-supported GCL, the geomembrane component was removed so that only the bentonite component was tested.

RESULTS

Flexible-Wall Permeameters

The results of tests performed using flexible-wall permeameters are summarized in Table 2. Hydraulic conductivity of air-dry GCLs permeated directly with mineral oil was found to be high (> 1 x 10^{-6} cm/s). The precise value of hydraulic conductivity could not be determined because the flow rate was limited by head loss in valves, fittings, and tubing in the flexible-wall cell. An increase of effective stress from 12 kPa to 138 kPa did not cause any change in the hydraulic conductivity of dry samples permeated directly with mineral oil. The GCL did not swell at all when exposed to mineral oil. The results for mineral are essentially identical to those for petroleum hydrocarbons reported by Daniel et al. (1993).

Table 2 Summary of the Results of Flexible-Wall Hydraulic Conductivity Tests.

Permeant Liquid:	Mineral Oil		Water		Mineral Oil	
Hydration:	Air-Dry		Air-Dry		Fully Hydrated	
Effective Stress:	12 to 138 kPa		16 kPa		17 kPa	
	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)
Geotextile-Encased GCL	> 10^{-6}	> 10^{-14}	4 x 10^{-9}	4 x 10^{-18}	3 x 10^{-9}	3 x 10^{-17}
Geomembrane-Supported GCL	> 10^{-6}	> 10^{-14}	2 x 10^{-9}	2 x 10^{-18}	-	-

After 1.0 pore volume of water had passed through a sample of the geotextile-encased GCL, mineral oil was introduced. The hydraulic conductivity to mineral oil was 3 x 10^{-9} cm/s, compared to a hydraulic conductivity of 4 x 10^{-9} cm/s to water. At the end of the test, the effluent was pure

mineral oil, indicating full breakthrough of the mineral oil in the effluent liquid. Although the hydraulic conductivities for water and mineral oil were nearly identical, the intrinsic permeability to mineral oil was an order of magnitude higher than the intrinsic permeability to water. Thus, the mineral oil did cause some increase in intrinsic permeability of the water-saturated GCL.

The sample that was permeated first with water and then with mineral oil was examined at the completion of the test. The amount of water collected in the effluent (after mineral oil was introduced) was much less than the initial volume of water in the GCL and porous disks. Also, there was no change in the thickness of the GCL when the mineral oil was introduced. In addition, at the completion of the test, the water content (dry weight basis) of the GCL was 177%, and the oil content (again, dry weight basis) was only 2%. The mineral oil likely penetrated only the largest pores in the bentonite; very little water in the sample was displaced by the mineral oil. This is similar to the observations made by Fernandez and Quigley (1985) for clay soils in general, in which it was demonstrated that the clay is highly hydrophilic and highly oil-phobic (i.e., the clay tends to retain water but not oil).

Bench-Scale Hydraulic Conductivity Test Results

Geomembrane-Supported GCL. Test results are summarized in Table 3. The reported hydraulic conductivity is based on flow through the bentonite in the overlapped area only.

Table 3. Summary of Bench-Scale Tests on Geomembrane-Supported GCL.

Permeant Liquid:	Mineral Oil		Water		Mineral Oil	
Hydration:	Air-Dry		Air-Dry		Partially Hydrated	
Effective Stress:	2.0 kPa		1.6 kPa		1.9 kPa	
	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)	Hydr. Cond. (cm/s)	Intrinsic Perm. (m^2)
	1×10^{-3}	3×10^{-11}	No flow	No flow	0 to 2×10^{-5}	0 to 3×10^{-11}

For the air-dry, overlapped GCL panels placed on dry sand and permeated with mineral oil, the hydraulic conductivity was found to be 1×10^{-3} cm/s. The gravel overburden was 150 mm thick, which produced an effective vertical stress of 2.0 kPa. The bentonite, as expected, did not swell; rather, there were signs of small cracks forming in it. The flowing mineral oil apparently washed out the adhesives that held the bentonite to the HDPE sheet. After the test, the HDPE sheet came off the bentonite layer.

The hydraulic conductivity calculations were based on the assumption that all the flow occurred through the bentonite in the overlap. However, for regulatory purposes, the hydraulic conductivity is evaluated on the assumption of flow across the entire area of the GCL. If a 4-mm-thick panel of 5.3 m width is placed with an overlap of 150 mm under similar conditions, then the hydraulic conductivity for the regulatory purpose would be much lower. Based on the hydraulic conductivity to the mineral oil for the dry GCL determined from the experiments and a hydraulic head of 1 m, the overall or equivalent hydraulic conductivity would be 2×10^{-8} cm/s. Doubling the overlap width to 300 mm would cut this equivalent hydraulic conductivity in half to 1×10^{-8} cm/s.

When the geomembrane-supported GCL with a 150 mm overlap was placed on dry sand and permeated with water, no flow of water was observed during the two-month-long testing period. The water contents at various locations are shown in Figure 5. Only the portion of the overlap closest to the point of hydration was hydrated. The remainder of the overlap and the other areas of the GCL panels were at a water content of 20%, which was the initial water content of the GCL.

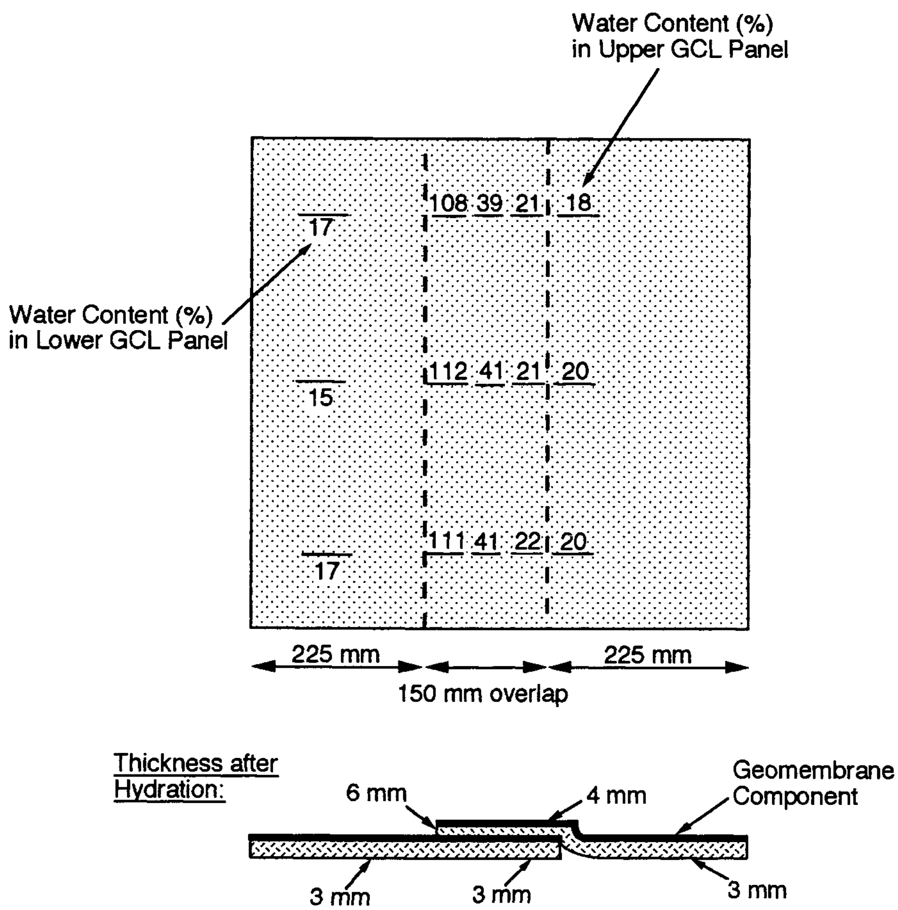

Figure 5. Water Contents in Bentonite Component of Geomembrane-Supported GCL after Two Months of Permation with Water.

When the geomembrane-supported GCL panels with a 150 mm overlap were placed on moist sand with an initial water content of 8% for 9 weeks under 150 mm of gravel (2.4 kPa), the resulting hydration of the bentonite at various locations occurred is shown in Figure 6. The average water content in the bentonite that was in direct contact with the sand was about 93%, and the bentonite swelled to an average thickness of 7 mm from an initial thickness of 3 mm. However, because the flow of mineral oil would only be through the overlap, the water content attained by the bentonite in the overlap is critical. The bentonite in the top layer of the overlapped portion was underlain by the HDPE component of the bottom layer. Nevertheless, the bentonite managed to hydrate to some degree by drawing moisture from moist bentonite beside it. The bentonite in the overlap had a water content of 28 to 69%. Given more time to absorb water, the water contents would have been higher. Also, had some water been introduced at the surface, the overlap would have hydrated laterally from both directions, rather than just one.

After the panels had absorbed subgrade moisture for 9 weeks, mineral oil was introduced and allowed to permeate for 6 months. There was little flow of mineral oil initially, and eventually the flow ceased. The hydraulic conductivity and the intrinsic permeability of the partially hydrated bentonite in the overlap at the end of each month of permeation are shown in Table 4.

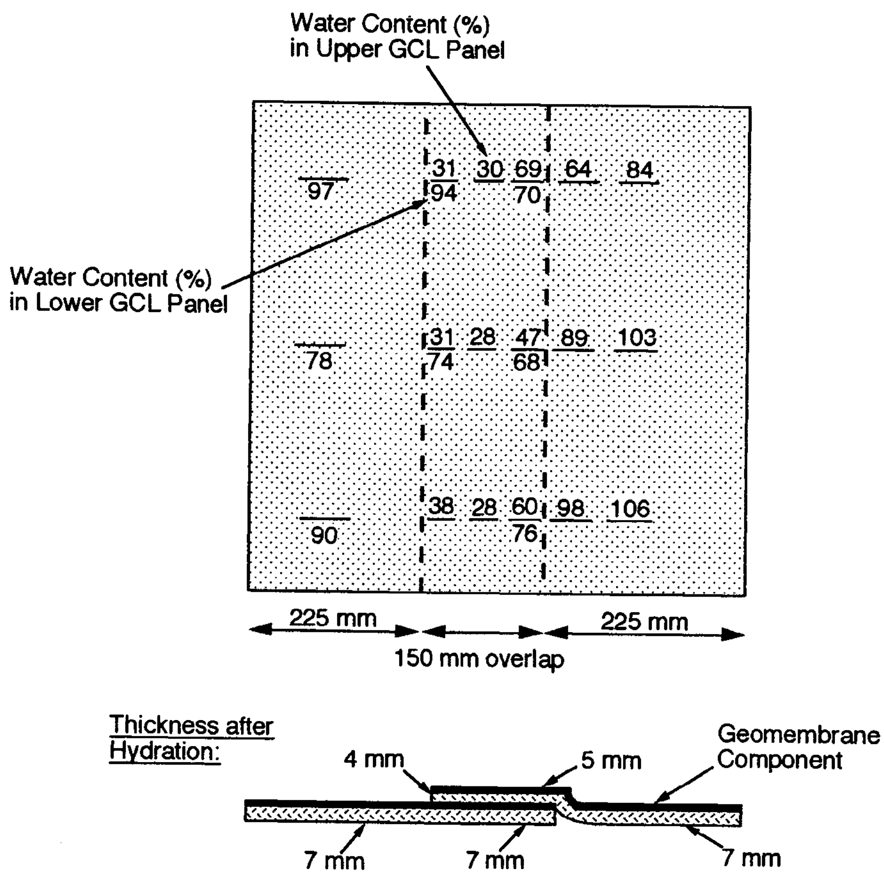

Figure 6. Water Contents in Bentonite Component of Geomembrane-Supported GCL after Nine Weeks of Exposure to Underlying Sand Having a Water Content of 8%.

Table 4. Hydraulic Conductivity Geomembrane-Supported GCL Overlap to Mineral Oil.

Time of Permeation	Hydraulic Conductivity (cm/s)	Intrinsic Permeability (m^2)
1 month	2×10^{-5}	3×10^{-13}
2 months	6×10^{-6}	1×10^{-13}
3 months	4×10^{-6}	9×10^{-14}
4 months	4×10^{-6}	8×10^{-14}
5 months	4×10^{-6}	8×10^{-14}
6 months	No flow	—

It is not known why the flow completely stopped after 6 months. Perhaps the bentonite finally absorbed enough water in the overlap to self-seal (bentonite has a much stronger affinity for water than mineral oil or other non-aqueous-phase liquids).

The hydraulic conductivity's shown in Table 4 are for the overlap area only. For a panel that measures 5.3 m in width and an overlap width of 150 mm, the equivalent hydraulic conductivity of the entire GCL when the hydraulic conductivity of the overlap bentonite is 4×10^{-6} cm/s is approximately 4×10^{-11} cm/s. This value is typically well below regulatory values, thus confirming that the GCL panels should function effectively from hydration associated with subgrade moisture, based on the hydraulic conductivity measured in this investigation. Hydration from above (e.g., from precipitation) would be expected drive the hydraulic conductivity even lower.

Geotextile-Encased GCL

The results of the hydraulic conductivity tests on the geotextile-supported GCL are summarized in Table 5.

Permeation of Dry GCL with Oil. The hydraulic conductivity of the air-dry GCL permeated with mineral oil under an overburden of 150 mm of gravel could not be determined precisely because the flow rate was so high that the head losses in the system governed. It can be only said that the hydraulic conductivity was greater than 2×10^{-4} cm/s and that the intrinsic permeability was greater than 4×10^{-12} m^2.

Permeation of Dry GCL with Water. Hydraulic conductivity to water was determined under overburdens of 150 mm, 300 mm, and 450 mm of gravel, and in a flexible-wall permeameter. The hydraulic conductivity's are plotted versus effective vertical stress in Fig. 7.

Table 5. Summary of Bench-Scale Hydraulic Conductivity Tests on Geotextile-Encased GCL.

Condition	Gravel Thick. (mm)	Effective Stress (kPa)	Water Content (%)	GCL Thickness (mm)	Hydraulic Conductivity (cm/s)	Intrinsic Permeability (m^2)
Mineral Oil Permeating Intact, Air-Dry Panel	150	2.2	20	4	$> 2 \times 10^{-4}$	$> 4 \times 10^{-12}$
Water Permeating Intact, Air-Dry Panels	150	1.8	380	16	5×10^{-8}	5×10^{-17}
	300	3.6	317	14	2×10^{-7}	2×10^{-16}
	300	3.6	230	14	4×10^{-8}	4×10^{-17}
	450	6.3	236	13	2×10^{-8}	2×10^{-17}
Mineral Oil Permeating Intact, Hydrated Panels	150	2.1	147	12	1×10^{-5}	2×10^{-13}
	300	3.6	122	10	1×10^{-5}	3×10^{-13}
	450	6.7	156	9	8×10^{-8}	2×10^{-15}
	450*	6.7	184	12	3×10^{-8}	5×10^{-16}
Mineral Oil Permeating Overlapped Hydrated GCL Panels	150	2.1	90-152	12	3×10^{-5}	5×10^{-13}
	450	6.4	72-118	10	2×10^{-6}	5×10^{-14}
	450*	6.7	174-218	12.25	3×10^{-8}	5×10^{-16}

* Fully hydrated with water prior to permeation with mineral oil

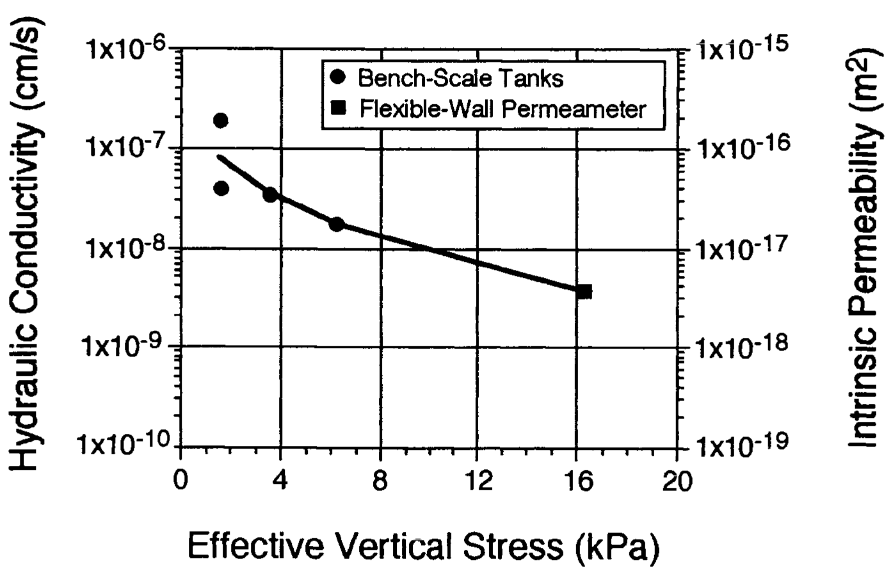

Figure 7. Hydraulic Conductivity of Intact Geotextile-Encased GCL to Water.

Permeation of Partly Hydrated GCL with Oil. Overlapping and intact panels were first hydrated by keeping them on moist sand for 8 to 10 weeks, and then they were permeated with the mineral oil. A typical distribution of water content under an overburden of 150 mm of gravel is shown in Fig. 8.

The intact portion reached a water content well of about 150% . However, the water content in the top layer of the overlapped portion was about 90%. Also, the bottom layer of the overlapped portion had lower water content than the intact portion of the same sample. Bentonite in the top layer of the overlapped portion acquired water by drawing moisture out of the underlying wet bentonite, which, in turn, drew water out of the moist sand below it. Therefore, water contents in both the layers at the overlapped zones were lower than the water contents at the intact portions. Higher hydraulic conductivity to the mineral oil in the overlapping panels may be attributed at least partially to the lower water content in the overlapped portion.

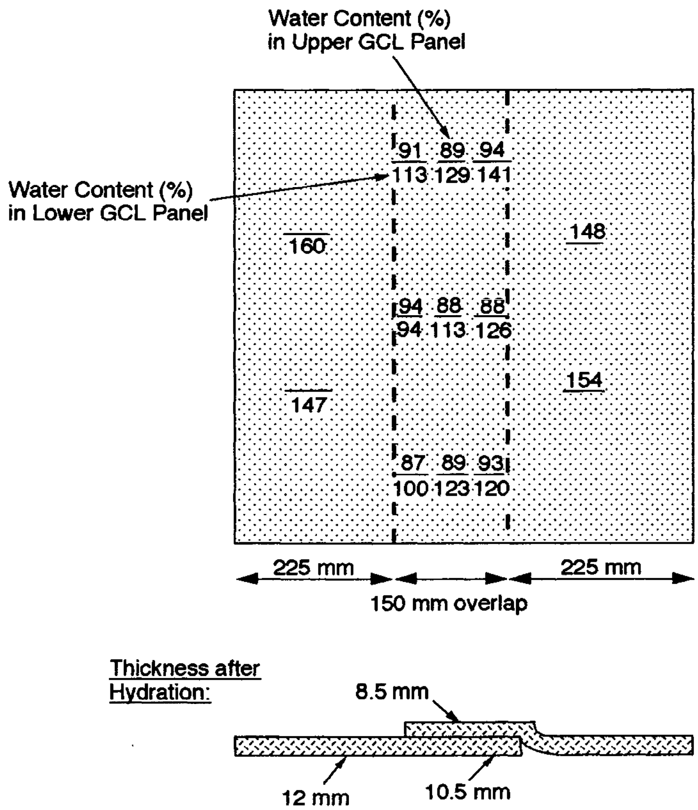

Figure 8. Hydration Pattern of Overlapped Panels of Geotextile-Encased GCL Hydrated for Nine Weeks on Sand with an Initial Water Content of 8%.

In further studies, an intact panel and overlapping panels that had been prehydrated with water and permeated with mineral oil were next permeated with water. Once the GCLs attained a steady hydraulic conductivity to water, they were again permeated with the mineral oil. Intrinsic permeability of theses two samples is plotted in Fig. 9. Hydraulic conductivity to water was the same for the intact and overlapping panels. The initially higher intrinsic permeability to the mineral oil of the overlapping panels disappeared when the panels were fully hydrated by water and permeated again with mineral oil, providing some validation of the assumption that the overlapped panels hydrated from subgrade moisture were more permeable to mineral oil than intact panels because of a lower water content in the overlap area. As with the laboratory tests, the intrinsic permeability of the geotextile-encased GCL was higher to mineral oil than water.

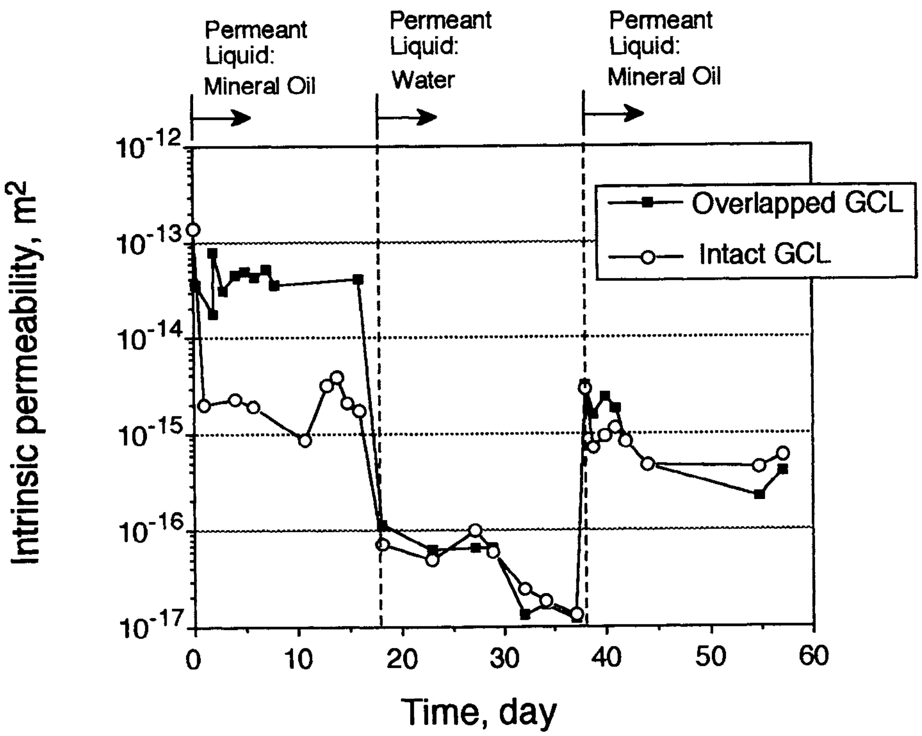

Figure 9. Intrinsic Permeability of Geotextile-Encased GCL that was Hydrated by Subgrade Moisture, Permeated with Oil, then Permeated with Water, and Finally Re-Permeated with Oil.

Effective stress was found to be an important factor affecting the geotextile-encased GCL used in this study. The pronounced effect of effective stress upon intrinsic permeability can be seen in Fig. 10. The water contents for the partially hydrated intact samples at different overburden stresses were not significantly different. But an increase of the overburden from 150 mm of gravel to 450 mm of gravel decreased the intrinsic permeability to the mineral oil by two orders of magnitude.

It should be emphasized that the geotextile-encased GCL used in this study is internally reinforced with stitches spaced 100 mm apart. It is expected that needlepunched GCLs would provide significantly greater confinement to the bentonite than the material used in this study and, therefore, be less sensitive to vertical stress.

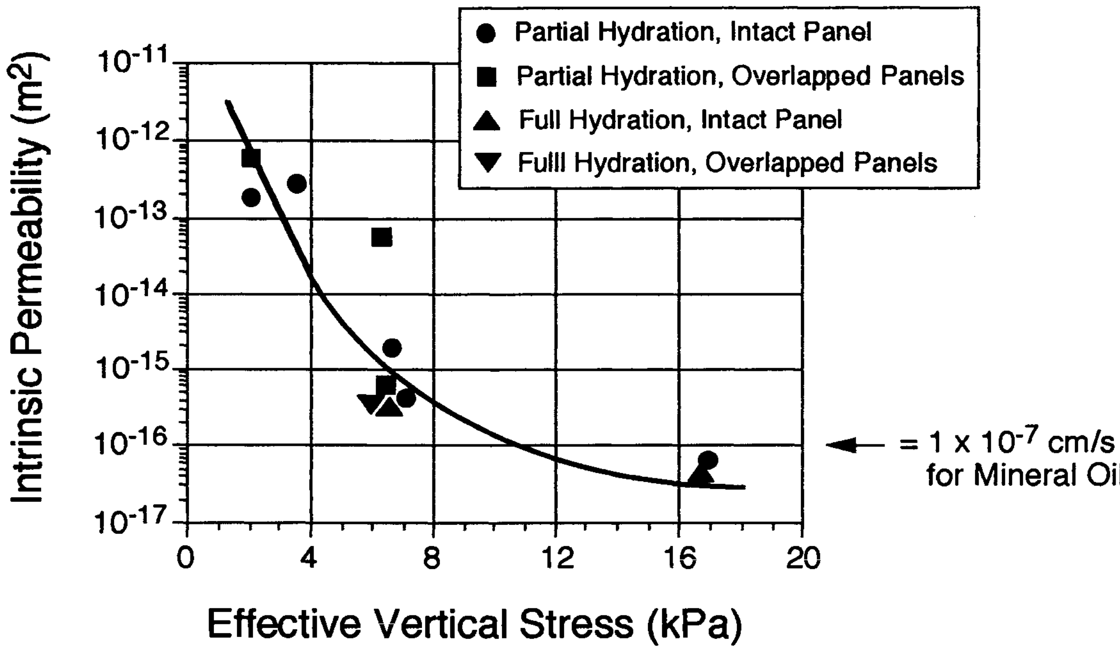

Figure 10. Intrinsic Permeability of Intact and Overlapped Geotextile-Encased GCL to Mineral Oil.

CONCLUSIONS

The intact and overlapped GCLs investigated in this study were placed on moist subgrade soils, covered, and left to absorb subgrade moisture for 8 to 10 weeks. The bentonite component of the GCLs attained moisture contents of approximately 80% to 100% (geomembrane-supported GCL) and 120% to 150% (geotextile-encased GCL). The GCLs were then permeated with mineral oil. Flow through the overlapped panels of the geomembrane-supported GCL occurred only at the overlap, and the hydraulic conductivity was found to be about 4 x 10^{-6} cm/s. When the average hydraulic conductivity of full-size panels is calculated assuming this hydraulic conductivity for overlaps and zero hydraulic conductivity for intact sections (due to geomembrane component), the overall hydraulic conductivity was found to be approximately 4 x 10^{-11} cm/s. For the geotextile-encased GCL used in this study, the hydraulic conductivity to mineral oil was found to be sensitive to overburden pressure, decreasing from a value of 1 x 10^{-5} cm/s for 150 mm of overburden soil to a value of 8 x 10^{-8} cm/s for 450 mm of overburden soil. These experiments demonstrate that GCLs do absorb moisture from subgrade soils, and that this moisture is beneficial to the GCL in achieving a relatively low hydraulic conductivity to oil. Of course, the GCL must be adequately protected from desiccation or chemical alterations caused by ion exchange. The experiments were short-term tests lasting a few months; long-term effects were not evaluated.

ACKNOWLEDGMENT

Funding for this project was provided by the Texas Advanced Technology Program.

REFERENCES

Daniel, D.E. (1994), "State-of-the-Art: Laboratory Hydraulic Conductivity Tests for Saturated Soils," *Hydraulic Conductivity and Waste Contaminant Transport in Soil, ASTM STP 1142*, D.E. Daniel and S.J. Trautwein (Eds.), American Society for Testing and Materials, Philadelphia, pp. 30-78.

Daniel D.E., H.-Y. Shan, and J. D. Anderson (1993), "Effects of Partial Wetting on the Performance of the Bentonite Component of a Geosynthetic Clay Liner", *Geosynthetics '93*, Industrial Fabrics Association International, Vancouver, Canada, Vol. 3, pp. 1483-1496.

Dobras, T.N., and Elzea, J.M. (1993), "In-Situ Soda Ash Treatment for Contaminated Geosynthetic Clay Liners," *Geosynthetics '93*, Vol. 3, Industrial Fabrics Association International, St. Paul, Minnesota, pp. 1145-1160.

Fernandez, F and R. M. Quigley (1985), "Hydraulic Conductivity of Natural Clays Permeated with Simple Liquid Hydrocarbons", *Canadian Geotechnical Journal*, Vol. 22, pp. 205-214.

Geoservices, Inc. (1989), "Report of Moisture Retention Tests, Claymax CR," Norcross, Georgia.

Gleason, M.H., Daniel, D.E., and Eykholt, G.R. (1997), "Calcium and Sodium Bentonite for Hydraulic Containment Applications," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 5, pp. 438-445.

Jahangir, M.A. (1994), "Containment of Petroleum Hydrocarbons by Geosynthetic Clay Liners Using Natural Soil-Moisture", Master of Science Thesis, The University of Texas at Austin.

James, A.N., Fullerton, D., and Drake, R. (1997), "Field Performance of GCL under Ion Exchange Conditions," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 10, pp. 897-902.

Leisher, P.J. (1992), "Hydration and Shear Strength Behavior of Geosynthetic Clay Liners," M.S. Thesis, Drexel University, Philadelphia.

GAS DIFFUSION THROUGH A GCL

MICHEL AUBERTIN
ÉCOLE POLYTECHNIQUE DE MONTRÉAL, QC, CANADA

KARINE AUTHIER
ÉCOLE POLYTECHNIQUE DE MONTRÉAL, QC, CANADA

ANNE-MARIE JOANES
ÉCOLE POLYTECHNIQUE DE MONTRÉAL, QC, CANADA

MONICA MONZON
ÉCOLE POLYTECHNIQUE DE MONTRÉAL, QC, CANADA

MOSTAFA AACHIB
ÉCOLE POLYTECHNIQUE DE MONTRÉAL, QC, CANADA

ABSTRACT

Geosynthetic clay liners (GCLs) are often considered as part of the closure scheme for waste disposal sites. When used in covers, GCLs mainly serve as a infiltration barrier to reduce water inflow. In many situations however, covers must also insure that gas migration from or into the disposal site be limited to acceptable levels. This is the case for instance with municipal waste dumps where bio-gas must not escape from the site, and with acid generating mine tailings which require a control of oxygen availability to the sulphidic waste. In this paper, the authors first review the basic principles of diffusive gas transport in porous media. Then, a new experimental procedure to measure gas flux through a GCL is presented. Laboratory test results obtained on a GCL at various water contents are also presented and compared to predictions made from a diffusion model. Finally, some sample calculations are done to evaluate effectiveness of cover barriers made with a GCL and with fine grained soils.

KEYWORDS : GCL, diffusion, testing, covers, gas.

INTRODUCTION

Closure and reclamation of waste disposal sites often involve the design and construction of cover systems aimed in part at controlling the transport of solids and fluids in and out of the facilities. Various materials, used alone or in combination, have been considered for ensuring the efficiency of cover systems, including different types of soils, geomembranes, cement products, bitumen, and waste materials (e.g. Senes, 1994; Aubertin et al.,1995, 1996 ; Koerner and Daniel, 1997). In most instances, at least one of the selected materials has a low hydraulic conductivity to limit the amount of water infiltration through the cover. Over the past few years, geosynthetic clay liners (GCLs) have received a lot of attention from designers and researchers alike, for their potential use in cover and liner systems. This relatively new type of geocomposite is often made from thin layers of bentonite placed between two attached woven or nonwoven geotextiles. In this case, the bentonite provides the desired hydraulic properties, while the geotextiles main purpose is mechanical stability of the thin geocomposite. GCLs represent an interesting alternative to other natural or man made materials used in confining works for wastes from industrial, domestic or mining origin. A general overview of GCLs characteristics can be found in Koerner (1994, 1996), and Koerner et al. (1995), with more information available in the cited references thereupon.

As their use broadens, GCLs are being investigated intensively, especially in regard to their hydraulic characteristics (e.g. Estornell and Daniel, 1992 ; Bouazza et al. , 1996; Takahashi et al., 1996; Daniel, 1996; Lake et al., 1997; Petrov and Rowe, 1997) and to their mechanical behavior (e.g. Koerner and Narejo, 1995; Frobel, 1996; Fox et al., 1996; Gilbert et al., 1996; Richardson, 1997; Daniel et al., 1998).

So far however, little attention has been paid to the capabilities of GCLs to control gas flux. This is nevertheless an important issue for many types of waste, such as for bio-gas emitted from municipal waste dumps (e.g. BRGM, 1994, 1995) or for oxygen flow from the atmosphere to acid producing sulphidic tailings (Nicholson et al., 1989 ; Collin and Rasmuson, 1990 ; Aubertin et al., 1993, 1995, 1996). The question of GCLs efficiency to control gas flux has recently been raised because of its use in actual mine reclamation projects (e.g. Bienvenu, 1998).

Few relevant studies are available; gas permeability of GCLs has been investigated by Naue-Fasertechnik (1992), while Koerner and Allen (1997) have reported on the difficulties related to measurement and interpretation of water vapor transmission through geosynthetics. Beyond a general lack of available information on gas flux, there is an acute paucity of research dealing with the diffusion transport mechanism. This is creating problems for designers because of the absence of available data on gas diffusion for GCLs.

In this paper, the authors briefly review the theory on one dimensional diffusive movement for gas in media with various degree of saturation and water content. A proposed laboratory test apparatus and methodology developed for this study on GCLs is then described. Preliminary test results for oxygen diffusion are also presented and discussed. The relevance of these measurements is illustrated through some sample calculations for gas diffusive flux through typical cover systems.

GAS DIFFUSION

Diffusion is a process controlled by concentration (or partial pressure) gradient which induce ionic or molecular movement from regions of higher concentration to regions of lower concentration. Diffusion is a well-known transport process for contaminants in fine grained soils used in waste containment facilities, as it may add to the mass flux of solute due to advection through earthen barriers (e.g. Desaulniers et al., 1984 ; Gillham et al., 1984 ; Rowe, 1987 ; Daniel and Shackelford, 1988). Physical testing and constitutive equations for such diffusion phenomena have been reviewed by Rowe et al. (1988, 1995) and by Shackelford (1991). Recently, Lake at al.(1997) have proposed a methodology to evaluate diffusion of solute through GCL.

Diffusion may also be a dominant transport process for gas and water vapor through the air and water phase of porous media since they are conductive for diffusion processes. The pore structure and fluid distribution then determine the path and cross-sectional area available for transport (e.g. Rolston, 1986, Collin et Rasmuson 1988 ; Fredlund and Rahardjo, 1993).

Diffusion flux is usually defined from Fick's equation initially developed for heat transfer (Crank, 1975 ; Freeze and Cherry, 1979). For unidimensional conditions, the mass flux is given by Fick's first law :

$$F_g = -D_e \frac{\partial C}{\partial z} \tag{1}$$

where F_g is the diffusive gas through a unit area (M/L^2T), D_e is the effective diffusion coefficient (L^2/T), C is the molecular concentration of the diffusive constituents (O_2, CO_2, N, etc) in the gas phase (M/L^3), and z is the distance between the points of interest (L). Although the flux can also be expressed as a function of the partial pressure gradient (e.g. Fredlund and Rahardjo, 1993), description will only be given here in terms of concentration ; the minus (-) sign in Equation (1) means that mass transfer over time occurs in the opposite direction of the latter. This equation can be generalized for multidimensionnal flow (Crank, 1975), but Equation (1) suffice for our purpose.

Equation (1) implies that for a constant D_e, there is a linear relationship between the flux and the concentration gradient between 2 points. It is thus mathematically similar to Darcy's law for saturated seepage with D_e playing the role of hydraulic conductivity and controlling the ease with which gas flow occurs.

During transient conditions, concentration may change over time and location. Continuity implies that concentration variation in time must be balanced by the flux change with distance. This gave rise to the second Fick's law, which can be expressed as follows for unidirectional conditions (Freeze and Cherry, 1979) :

$$\frac{\partial C}{\partial t} = D_e \frac{\partial^2 C}{\partial z^2} \tag{2}$$

Again, D_e is the parameter that controls the rate of concentration variation in the media, which in turn controls the mass flux. In practice, the value of D_e is not necessarily a constant, as it depends on the pores and fluid characteristics (i.e., total porosity, tortuosity, degree of saturation, molecular weight, etc).

Diffusive flux is much easier through the air phase, but it can also occur through the water phase (it is usually considered that diffusion does not occur through solid phase). As there are almost 4 orders of magnitude difference between the value of the diffusion coefficient in air ($D_a^\circ \cong 1.8 \times 10^{-5} m^2/s$ for O_2) and in water ($D_w^\circ \cong 2.5 \times 10^{-9}$ m^2/s for O_2), gas diffusion flux through water becomes appreciable only when the degree of saturation S_r is large (S_r above 90% to 95% ; e.g. Aubertin et al., 1995). Hence, at low S_r, water may be considered an obstacle like the solid, and D_e is sometimes replaced by $\theta_a D_a^*$, where θ_a is the air filled porosity ($\theta_a = n(1 - S_r)$, where n is the porosity) and $D_a^* = D_e / \theta_a$. At higher degree of saturation, D_e becomes :

$$D_e = D_a + D_w \tag{3a}$$

or

$$D_e = \theta_a D_a^* + \theta_w D_w^* \tag{3b}$$

where D_a^* and D_w^* represent diffusion through air and water respectively ; θ_w is the water filled porosity ($\theta_w = n - \theta_a$) also called volumetric water content.

Equation 3 shows that the value of D_e can be decomposed into the air and water components. Each of these components can be described by an appropriate function such as:

$$D_a = D_a^\circ a(1 - S_r)^b \tag{4}$$

$$D_w = H D_w^\circ a S_r \tag{5}$$

where a and b are model parameters, and H is the gas relative solubility constant ($H \approx 0.03$ for O_2 at 25°C). The values of a and b can be obtained from fitting the parameters to experimental data ; in this case, a and b are usually found to be in the range of 0.27±0.08 and 3.3±0.5 respectively (Eberling et al., 1994). Alternatively, Collin and Rasmuson (1988), who have modified the model from Millington and Shearer (1971), have proposed to define a and b from explicit functions of S_r and n. In this case, one obtains for equations 4 and 5 :

$$D_a = D_a^{\circ}(1-S_r)^2\left[n(1-S_r)\right]^{2x} \tag{6a}$$

or

$$D_a = D_a^{\circ}\frac{1}{n^2}\theta_a^{\;2(1+x)} \tag{6b}$$

$$D_w = HD_w^{\circ}S_r^{\;2}(nS_r)^{2y} \tag{7a}$$

or

$$D_w = HD_w^{\circ}\frac{1}{n^2}\theta_w^{\;2(1+y)} \tag{7b}$$

The value of exponents x and y are retrieved from :

$$\left[n(1-S_r)\right]^{2x} + \left[1-n(S_r)\right]^{x} = 1 \tag{8}$$

$$(nS_r)^{2y} + (1-nS_r)^{y} = 1 \tag{9}$$

For soils, x and y are typically between 0.5 and 0.75. Equations 6 to 9 have been successfully used previously to predict the value of D_e of partly saturated soils and mine tailings (Aachib et al., 1993 ; Aubertin et al., 1995, 1997). These equations will be used in the next section to compare measured and predicted values.

To solve Fick's laws, one can use analytical solutions for specific limiting conditions (e.g. Crank, 1975 ; Eberling et al., 1994) or numerical integration for more general applications (e.g. Rowe et al., 1994). Both types of solution will be used in the followings. But before presenting calculation results, the experimental procedure proposed to evaluate the value of D_e is first introduced.

DIFFUSION TESTING

The experimental evaluation of D_e was done here for oxygen, but the procedure can easily be adapted for other gas constituents. Tests were performed in diffusion cell consisting of a source reservoir, the porous medium through which flux takes place, and a receptor reservoir. The selected procedure involves transient state in a closed system with source concentration decreasing over time while that of the receptor increases proportionally with diffusive flux (e.g. Shackelford, 1991). Figure 1 shows the experimental setup (Authier, 1997).

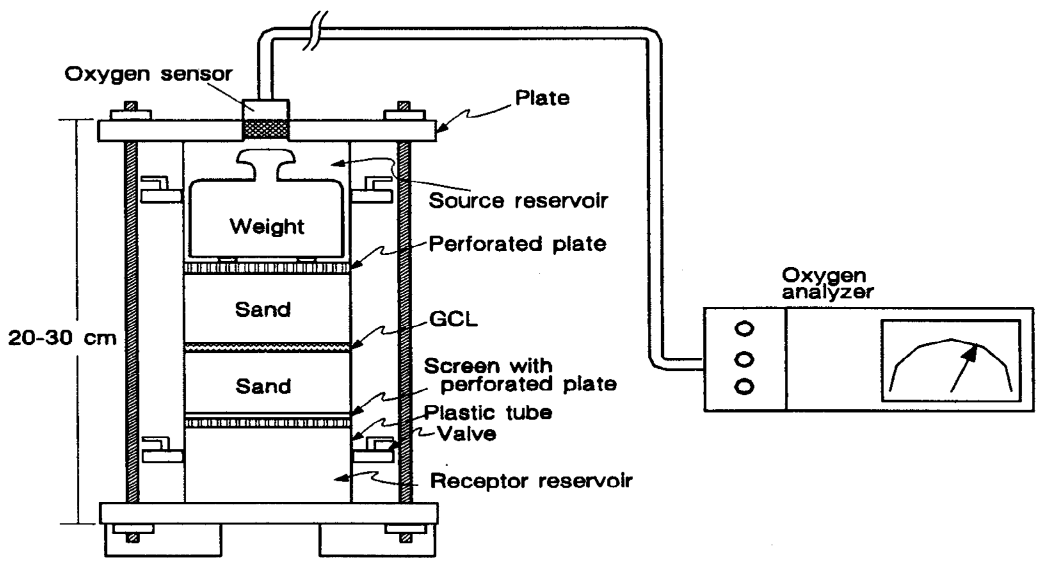

Figure 1 - Experimental set-up for the diffusion cell

The concentration of the source reservoir (and of the receptor reservoir if required) is measured periodically to follow its evolution. The diffusion coefficient D_e through the porous medium can then be obtained by fitting the experimental data to a theoretical solution using POLLUTE, a finite layer contaminant transport program (Rowe et al., 1994). The equations used to solve the problem for the test boundary conditions have been given by Rowe et Brooker (1985) and will not be repeated here. A similar approach has been applied to evaluate D_e in soils and tailings (e.g. Yanful, 1993 ; Aachib et al., 1993 ; Aubertin et al., 1995, 1997).

The experimental setup consists of a transparent PVC cylinder with an internal diameter of 8.5 cm and a length between 20 and 30 cm (according to the size of the sample). The cylinder is closed by 2 caps. A Teledyne (320P or 340 FBS) oxygen concentration measurement device was used, with the sensor fixed at the top of the source reservoir. To check the mass balance during some tests, another sensor was also fixed in the bottom reservoir, but this is not really required for non reactive materials. The four valves in the PVC tube serve to purge the system with nitrogen before the test, and they remain closed during the diffusion so the cell is air-tight. Measurements with manometers during a few tests have shown that the pressure gradient between the bottom and top of reservoir was small enough not to create significant advective gas transport.

As GCL performance depends on water content, porosity and confining pressure, special care was taken to control these factors as much as possible. The GCL sample is first cut to the size of a cell ring. The tube is coated with a special grease to ensure contact between the walls and the GCL. Some bentonite is also added to improve contact. The GCL is placed on a 5 cm thick sand layer, and covered by another 5 cm of sand ; a concrete sand having about 85% of the grains smaller than 1 mm (and a small air entry value) was used. The amount of water added was calculated according to the manufacturers specifications, to fully (or partly in some cases) saturate the GCL. The top sand layer is covered with a perforated plate that supports a dead load equivalent to expected vertical pressure in situ (typically between 10 and 50 kPa). Enough free space is keep for oxygen to flow freely to the sand layer.

It usually took about 10 days to fully hydrate the GCL, as seen by swelling of the sample (Authier,1997). Once this is completed, the cell is purged with nitrogen having a small amount of water vapor so there would be no humidity loss in the GCL. The top reservoir is then very briefly opened to reach atmospheric conditions with an oxygen concentration of about 20.9%. This constitutes the initial condition of the test, which is then allowed to run until a steady state is reached. This may take a few days to a few weeks, depending on the D_e values. The concentration profile over time serves to back calculate D_e from similar curves obtained from POLLUTE, which are adjusted to fit the experimental curves. Various verifications were performed to check the air-tightness of the cells, the accuracy of the oxygen sensor, their stability over time and their proneness to consume oxygen.

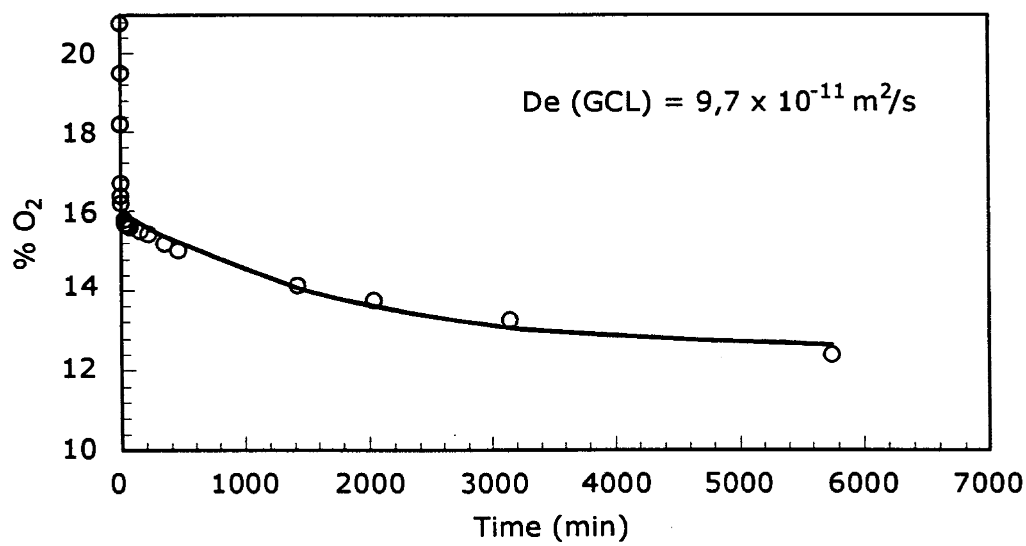

Figure 2 - Measured and calculated values of oxygen concentration in the source reservoir during a diffusion test

During these tests, diffusion occurred in two phases, because two materials are used. The first phase, relatively short (a few minutes), is related to diffusion in sand which usually has a low degree of saturation (because of the capillary effects created by the GCL) and has a high D_e value (fairly close to D_a° according to Equations 6 to 9). The second and much longer phase is that of oxygen diffusion through a highly saturated GCL. An example of experimental results with calculations from POLLUTE are shown on Figure 2. Calculations with POLLUTE (with appropriate values of D_e first established for the top sand layer) illustrate the two phases described above. The interpretation of the diffusion tests with POLLUTE required a slight modification of how Fick's law is treated by the program. Here, the porosity is replaced by an equivalent porosity θ' so that D_e is given by $\theta' D_p$, where D_p is the apparent diffusion coefficient of the gas used in (and back calculated from) POLLUTE. The value of θ' becomes $\theta_a + H\theta_w$, and can be considered an effective gas porosity. More details on this aspect are given in Aachib and Aubertin (1999).

Results shown in Table 1 were obtained on a Bentofix NW (non woven) GCL which contains 3500 g/m^2 of bentonite. At this point, the expected accuracy of the procedure is about 20 to 30%.

Table 1. Experimental data from the diffusion tests on GCL

Test	S_r %	w (%)	n	hydrated thickness (mm)	Estimated D_e m^2/s
1	100	145.3	.757	7.6	1.4×10^{-11}
2	100	105.9	.609	8.6	9.7×10^{-11}
3	71	59.0	.638	7.4	9.5×10^{-8}

In Table 1, w is the water content ($w = \theta_w[(1-n)G_s]^{-1}$, where G_s is the relative solid density ; $G_s = 2.13$ was used for the GCL)

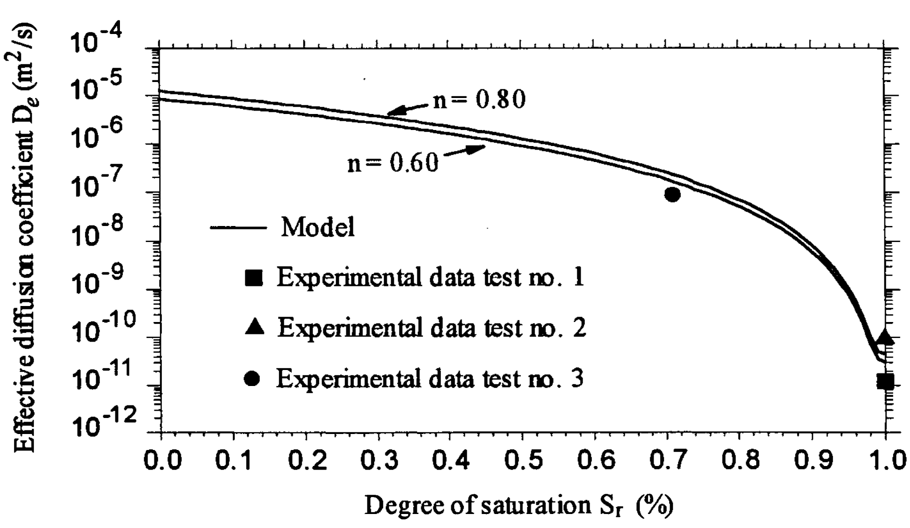

Figure 3 – Estimated De values as a function of Sr; experimental data are compared to predictions from equations 3, 6-9

ANALYSIS

The estimated values of the effective diffusion coefficient D_e can be compared to those obtained from predictive models such as the one given by Equations 6 to 9. Figure 3 shows such a comparison. The experimental results seems to correspond fairly well to the predicted values, for both fully saturated and partially saturated conditions. The accordance between measured and predicted values is as good as that obtained on other porous media (e.g. Aachib et al., 1993, Aubertin et al., 1995, 1997). The results shown in this Figure also show, from a theoretical and a experimental point of view, the decrease of D_e as the degree of saturation increases.

The scatter and limited accuracy of the results may be due in part to the uncertainty of the actual GCL thickness and porosity, the heterogeneity of water distribution, the relative precision of the oxygen sensor, and the adjustment of the predicted curve (from POLLUTE) to measured values. Nevertheless, these preliminary measurements indicate that, provided some minor refinements, the proposed methodology has the necessary attributes to give a good estimate of D_e.

From a practical view point, such measurements allow calculation of gas flux through the GCL used in cover. To illustrate such an application, the steady state solution of Fick's laws is used. This can be written as :

$$F_g = \frac{D_e(C_o - C_1)}{L} \tag{10}$$

where C_o: concentration above the GCL
C_1 :concentration below the GCL
L : thickness of the GCL

For a GCL sandwiched between two sand layers, and placed on reactive tailings that quickly consume oxygen, the flux can be estimated using the following parameter values : C_o = 0.3 kg/m^3 (atmospheric concentration), C_1 = 0 (oxygen consumed), D_e = 5.5 x 10^{-11} m^2/s and L = 0.008 m (from our measurements). This gives a flux F_g of 65 g per square meter per year. This is somewhat higher than the expected flux from an effective cover to control acid mine drainage (e.g. Aubertin et al., 1995, 1997). This would nevertheless represent a decrease of the oxygen flux by a factor of about 250 when compared to the uncovered situation. The performance of the cover with a GCL could be improved by adding a layer of fine grained soil with a high capacity to retain water by capillarity. For instance, 30 cm of silt with a degree of saturation of 80% (with $D_e \cong 2 \times 10^{-8}$ m^2/s) above the GCL would give a flux of about 58 g/m^2/yr ; if the silt was able to maintain a minimum degree of saturation of 90%, as expected from our unsaturated flow modeling calculations (with $D_e \cong 2.5 \times 10^{-9}$ m^2/s), then the flux F_g would be reduced to less than 36 g/m^2/yr, which is close to the targeted range.

These sample calculations show the effect of the GCL on gas flux. To make such necessary calculations for an actual cover design, a measure of D_e is required. The proposed method described above can be used for that purpose.

CONCLUSION

Gas flux by diffusion through a GCL can be an important transport mechanism for various waste disposal sites. Knowing the mass flux of gas going through may become important for various cases such as when bio-gas needs to be contained under the cover system or for limiting oxygen availability to acid generating tailings. In this paper, the diffusion theory is briefly reviewed to highlight the importance of the effective diffusion coefficient D_e on the amount of gas flux. A laboratory procedure is then proposed to measure the value of D_e using a transient flux condition in a closed cell. Preliminary results are shown and compared to predicted values. A few sample calculations are finally made to estimate the mass flux through a GCL with and without adjacent soils. This investigation shows the potential of the diffusion experimental cell and that, with some minor improvements, the proposed method could be used on a regular basis to evaluate the value of D_e, which is required by designers to calculate the gas flux across cover systems.

ACKNOWLEDGMENTS

The authors would like to thank Don Stewart from Terrafix Geosynthetics Inc, and Daniel Jetté and André Rollin from Solmers Internationale Experts-Conseils Inc, who have provided information on GCL. Antonio Gatien has helped with the laboratory tests.

REFERENCES

Aachib, M., Aubertin, M. (1999): Gas diffusion through variably saturated porous media with application to oxygen barrier (submitted for publication).

Aachib, M., Aubertin, M. Chapuis, R.P. (1993) : Étude en laboratoire de la performance des barrières de recouvrement constituées de rejets miniers pour limiter le drainage minier acide. Un état de la question. Report EPM/RT - 93/32. École Polytechnique de Montréal.

Aubertin, M., Aachib, M., Chapuis, R.P., Bussière, B. (1996) :Recouvrements multicouches avec effets de barrière capillaire pour contrôler le drainage minier acide : études en laboratoire et in situ. Exemples majeurs et récents en géotechnique de l'environnement, Presse de l'École Nationale des Ponts et Chaussées, Paris, pp.181-197.

Aubertin, M., Aachib, M., Monzon, M., Joanes, A.-M., Bussière, B., Chapuis, R.P. (1997): Étude de laboratoire sur l'efficacité des barrières de recouvrement construites à partir de résidus miniers. Rapport de recherche. Project CDT 1899 (Dec. '97) Submitted to MEND-CANMET, 107 pages.

Aubertin, M., Chapuis, R.P., Bussière, B. Aachib, M. (1993) : Propriétés des résidus miniers utilisés comme matériau de recouvrement pour limiter le DMA. Geoconfine '93. Arnoud, Barrès and Comes (eds). Balkema. Vol I. pp. 299-308.

Aubertin, M., Chapuis, R.P., Aachib, M., Bussière, B., Ricard, J.F. Tremblay, L. (1995): Evaluation en laboratoire de barrières sèches construites à partir de résidus miniers. École Polytechnique de Montréal, MEND Projet 2.22.2a, CANMET, Ottawa (March '96), 164 pages.

Authier, K. (1997) : Étude du transport de l'oxygène dans un géocomposite bentonitique. Unpublished Report (Projet de fin d'étude), École Polytechnique de Montréal, 87 pages.

Bienvenu, L. (1998): Activités de recherche du ministère des ressources naturelles du Québec sur le drainage minier acide, Rapport 1997-1998. Entente auxiliaire Canada-Québec sur le développement minéral. MNRQ, 38 pages.

Bouazza, A., Van Impe, W.F., Van Den Broek, H. (1996): Hydraulic conductivity of a geosynthetic clay liner under various conditions. Environmental Geotechnics, Karmon (ed.) Balkema, Rotterdam. pp. 453-457.

BRGM (1994, 1995): Définition des éléments constitutifs nécessaires à une couverture finale en fonction des exigences des différents types de décharges-C.S.D., Technical reports (Rapports d'avancement No. 1, 2, 3 et Document Synthèse). ANTEA, Orléans, France.

Collin, M., Rasmuson, A. (1988): Gas diffusivity models for unsaturated porous media. Soils Sciences Amer. J., Vol 52, pp.1559-1565.

Collin, M., Rasmuson, A. (1990): Mathematical modelling of water and oxygen transport in layered soil covers for deposits of pyritic mine tailings. GAC-MAC Annual Meeting - May. Acid Mine Drainage Designing for Closure, Vancouver. pp. 311-333.

Crank, J. (1975): The Mathematics of Diffusion. 2nd edition. Clarendon Press, Oxford, United Kingdom

Daniel, D.E. (1996): Geosynthetics clay liners, Part two: Hydraulic properties. Geotechnical Fabrics Report, Vol.14, June/July '96, pp. 22-26.

Daniel, D.E., Shackelford C.D. (1988) : Disposal barriers that release contaminants only by molecular diffusion. Nucl. Chem. Waste Manage. Vol.8, pp. 299-305.

Daniel, D.E., Koerner, R.M., Bonaparte, R., Landreth, R.E., Carson, D.A., Scranton, H.B. (1998); Slope stability of geosynthetics clay liners tests plots. Journal of Geotechnical And Geoenvironmental Enginnering, ASCE, Vol,124, No.7, pp. 628-637.

Desaulniers, D.E., Cherry, J.A., Fritz, P. (1984) : Hydrogeologic analysis of long term solute migration in thick clayey quaternary deposits. Proc. Int. Symp. on Groundwater Ressources Utilization and Contaminant Hydrogeology, Montréal, Qué., May 1984, Int. Assoc. Hydrodeol., Vol. II, pp. 349-356.

Eberling, B., Nicholson.R.V., Reardon, E.J., Tibble, P. (1994) : Evaluation of sulphide oxidation rates : a laboratory study comparing oxygen fluxes and rates of oxidation product release. Canadian Geotechnical Journal, Vol. 31 pp. 375-383.

Estornell, P., Daniel, D.E. (1992): Hydraulic conductivity of three geosynthetic clay liners. Journal of Geosynthetic Engineering, ASCE, Vol.118, No.10, 1592-1606.

Fox, P.J., De Battista, D.J., Chen, S.H. (1996): Bearing capacity of geosynthetic clay liners for cover soils of varying particle size, Geosynthetics International, IFAI, Vol.3, No.4, pp. 447-461.

Fredlund, D.G., Rahardjo, H. (1993) : Soils Mechanics for Unsaturated Soils. Wiley-Interscience Publication. 517 pages.

Freeze, R.A., Cherry, J.A. (1979): Groundwater, Prentice-Hall, Englewoods Cliffs, New Jersey, 600 pages.

Frobel, R.K. (1996): Geosynthetic clay liners, Part four: Interface and internal shear strength determination. Geotechnical Fabrics Report, Vol.15, March '97, pp. 20-25.

Gilbert, R.G., Fernandez, F., Horsfield, D.W. (1996): Shear strength of reinforced geosynthetic clay liner, Journal of Geotechnical Engineering, ASCE, Vol.122, No.4, pp. 259-266.

Gillham, R.W., Robin, M.J.L., Dytynyshyn, D.J., Jonhston, H.M. (1984): Diffusion of nonreactive and reactive solute through fine grained-barrier materials, Canadian Geotechnical Journal, Vol.21, pp. 541-550.

Koerner, G., Allen, S. (1997): The water vapor transmission testing controversy. Geotechnical Fabrics Report, Vol.15, March '97. pp 8-12.

Koerner, R.M. (1994): Designing with Geosynthetics, 3rd Ed., Prentice-Hall, Engelwood Cliffs, N.J.

Koerner, R.M. (1996): Geosynthetics clay liners, Part one: An overview, Geotechnical Fabrics Report, Vol. 14, May '96, pp.22-25.

Koerner, R.M., Daniel, D.E. (1997): Final Covers for Solid Waste Landfills and Abandoned Dumps. ASCE.

Koerner, R.M., Gartung, E. Hanzinger, H. (eds)(1995): Geosynthetics Clay Liners. Proc. Int. Symp., Nürnberg A.A. Balkema, Rotterdam, 256 pages.

Koerner, R.M., Narejo, D. (1995): Bearing capacity of hydrated geosynthetic clay liners, Journal of Geotechnical Engineering, ASCE, Vol.121, No.1, pp. 82-85.

Lake, C.B., Rowe, R.K., Petrov, R.J, Stewart, D., Von Maubeuge, K. (1997): Diffusion of chloride through a geosynthetic clay liner. Off-Print. 50th Canadian Geothechnical Conference, Ottawa , 8 pages.

Millington, R.J., Shearer, R.C. (1971): Diffusion in aggregated porous media. Soil Science, Vol. 111, pp. 372-378.

Naue-Fasertechnik (1992): Gas permeability of Bentofix® Clay Liners to Methane Gas and Petrol Fume. Test Report (unpublished). 9 pages.

Nicholson, R.V., Gillham, R.W., Cherry, J.A., Reardon, E.J. (1989): Reduction of acid generation in mine tailings through the use of moisture-retaining cover layers as oxygen barriers. Canadian Geotechnical Journal, Vol. 26, pp. 1-8.

Petrov, R.J., Rowe, R.K (1997): Geosynthetics clay liners (GCL)- Chemical compatibility by hydraulic conductivity testing and factors impacting performance, Canadian Geotechnical Journal , Vol. 34, pp. 863-885.

Richardson, G.N. (1997): GCL internal shear strength requirements. Geotechnical Fabrics Report, Vol.15, March '97, pp. 20-25.

Roslton, D.E. (1986) : Gas diffusivity. Method of Soil Analysis. Part 1. Physical and Mineralogical methods. Chap. 47. Klute (eds), Agronomy Monograph. 9, ASA.

Rowe, R.K. (1987) : Pollutant transport through barriers. R.D. Woods (ed), Geotechnical Practice for Waste Disposal. ASCE,. No.13, pp. 159-181.

Rowe, R.K., Brooker, J.R. (1985) : 1-D pollutant migration in soils of finite depth. Journal of Geotechnical Engineering, ASCE, Vol. 111, (GT4) pp. 479-499.

Rowe, R.K., Brooker, J.R., Fraser, M.J. (1994). POLLUTEv6 and POLLUTE-GUI User's Guide. GAEA Environmental Engineering Ltd, London, Ontario, 305 pages.

Rowe, R.K., Caers, C.J., Barone, F. (1988): Laboratory determination and distribution of contaminants using undisturbed clayey soil. Canadian Geotechnical Journal. Vol.25 pp.108-118

Rowe, R.K., Quigley, R.M., Brooker, J.R. (1995) : Clayey Barrier Systems for Waste Disposal Facilities. E&FN Spon (Chapman & Hall) London : 390 pages.

Senes consultants LTD. (1994): Evaluation of Alternate Dry Covers for the Inhibition of Acid Mine Drainage from Tailings. Final Report, MEND Project 2.20.1, CANMET, Ottawa.

Shackelford, C.D. (1991): Laboratory diffusion testing for waste disposal- A review. Journal of Contaminant Hydrology, Vol.7, pp. 177-217.

Takahashi, S., Kondo, M., Karmon, M. (1996): Evaluation of swelling behavior and permeability of geosynthetic clay liner. Environmental Geotechnics, Karmon (ed.) Balkema, Rotterdam. pp. 609-614.

Yanful, E.K. (1993): Oxygen diffusion through soil covers on sulphidic mine tailings. Journal of Geotechnical Engineering, ASCE, 119, No.8, pp. 1207-1228.

PEEL TESTING OF NEEDLE-PUNCHED GEOSYNTHETIC CLAY LINERS' TEST PROCEDURE, INTERPRETATION AND TEST RESULTS

ROBERT E. MACKEY, P.E., S2L, INC.
UNITED STATES OF AMERICA
KENT P. VON MAUBEUGE, NAUE FASERTECHNIK GMBH & CO KG
FEDERAL REPUBLIC OF GERMANY

ABSTRACT

Needle-punching or stitch-bonding of the overlying and underlying geotextiles is commonly used to increase the internal shear strength of geosynthetic clay liners (GCLs). ASTM D35.04 Subcommittee on GCLs is currently developing a peel strength standard for needle-punched GCLs. In some previous publications, it appeared as if a correlation could be made between GCL peel strength and GCL internal shear strength. A possible improved correlation of test data may be possible in the future if the peel test procedure could be standardized.

This paper details the proposed test method for determination of peel strength, developed by ASTM, and possible future correlation of the peel strength to internal shear strength.

INTRODUCTION

Geosynthetic clay liners (GCLs) consist of factory-manufactured rolls of bentonite placed between geotextiles or adhesively bonded to a geomembrane. The bentonite is the low-hydraulic conductivity (or permeability) component of this composite material, while the geosynthetics act as containment materials. The geosynthetics also provide manufacturers with the opportunity to stitch-bond, needle-punch, or adhesively bond the bentonite into a product stable for handling, transportation and placement as a composite barrier material (Koerner, 1996(a)).

Bentonite composed of sodium montmorillonite (i.e., sodium bentonite) is commonly used in GCLs. Sodium bentonite has an extremely low hydraulic conductivity and remarkable swelling and self-healing properties (Daniel 1996). However, the shear strength of bentonite has caused significant concern over the use of unreinforced GCLs. Bentonite has a relatively low shear strength (Olson 1974). The low shear strength has spurred a significant number of investigations regarding GCLs' interface and internal shear strengths and their impact for GCL use on side slopes. The most notable study is the Cincinnati GCL Test Plots (Daniel, 1998) where different conbinations of soil, geocomposites, geomembranes and GCLs were placed to evaluate their side-slope stability (Koerner, 1996(b)).

Bentonite has an angle of internal friction of approximately 8 degrees (peak) (Daniel, 1994) and a residual value of approximately 4 to 5 degrees. Needle-punching of a GCL binds two geotextiles of the GCL together, and also acts to increase the internal shear strength of the GCL. Gilbert et al (1996) reported that the reinforcing fibers served to increase the peak internal shear strength of the GCL. Stark and Eid (1996) reported that the internal shear strength of reinforced GCLs depends on the resistance against pull-out and/or tearing of the reinforcing fibers, and on the shear strength of the bentonite, with fiber resistance representing the predominant factor. Several studies have been performed to relate the peel strength of needle-punched or stitched GCLs to their internal shear strength (Heerten et al, 1995, Berard, 1997, Richardson, 1997, Fox, 1998, von Maubeuge and Eberle, 1998). Although these studies show a relationship between peel strength of a GCL and its internal shear strength, correlation between these studies has been difficult. This is due to the use differing test methods to evaluate peel strength and internal shear strength.

ASTM D-35 Committee on Geosynthetics has recently released ASTM D6243-98, "Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liners by the Direct Shear Method". Compliance with this method will standardize the testing procedures for determining internal shear strength. What is now needed is a standard method for the determination of bonded peel strength of needle-punched and stitched GCLs. ASTM D35.04 Subcommittee on Geosynthetic Clay Liners is in the process of developing such a standard method for needle-punched GCLs. This current draft test method is called, "Standard Test Method for Determining Bonding Peel Strength between the Top and Bottom Layers of Needle-punched Geosynthetic Clay Liners".

This paper addresses the proposed test procedures for the bonded peel strength test method and the need for this standard as an index test for manufacturers' quality control. This paper also describes the internal shear strength relationship with bonded peel strength and prior attempts to relate peel strength to internal shear strength.

NEEDLE-PUNCHED GCLs

The manufacturing process for a needle-punched GCL is shown in Figure 1. The GCL is manufactured on a continuous production line, whereby a predetermined quantity of sodium bentonite is confined between two geosynthetics. For the cover (upper) layer, needle-punched staple fiber nonwovens are used, whereas the carrier (lower) layer can be a nonwoven (sometimes even scrim reinforced), a woven or any other geosynthetic which allows an anchoring of fibers from the cover layer through needle-punching. The geosynthetics are needle-punched together, through the thickness of the bentonite, securing the bentonite in place and reinforcing the otherwise weak layer of clay (when hydrated) over the entire surface. The goal of needle-punching a GCL is to achieve an interlocking of the fibers of the cover geotextile with the carrier geotextile.

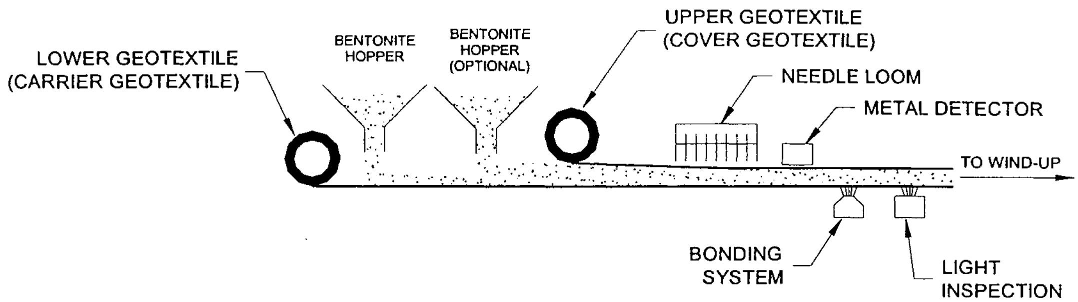

Figure 1. Schematic diagram of the manufacturing process for needle-punched GCL (Koerner 1996a)

Needle-punching is the most common method of forming nonwoven geotextiles, creating a mat with optimal stress/strain characteristics due to the complex entanglement of the fibers. Needle-punching of a nonwoven through the bentonite layer of a GCL not only provides bonding between the carrier geosynthetic and the cover nonwoven, but also serves to fix the bentonite in place, preventing displacement within the plane of the GCL in changing conditions. The geotextile components and needle-punching of the GCL allow the bentonite layer to be held in position during transportation and installation. The bentonite is held in place during the expected service life of the GCL. Bentonite extrusion has been observed through the woven components and occasionally through thinner nonwovens (< 200 g/m^2) (Fox, et al, 1998).

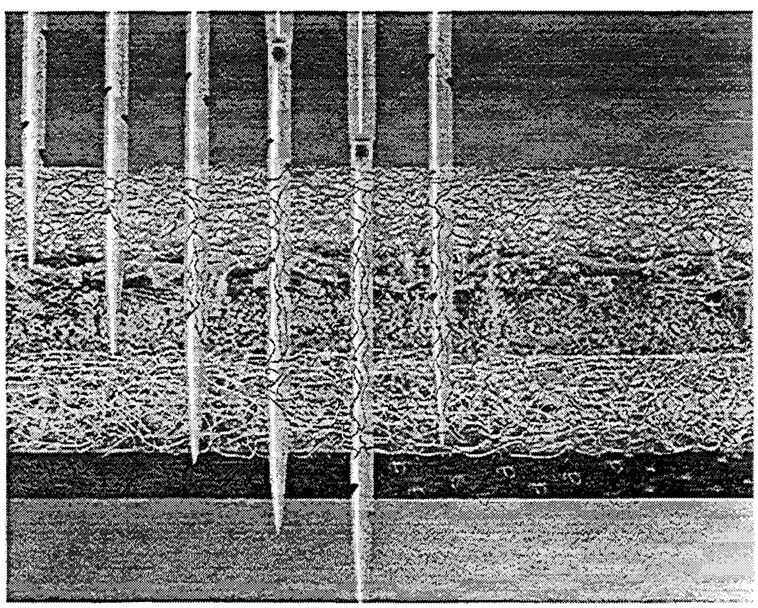

Figure 2. Needle-punched GCL Process

The GCL peel test evaluates the bonding strength of the cover and carrier geotextiles of a needle-punched GCL, and is an index value used to determine possible wearing of the needles due to abrasion during the production process (Figure 2). A reduction of the peel values will therefore indicate that the needle-punching process does not carry enough fibers from the cover layer to the carrier layer and that operational changes must be carried out to ensure the minimum guaranteed peel strength. Once the peel results begin to approach minimum acceptable values, one operational procedure would be to replace the worn needles with new ones. Peel testing is usually performed at a minimum frequency of 5 test specimens per 4,000 m^2 (40,000 ft^2) of produced GCL. Typically the testing frequency is increased once the peel strength approaches the minimum acceptable value. The current reported value for GCL manufacturers is the average of 5 peak values, and the minimum reported average value usually ranges between 30 and 100 N/10 cm (7 to 23 lbs/4 inches) using a wide-width tensile test.

It is important for project-specific testing (such as shear testing) to measure the peel strength of the GCL from specimens taken from the same GCL sample as the shear test specimens. It should be understood that higher peel strength correlates with higher shear strengths. This is clearly documented by Berard 1997, Heerten et al, 1995, Fox 1998, and von Maubeuge and Eberle, 1998.

PEEL TESTING OF NEEDLE-PUNCHED GCLs

Past research into the relationship between the bond (peel) strength and internal shear strength of GCLs has required investigators to modify existing ASTM test methods to determine peel strength. Test methods used in the past include ASTM D 4437 for determining the integrity of geomembrane field seams (Berard, 1997), ASTM D 4632 for grab load and elongation of geotextiles (Fox 1998, Richardson, 1997) and ASTM 4595 for wide width tensile strength. Each test method records the maximum load required to peel the GCL during the test. Since these methods were not developed for peel testing of GCLs, their ability to accurately determine peel strength is subject to improvement.

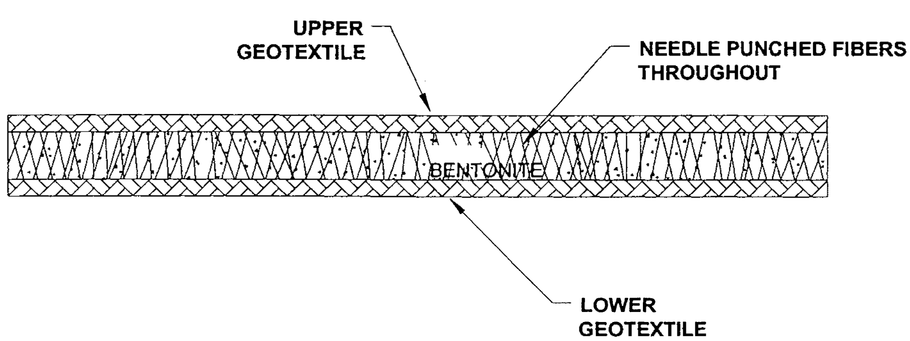

Figure 3. GCL Cross-Section.

The objective of the peel test is to place a tensile load on the geotextile fibers joining one geotextile to another. If cross-sections were taken across a GCL (Figure 3) at different locations, a varying number of fibers could be seen running through the bentonite component of the GCL. It is these fibers which translate upper geotextile movement to the lower geotextile and thus increase the internal shear strength of a GCL. The number of fibers connecting the upper and lower geotextiles usually ranges from 2 to 3 million per m^2, and their strength is based on the GCL manufacturing process, number of needles per square meter, and age/effectiveness of the needle-punching operation.

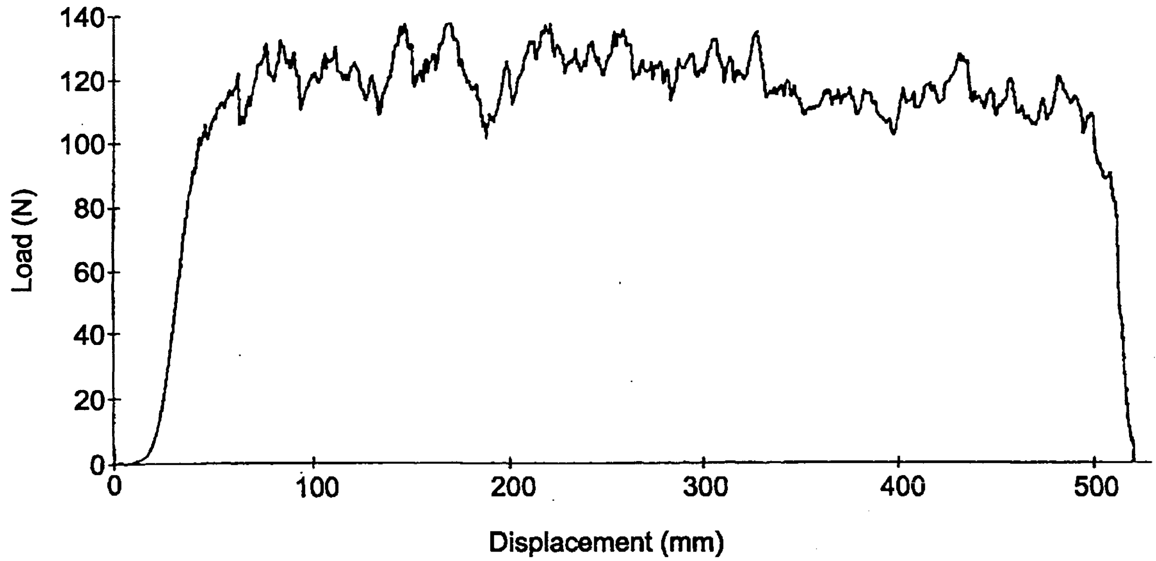

Figure 4. Typical Peel Test Results for a Needle-Punched GCL

Figure 4 shows typical results of peel testing of a needle-punched GCL using a 10 cm wide GCL specimen in ASTM D4595 clamps at a rate of 300 mm/min. (The result shown was performed by NSC Aurora, November 17, 1997, for the ASTM round-robin peel test evaluation. The specimen size was 100 mm x 300 mm and separation of top and bottom geotextile for the initial 50 mm.) The various peaks and low points indicate fibers being placed under tension until they break or pull-out, followed by relaxation of the specimen until other fibers are subjected to the tensile load. There was considerable discussion by the ASTM Subcommittee on GCLs (D35.04) in charge of developing a peel test regarding the ability to interpret these data and to define the separation speed and the clamping method.

It was clear to the Subcommittee that the initial portion of the curve represented the geotextiles of the specimen being placed under tensile loads and elongating before peeling of the GCL actually occurs. This portion of the curve needed to be discarded as part of the peel strength evaluation. The remaining portion of the curve required review of several options to calculate the peel strength of the GCL.

These options included:

- Maximum load,
- Minimum load,
- Averaging the 10 highest peaks,
- Average of 10 intervals (time periods or length of GCL), and
- Average load from "x" mm to "y" mm.

Each method represented different interpretations of the data and, therefore, different end-results. The Subcommittee did develop a consensus, and a draft standard is currently under review by the ASTM D35.04 Subcommittee. The current draft requires the test apparatus to average the load required to peel the GCL over 100 mm (4 inches) of the test specimen. The initial and final recorded peel strengths of the GCL are to be disregarded, as detailed in the following section. The Subcommittee believes that this approach for determining needle-punched GCL peel strength offers a reasonable and fair test result for the product.

DESCRIPTION OF PROPOSED PEEL TEST METHOD

The proposed test method requires the cutting of five 100 mm by 200 mm (4 inches x 8 inches) test specimens at random locations across the entire length of the sample. All specimens should be parallel to the machine direction. A knife or razor is used to separate the top and bottom layer of the GCL for 50 mm (2 inches). Each specimen is mounted centrally in clamps with the dimension of 25 mm x 100 mm (1 inch x 4 inches) (Figure 5). The distance between the clamps is set at 50 mm apart at the start of the test. The instrument places a tensile load to peel the GCL at a constant rate of 300 mm/min (12 in./min).

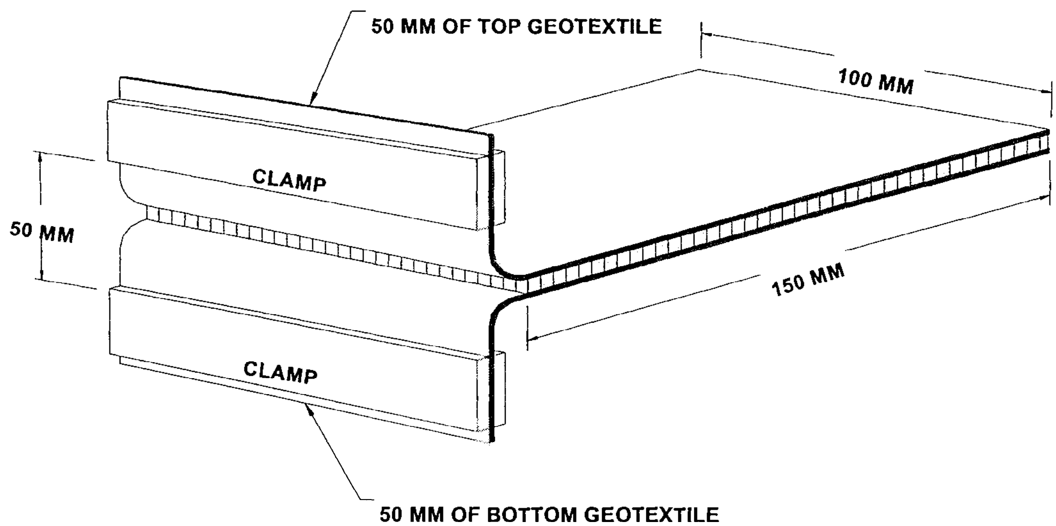

Figure 5. GCL Specimen Mounted in Clamps

Readings for the test specimen are to be taken after 50 mm (2 inches) of clamp displacement. This allows for the geotextile to elongate and start separating the top and bottom geotextiles before the readings start. The test continues until 250 mm of clamp displacement occurs. The elongation of the geotextiles during the period of peel test readings is considered to have no bearing on the test results. The start of the peel test readings can be delayed past the initial 50 mm (2 inches) of clamp separation if the readings indicate that the needle-punched fibers are not sufficiently under stress. Peel test readings must be taken over 200 mm of grip displacement or approximately 100 mm (4 inches) of GCL geotextile layers separation in order to have a valid test result. The operator should disregard any readings at the end of the peel test where fewer fibers are under stress and the recorded load significantly decreases (Figure 4).

The average tensile load is calculated for the test period and expressed with the full number, e.g. 655 N/m (369 lbf/in) per unit width. The reported value is the average result of the five specimens.

The Subcommittee believes the loss of bentonite during the cutting process should have no effect on the results of the test. Shaking the bentonite out of each specimen before testing may also have no effect on the test. However, a future evaluation of a significant loss of bentonite from the specimen before testing and the subsequent test results should be performed before this becomes standard practice with testing laboratories. This test method is expected to be somewhat "messy" and requires constant cleaning of bentonite from the test instrument.

The above procedure should give a representative measurement of the peel strength of needle-punched GCLs. The Subcommittee must work to finalize the above peel test method and obtain approval from the ASTM D-35 Committee. After approval is gained, the Subcommittee will develop a Precision and Bias Statement to denote the ability of inter-laboratory testing to obtain the same test results. This test method will be used initially and primarily as an index test for manufacturing quality control. However, it is possible that correlation could be obtained between peel strength and internal shear strength of GCLs. It is only recently that a standard test method for determining the shear strength of GCLs has been developed.

INTERNAL SHEAR TEST PROCEDURES

ASTM D-35 has recently released ASTM D 6243 to be used to determine the interface and internal shear resistance of GCLs. Previous investigations into interface and internal shear strengths of GCLs used ASTM D 5321. Many people active in the testing of GCLs believe ASTM D 5321 was not fully appropriate for the testing of GCLs. This was particularly true regarding the testing of GCLs for internal shear strength. A significant difference between the two test methods is that ASTM D 5321 requires only clamping the GCL into the apparatus, and ASTM D 6243 allows the GCL to be held in place by roughened surfaces and clamps if the internal shear strength is investigated. The roughened surfaces reduce the possibility of the GCL pulling away from the apparatus or "rolling" within the apparatus and create a realistic soil behavior. It also reduces progressive shear failure.

The ASTM D 6243 procedure for determining internal shear resistance by the direct shear method generally consists of adjusting the lower roughened surface so that it is one-half the thickness of the GCL below the top of the lower box. The GCL is placed over the entire roughened surface and clamped into place at the ends. The two halves of the shear box are slid together and fixed in the start position. The top roughened surface is placed over the GCL. Then the loading plate is fixed into place and a normal load applied. The initial load is a seating load. If the test is for a hydrated GCL, vertical displacement is monitored until stabilized. Once stabilized, the normal load for testing is applied and vertical displacement monitored. The shear force is applied at a constant rate and is recorded as a function of displacement. The test normally runs until the horizontal displacement exceeds 50 mm (2 inches). After a minimum of three specimens have been tested, the internal shear strength parameter is calculated using set procedures detailed in the test method.

The Subcommittee is in the process of developing a Precision and Bias Statement to denote the ability of inter-laboratory testing to obtain consistent test results. The initial objective of the Subcommittee is to determine the precision and bias of the test method for "interface" shear resistance, and then later assess "internal" shear resistance. Koerner and Soong (1998) reported significant scatter in the direct shear test data for the testing of GCL/textured geomembrane interface using ASTM D 5321. This data scatter or lack of precision of this test method warrants concern regarding the ability to correlate direct shear test results to peel strength test results. The Subcommittee hopes that ASTM D 6243 significantly improves the precision for the direct shear testing of GCLs in comparison to the test results using ASTM D 5321.

CORRELATION OF PEEL STRENGTH AND INTERNAL SHEAR STRENGTH

Heerten et al (1995) demonstrated that there is a correlation between peel values and the internal shear strength of needle-punched GCLs, when tested under similar conditions. In their studies, GCLs were hydrated for 24 hours under no confining stress. Under normal installation conditions, it is unlikely that the GCL would hydrate within 24 hours with no confining stress. Usual installation procedures require the GCL to be immediately covered if inclement weather is expected. However, in Germany, GCLs are used as canal liners and have been installed under water (Daniel and Boardman, 1993, von Maubeuge and Witte, 1998). Under these conditions, it was likely that the GCL would be exposed to a 24 hour period of unconfined hydration, and therefore the correlation studies did simulate realistic site conditions.

The objective for the correlation between peel strength and internal shear strength is to develop a design tool, such as detailed in Figure 6. Figure 6 shows a design diagram for the determination of maximum permissible slope angle as a function of cover soil depth ($\gamma = 20$ kN/m^3). For Figure 6, the GCL was prehydrated under 0 kN/m^2 (no confining stress) for 24 hours and then sheared internally. The peel strength of the GCL was also measured to develop the detailed correlation. The peel strength of the tested needle-punched GCLs assumes a zero cohesion intercept and no passive wedge at the bottom of the slope. The shear plane is assumed to be outside the needle-punched GCL if the determined value lies above the design inclination.

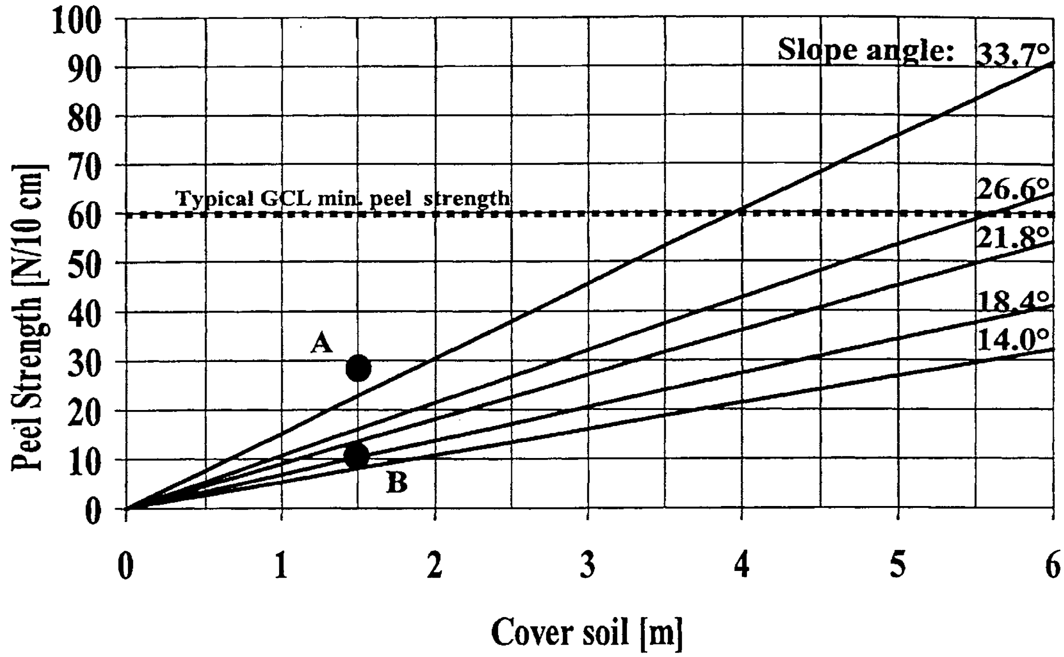

Figure 6. GCL Design Diagram for Peel Strength versus Depth of Cover Soil and Slope Angle. (Revised Figure of Heerten, et al, 1995)

In a landfill cap with 1.50 m (5 ft) of cover soil and a slope inclination of 18.4 degrees (3H:1V), a GCL peel value of approximately 10 N/10 cm (dot B in Figure 6) would be sufficient to provide enough internal shear strength to withstand these design criteria. If a safety factor of 1.5 were required, the designer would use the 2:1 or 26.6 degree slope and determine the GCL peel strength of 16 N/10 cm is required for the same depth of cover soil.

Please note that GCLs usually have peel strengths of 60 N/10 cm or more. Long-term shear tests (since October 1993) have demonstrated that a peel strength of 29 N/10 cm on a 25 degree slope (2.1:1) is resisting the shear stress from a 31 kN/m² normal load (6 kN/m² gravel and 25 kN/m² steel plates – dot A in Figure 6) (von Maubeuge and Eberle, 1998).

Increasing hydration time under unconfined conditions will reduce the internal shear values of a needle-punched GCL. Such tests were carried out by Berard (1997) and are plotted in Figure 7. The data show the maximum shear strength of a needle-punched GCL with a peel value of 78 N/10 cm at confining stresses ranging from 25 to 100 kN/m² and various GCL moisture contents. The marked lines (with arrows) represent data from Eberle and von Maubeuge (1998) on the water content of fully hydrated needle-punched GCLs under various confining stresses. The arrows indicate the maximum moisture content of the GCLs under the various confining stresses. Both data correlate well showing that the maximum water contents of the Berard shear tests are similar to the water contents of fully hydrated needle-punched GCLs.

For example, from Figure 6 a peel strength of 78 N/10 cm at a slope angle of 33.7 degrees allows a maximum of 5.25 m cover soil depth. Assuming a soil unit weight of 20 kN/m^3, the maximum design internal shear stress would be 70 kN/m^2 (5.25 m x 20 kN/m^3 x tan 33.7°). Dot C in Figure 7 represents the maximum shear strength at 78 N/10 cm from Figure 6 with a normal load of 105 kN/m^2 (5.25 x 20). Berard (1997) achieved values of approximately 85 kN/m^2 for a normal load of 100 kN/m^2. This demonstrates that the design diagram (Figure 6) is conservative.

It is important to note that Figure 6 was based on the GCL achieving full hydration under 0 kN/m^2 confining pressure. GCLs prehydrated under a confining stress have higher shear strengths than those prehydrated under no confining stress. An example of this effect is noted in Table 1. The increase in water content will generate positive pore pressures when the normal load is applied for the shear test, therefore reducing the interaction between the clay particles Berard (1997).

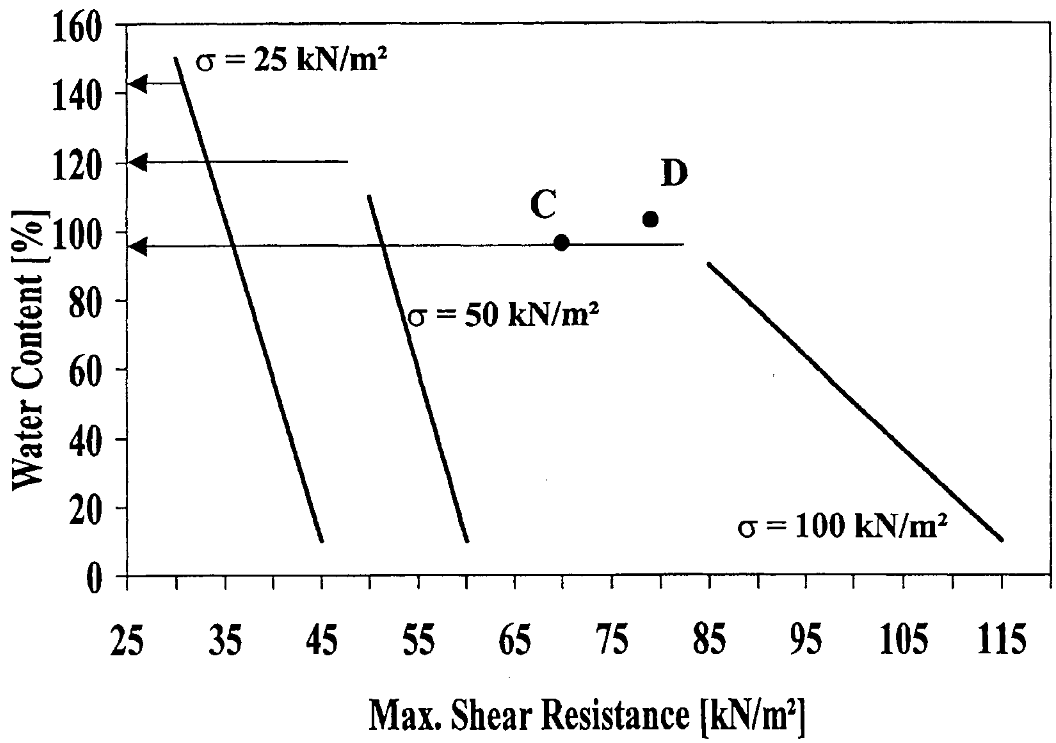

Figure 7. GCL Shear Resistance with a Peel Value of 78 N/10 cm at Different Water Contents

From another study (Prüfamt, 1995), the internal shear strength of a needle-punched GCL (peel 78 N/10 cm) at 104 percent hydration under a confining stress of 80 kN/m^2 is shown in Figure 7 as dot D with a shear resistance of 78 kN/m^2. This again indicates the measured shear results are higher than the maximum values shown in the conservative design diagram.

Table 1. Effects of Hydration Under Load Versus Shear

Peel strength	Shear Values (kN/m^2)	
(N/10cm)	Hydration under 0 kN/m^2	Hydration under 50 kN/m^2
48	48.4	60.0
65	63.8	93.8
77	70.0	87.0

If the testing conditions are clearly identified, a correlation between peel and shear strength can be found. In both of the above-mentioned investigations, the determined peel values were determined under similar conditions and the maximum peel values measured in the peel test were used for the correlation. Influencing factors for the correlation are time of unconfined hydration, separation speed, size and type of clamps, sample size and reporting method.

ACCURACY OF THE CORRELATION OF THESE TEST METHOD RESULTS

Although the review of the literature has shown that several studies have correlated peel testing with internal shear strength, the limitations or precision of these test methods have not been considered. If the precision of the peel strength test method and direct shear test method is taken into consideration, the ability to accurately compare the above figures cannot be verified at this time. Both test methods require reporting the average test results of multiple specimens, along with the standard deviation, if requested. The correlation of the test results should consider the precision/standard deviation of each test method and precision/standard deviation between the testing laboratories. The design engineer must incorporate a sufficient "factor of safety" into the design to accommodate the precision of these tests, especially if using charts/figures generated by someone else. A review of the precision of these test methods must be performed before a sufficient factor of safety for the design can be recommended.

RECOMMENDATIONS

Additional studies are required to accurately define the correlation between the peel strength of needle-punched GCLs and their internal shear strength. Until a sufficient database is generated, a conservative approach to the correlation of test results from these two different test methods is recommended. Figure 6 is an example of this conservative approach. Before the shear test was performed, the GCL was fully prehydrated under 0 kN/m^2 confining pressure. This test condition gave the lowest possible internal shear results. Since many applications for GCLs will allow the GCL to become hydrated under some normal load, the test condition of prehydrated GCL under 0 kN/m^2 confining pressure offers a good starting point for the design engineer.

It is possible to develop a better peel strength versus shear strength correlation that more accurately reflects actual design conditions (i.e. expected normal loads prior to hydration or water content of the bentonite). However, the design engineer should be fully aware of the individual test conditions for which the correlation is based. The design engineer should also account for the precision of each test method and its impact on correlation of the test results. This is particularly true for test results from differing test laboratories or other sources.

The design engineer should use a sufficient factor of safety in comparing peel test results to expected shear results. Until the GCL peel strength test method is finalized and the precision of the peel strength and direct shear test method determined, a conservative factor of safety is recommended. Since the peel strength of most needle-punched GCL is at 60 N/10 cm or more, designs that have GCLs with low normal loads should have a high factor of safety. However once the normal stress increases to a point where it exceeds the required peel strength (based on Figure 6), the design engineer should realize other factors may add to the internal shear strength of the GCL. For example, the water content of the bentonite has probably decreased and cohesion of the bentonite will start playing a major role in adding to the internal shear strength of the GCL.

CONCLUSION

Several studies have shown that a correlation can be developed between the peel strength and internal shear strength of needle-punched GCLs. The correlation between peel strength/internal shear strength could be of great benefit to the design engineer. It would allow the peel test results to confirm that the needle-punched bonding of the top and bottom geotextiles is sufficient to exceed the internal shear strength requirements of the design. The peel test would be a faster, simpler, and less expensive test to perform. However, additional studies are needed to understand the test/field conditions that impact the correlation of these test methods.

ASTM D 35.04 Subcommittee on Geosynthetic Clay Liners is currently developing a test method to determine the peel strength of needle-punched GCLs. The development of standardized test procedures should provide a sufficient level of confidence in the quality of the needle-punch method of binding the layers of geotextile together for GCL manufacturing. ASTM D 35 must first finalize and approve the test method for determining the peel strength of needle-punched GCLs. Once approved, this new test method and ASTM D 6243-98 must undergo precision and bias review to determine result reproducibility. Test conditions that influence the peel strength/internal shear strength correlation must be determined and fully understood. Only after these tasks are accomplished, can the design engineer have a simple and reliable design method for using GCLs on sideslopes and addressing "internal" shear design concerns.

REFERENCES

ASTM D 4437-88, “Standard Practice for Determining the Integrity of Field Seams Used in Joining Flexible Polymeric Sheet Geomembranes”, Annual Book of ASTM Standards, Vol. 04.09

ASTM D 4595-86, “Standard Test Method for Tensile Properties of Geotextile by Wide-Width Strip Method”, Annual Book of ASTM Standards, Vol. 04.09

ASTM D 4632-91, “Standard Test Method for Grab Breaking Load and Elongation of Geotextiles”, Annual Book of ASTM Standards, Vol. 04.09

ASTM D 5321-92, "Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method”, Annual Book of ASTM Standards, Vol. 04.09

ASTM D 6243-98, “Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liners by the Direct Shear Method”, Annual Book of ASTM Standards, Vol. 04.09. (To be published in 1999)

Berard, J.F., (1997) “Evaluation of Needle Punched Geosynthetic Clay Liners’ Internal Friction”, Geosynthetic ’97, Long Beach, March 1997, pp. 351-362

Daniel, D. E., et al, (1998) “Slope Stability of Geosynthetic Clay Liner Test Plots”, Journal of Geotechnical and Geoenvironmental Engineering, pp. 628-637

Daniel, D. E., (1996) “Geosynthetic clay liners, part two: hydraulic properties”, Geotechnical Fabric Report, June/July 1996, pp. 22-26

Daniel, D. E., (1994) “Designing with Geosynthetic Clay Liners,” Course Notes, University of Texas at Austin.

Daniel, D. E., Boardman, B. T., (1993), "Report of Workshops on Geosynthetic Clay Liners", EPA Report EPA 600/R-93/171, Washington

Eberle, M., von Maubeuge, K. P., (1998) "Measuring the in-situ moisture content of Geosynthetic Clay Liners (GCLs) using time domain reflectometry (TDR)", 6th International Conference on Geosynthetics, Atlanta, pp. 205-210

Fox, P. J., et al, (1998) “Internal Shear Strength of Three Geosynthetic Clay Liners”, Journal of Geotechnical and Geoenvironmental Engineering, pp. 933-944

Gilbert, R.B., et al, (1996) "Shear Strength of Reinforced Geosynthetic Clay Liner", Journal of Geotechnical Engineering, American Society of Civil Engineering, Vol. 122, No.4, pp. 259-266

Heerten, G., et al, (1995) "On the long-term shear behavior of geosynthetic clay liners (GCLs) in capping sealing systems", Proceedings of an International GCL Symposium, Nuremberg, pp. 141-150

Koerner, G.R. and Te-Yang Soong, (1998) "Scatter in Geosynthetic Performance Testing and Its Effect of Three Design Examples", 12th GRI Conference on Lessons Learned from Geosynthetic Case Histories, December 1998, pp. 139-152

Koerner, R. M., (1996a) "Geosynthetic clay liners, part one: an overview", Geotechnical Fabric Report, May 1996, pp. 22-25

Koerner, R.M., et al, (1996b) "Current Status of the Cincinnati GCL Test Plots", Re-print from GRI-10 Conference of Field Performance of Geosynthetic Related Systems, December 1996

Olson, R.E., (1974) "Shearing Strength of Kaolinite, Illite and Montomorillonite," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 100, No. GT11, pp. 1215-1229

Prüfamt für Grundbau, Bodenmechanik und Felsmechanik, Tu München, (1995) Test Report "Prüfung der Verbundscherfestigkeit (Innere Scherfestigkeit)", (Testing of the Peel Strength (Internal Shear Strength) Munich, 1995

Richardson, G.N., (1997) "GCL internal shear strength requirements", Geotechnical Fabrics Report, March 1997, pp. 20-25

Stark, T.D. and Eid, H.T., (1996) "Shear Behavior of Reinforced Geosynthetic Clay Liners", Geosynthetics International, Vol. 3, No. 6, pp. 771-786

von Maubeuge, K.P., Eberle, M., (1998) "Can Geosynthetic Clay Liners be used on slopes to achieve long-term stability?", 3rd International Congress on Environmental Geotechnic, Lisbon, Portugal, Sept. 1998, pp. 375-380

von Maubeuge, K.P., Witte, J., (1998) " Installation and Monitoring Difficulties Using Geosynthetic Clay Liners as Canal Liners in Major Hydraulic Systems", 12th GRI Conference on Lessons Learned from Geosynthetic Case Histories, December 1998, pp. 205- 211

von Maubeuge, K.P., Heerten, G., (1994) "Needle-punched Geosynthetic Clay Liners (GCLs)" Proceedings of the 8th GRI Conference, Geosynthetic Resins, Formulations and Manufacturing, Philadelphia, pp. 199-207