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COMPILATION OF INFORMATION ON ALTERNATIVE BARRIERS FOR LINER AND COVER SYSTEMS

by

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16. ABSTRACT

Workshop was held in Cincinnati, Ohio to present and discuss alternative barriers for liner and cover systems. "Alternative" barriers include thin, manufactured, low-permeability materials that are being used and being proposed for use in liner and cover systems for landfills, waste impoundments, site remediation projects, secondary containment structures, and other facilities. The materials are being considered as an extra component of a liner or cover system, e.g., to back up a flexible membrane line (FML), and in other cases, as a substitute for a thicker layer of compacted, low-permeability soil.

This report contains a compilation of available information concerning alternative barrier materials and summarizes the main points brought out in the workshop. There are four main alternative barrier materials currently being produced. Three of them consist of a thin layer of bentonite sandwiched between two geotextiles, and the fourth consists of a thin layer of bentonite glued to an FML. All of the materials appear to have a very low hydraulic conductivity (between 1×10^{-10} cm/s and 1×10^{-8} cm/s, depending upon the conditions of testing). All of the materials are seamed in the field by overlapping sheets of hydrates. Data on the hydraulic integrity of the seams are much less complete compared to data on the materials themselves. The expansive nature of bentonite provides the bentonitic blankets with the capability of self-healing small punctures, cracks, or other defects. The materials have many advantages, including fast installation with light-weight equipment. The most serious shortcomings are a lack of data, particularly on field performance, and the low shear strength of bentonite.

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FOREWORD

Today's rapidly developing and changing technologies and industrial products and practices frequently carry with them the increased generation of materials that, if improperly dealt with, can threaten both public health and the environment. The United States Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. These laws direct the U.S. EPA to perform research to define our environmental problems, measure the impacts, and search for solutions.

The Risk Reduction Engineering Laboratory is responsible for planning, implementing, and managing research, development, and demonstration programs to provide an authoritative, defensible engineering basis in support of the policies, programs, and regulations of the U.S. EPA with respect to drinking water, wastewater, pesticides, toxic substances, solid and hazardous wastes, and Superfund-related activities. This publication is one of the products of that research and provides a vital communication link between the researcher and the user community.

This report documents the available information concerning manufactured materials that might be utilized in liner and cover systems for landfills, impoundments, site remediation projects, and secondary containment structures. The information compiled in this report was obtained from literature, from information supplied by manufacturers, and from discussions at a 2-day workshop held on June 7 and 8 in Cincinnati. This report will be useful to scientists, engineers, and regulatory staff who are considering use of these types of materials.

E. Timothy Oppelt
Director
Risk Reduction Engineering Laboratory

ABSTRACT

On June 7-8, 1990, a Workshop attended by approximately 75 people was held in Cincinnati, Ohio, to present and discuss alternative barriers for liner and cover systems. Alternative barriers include thin, manufactured, low-permeability materials that are being used and being proposed for use in liner and cover systems for landfills, waste impoundments, site remediation projects, secondary containment structures, and other facilities. In some cases, the materials are being considered as an extra component of a liner or cover system, e.g., to back up a flexible membrane liner (FML), and in other cases the alternative barriers are being considered as a substitute for a thicker layer of compacted, low-permeability soil.

This report contains a compilation of information available concerning alternative barrier materials and summarizes the main points brought out in the workshop. There are four main alternative barrier materials currently being produced. Three of them consist of a thin layer of bentonite sandwiched between two geotextiles, and the fourth consists of a thin layer of bentonite glued to an FML. All of the materials appear to have a very low hydraulic conductivity to water (between 1×10^{-10} cm/s and 1×10^{-8} cm/s, depending upon the conditions of testing). All of the materials are seamed in the field by overlapping sheets of the material and relying upon the bentonite to form its own seal when it hydrates. Data on the hydraulic integrity of the seams are much less complete compared to data on the materials themselves. The expansive nature of bentonite provides the bentonitic blankets with the capability of self-healing small punctures, cracks, or other defects. The materials have many advantages, including fast installation with light-weight equipment. The most serious shortcomings are a lack of data, particularly on field performance, and the low shear strength of bentonite.

The advantages of alternative barrier materials are significant, and the materials warrant further evaluation.

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Section 1

Purpose of Workshop

A 3-ft-(0.9 m) thick layer of low-permeability, compacted soil is a required component of secondary liners for hazardous waste landfills and surface impoundments regulated under the Hazardous and Solid Waste Amendments (HSWA) to the Resource Conservation and Recovery Act (RCRA) [EPA, 1985]. The minimum primary liner for such facilities must consist of a flexible membrane liner (FML). In addition, a secondary leachate collection, detection, and recovery system (LCDRS) must be placed between the two liners beneath hazardous waste landfills and surface impoundments, and, for solid-waste landfills, a primary leachate collection and removal system (LCRS) must overlie the uppermost liner. The minimum required components of a RCRA hazardous waste landfill liner system are sketched in Fig. 1.1.

The recommended designs for cover systems over RCRA hazardous waste landfills and closed surface impoundments include a 60-cm-thick layer of low-permeability, compacted soil (EPA, 1989). A typical recommended design profile for a cover system is shown in Fig. 1.2.

Non-hazardous solid wastes are also regulated under RCRA, but requirements have yet to be published by the EPA. Presently, the states are establishing requirements for liner and cover systems for non-hazardous waste landfills. Requirements vary, but most minimum design requirements are similar to the concepts shown in Figs. 1.1 and 1.2.

No minimum design requirements for final covers over Superfund sites have been established. Typically, however, some type of control of water infiltration is included in the final cover design. Typically, a layer of low-permeability, compacted soil is part of the cover design.

Thus, a layer of low-permeability, compacted soil is either a required or recommended component of most liner systems for hazardous and non-hazardous waste landfills and surface impoundments, as well as final covers over buried wastes or contaminated soil. Program and regional officials of the U.S. Environmental Protection Agency (EPA) are currently evaluating requests to substitute thin, manufactured clay blankets (alternative barriers) for thicker, low-permeability, compacted soil in liners and covers. Representatives of EPA, as well as state regulatory personnel and design engineers, need to be aware of the advantages and disadvantages of the alternative barrier materials and need to have access to the full breadth of available information about the various materials.

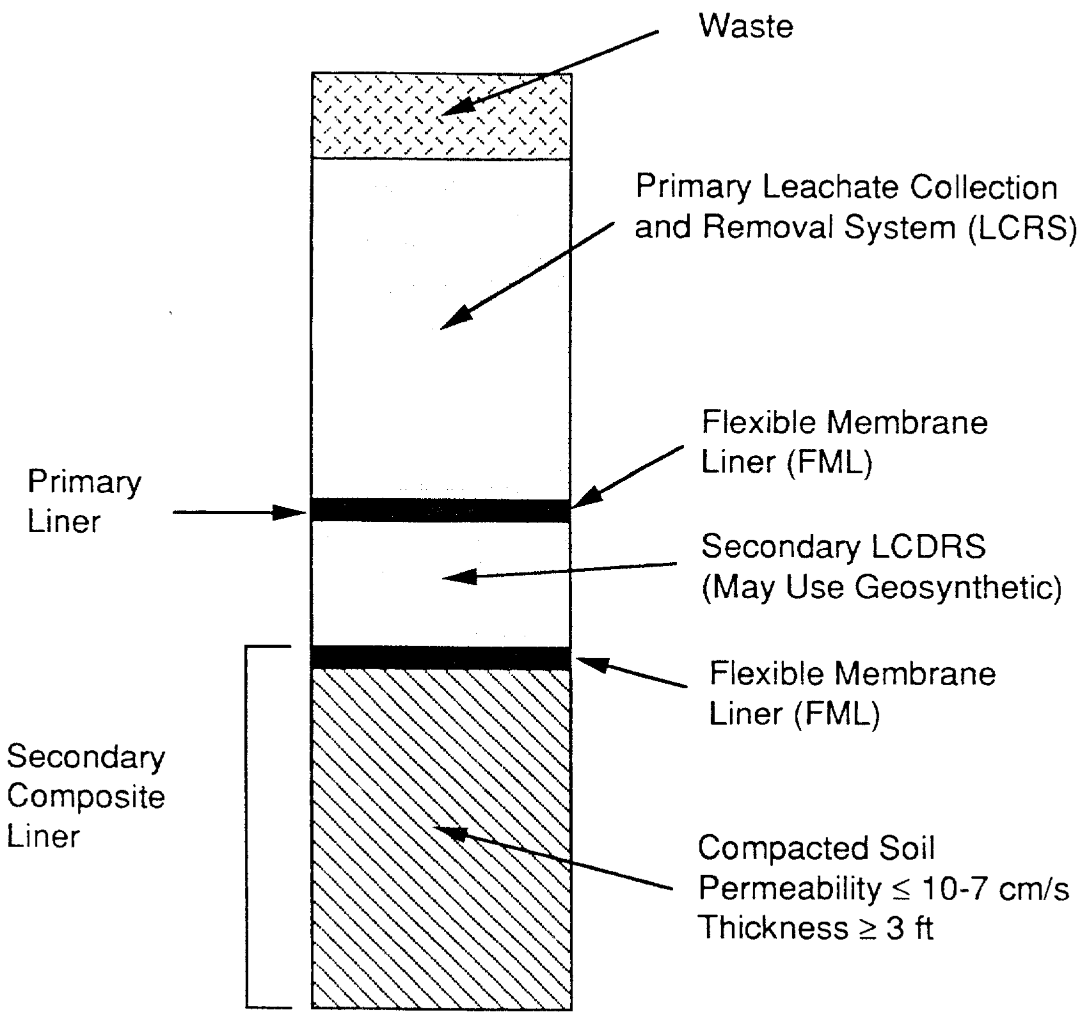


Figure 1.1. Minimum Requirements for Liner Systems for Hazardous Waste Landfills and Surface Impoundments (from EPA, 1985).

EPA-Recommended Cover Design
for Hazardous Wastes

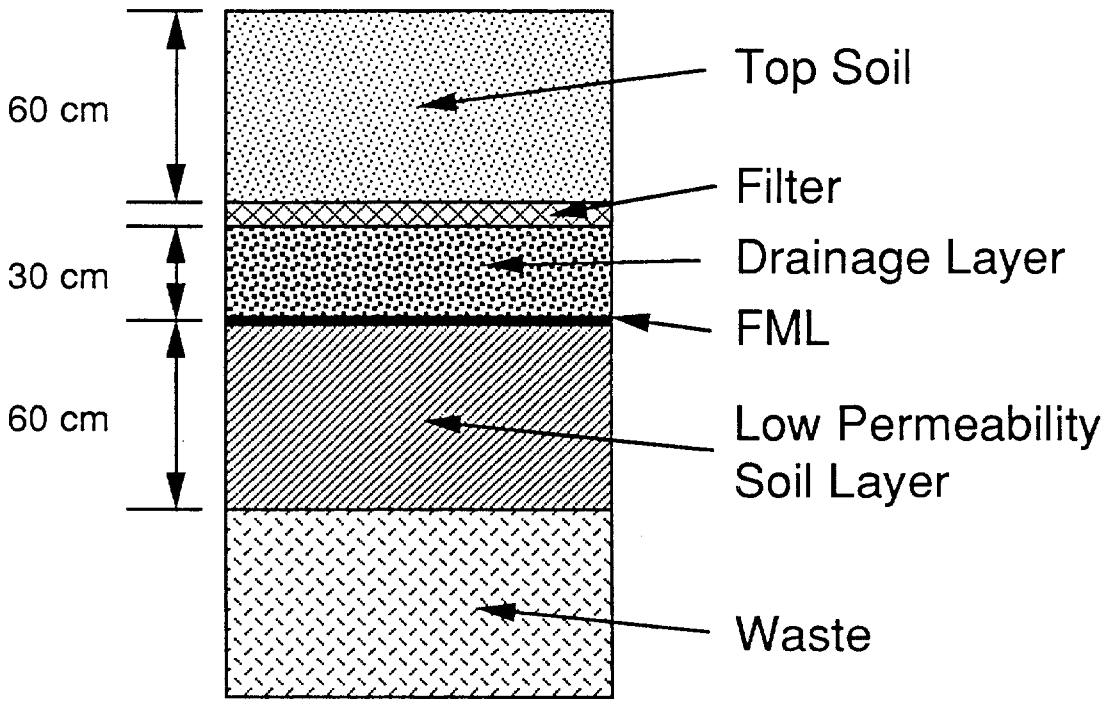


Figure 1.2. Recommended Design for Cover System for Hazardous Waste Landfills and Surface Impoundments (from EPA, 1989).

To disseminate information, a workshop was held on June 7 and 8, 1990, at the EPA's Risk Reduction Engineering Laboratory (RREL) in Cincinnati, Ohio. The purpose of the workshop was: (1) to present to EPA technical staff and contractors, as well as state regulatory officials, the latest available information concerning alternative barriers; and (2) to exchange ideas that might prove useful in making research on alternative barriers consistent with ongoing, parallel studies and responsive to the needs of permit writers and regulation developers.

The specific topics discussed at the workshop were as follows:

1. Presentation of background information on conventional, low-permeability, compacted soil liners; functions served by compacted soil; performance of compacted soil; factors to be considered in judging equivalency of other barrier materials (presented by D. E. Daniel).
2. Description of alternative barriers (presented by Bentomat®, Claymax®, and Gundseal representatives).
3. Discussion of parallel studies; results of various experiments performed on compacted soil and manufactured barrier materials (presented by D. E. Daniel, P. M. Estornell, and T. Zimmie).
4. Discussion of technical and regulatory concerns of EPA program offices, regional offices, and state regulatory agencies about alternative barriers (open discussion).
5. Discussion of the prospects of using alternative barrier materials in liners and covers for hazardous waste landfills, municipal solid waste landfills, and Superfund closure sites; discussion of case studies (open discussion).
6. Discussion of research required to address the needs of permit writers and regulation developers (open discussion).

This report will provide not only a summary of the proceedings of the Alternative Barriers Workshop but will also document results from experiments recently conducted on alternative barrier materials.

Background information on compacted soil barriers is provided in Section 2 of this report. Information about Bentomat®, Claymax®, Gundseal, and Bentofix is presented in Sections 3 through 6. Other alternative barriers are discussed in Section 7. The equivalency of the alternative barrier materials is addressed in Section 8. Concerns about the alternative barriers are summarized in Section 9. Research needs identified during the workshop are listed in Section 10. A list of attendees is presented in the Appendix.

Section 2

Compacted Soil Liners

Compacted soil liners are constructed primarily from naturally-occurring, low-permeability soils, although the liner may contain processed materials such bentonite or even synthetic materials such as polymers. Soil liners usually contain significant quantities of clay and thus are frequently called "clay liners" even though clay may not be the most abundant constituent in the liner material. Compacted soil liners are constructed in layers, called "lifts," that are typically about 9 in. (225 mm) in loose thickness and 6 in. (150 mm) in compacted thickness. Heavy compactors, or "rollers", are used to compact the soil.

2.1 Materials

The minimum requirements recommended by Daniel (1990) for most low-permeability, compacted liners constructed from naturally-occurring soils are as follows:

Percentage Fines:	$\geq$ 30%
Plasticity Index:	$\geq$ 10%
Percentage Gravel:	$\leq$ 10%
Maximum Particle Size:	1 to 2 in. (25 to 50 mm)

Percentage fines is defined as the percent by dry weight passing the No. 200 sieve, which has openings of 75 μm. Percentage gravel is defined as the percent by dry weight retained on a No. 4 sieve (4.76 mm openings). Local experience may dictate more stringent requirements, and, for some soils, more restrictive criteria may be appropriate.

If suitable materials are unavailable locally, local soils can be blended with commercial clays, e.g., bentonite, to achieve a low hydraulic conductivity. However, bentonite can be attacked by some leachates -- compatibility tests may be required. A relatively small amount of bentonite can lower hydraulic conductivity by several orders of magnitude (Daniel, 1987).

One should be cautious about using highly plastic soils (soils with plasticity indices >30 to 40%) because these materials form hard clods when the soil is dry and are very sticky when the soil is wet. Highly plastic soils, for these reasons, are difficult to work with in the field. However, special techniques, such as addition of lime, can ameliorate some of the problems with construction utilizing highly plastic soils so that even these soils may be useable.

2.2 Important Variables

Experience has shown that the water content of the soil, method of compaction, compactive energy, clod size, and degree of bonding between lifts of soil can have a significant influence on the hydraulic conductivity of compacted soil liners.

The water content of the soil at the time of compaction ("molding water content") influences the hydraulic conductivity of saturated soil as shown in Fig. 2.1. When soils are mixed to different water contents and then compacted, the dry unit weight is found to be maximum at a certain molding water content, which is called the "optimum water content" (dashed line in Fig. 2.1). Hydraulic conductivity is usually minimum for soils compacted at molding water contents greater than the optimum. Experience has shown that the primary causes for differences in hydraulic conductivity are differences in the arrangement of soil particles (Mitchell, Hooper, and Campanella, 1965) and the fate of clods of clayey soil (Benson and Daniel, 1990). With dry soils, the clods of soil are hard and difficult to remold. When the soil is wetted to water contents higher than optimum, the clods are soft and more easily remolded into a homogeneous mass that is free of large pores between clods. Thus, it is important that the water content of the liner material be carefully controlled; otherwise, undesirably large hydraulic conductivity may result, especially if the soil is too dry when it is placed and compacted.

The method of compaction can influence the hydraulic conductivity of compacted soil. Laboratory studies have shown that kneading the soil during compaction minimizes the hydraulic conductivity (Fig. 2.2). Thus, footed rollers are typically utilized to compact soils in the field; the "feet" from the drum of the roller penetrate into the soil to knead the soil during compaction.

The energy of compaction is also an important variable. As shown in Fig. 2.3, the larger the amount of energy delivered to the soil, the lower the hydraulic conductivity. In the field, it is important to make an adequate number of passes of a heavy roller, and not to use too thick a lift, to ensure that adequate compactive energy is delivered to the soil. The minimum weight and number of passes varies with soil and equipment (Daniel, 1987; Herrmann and Elsbury, 1987; and Daniel, 1990).

The size of clods of soil can also influence hydraulic conductivity. Benson and Daniel (1990) found that pulverization of clods of soil lowered the hydraulic conductivity of one highly-plastic soil by a factor of 10,000 when the soil was compacted dry of optimum water content. For wet soil with soft clods, the size of clods had little effect. For dry, hard soils, such a shales, mudstones, or dry, highly-plastic soils, preprocessing the material with mechanical pulverization may be required.

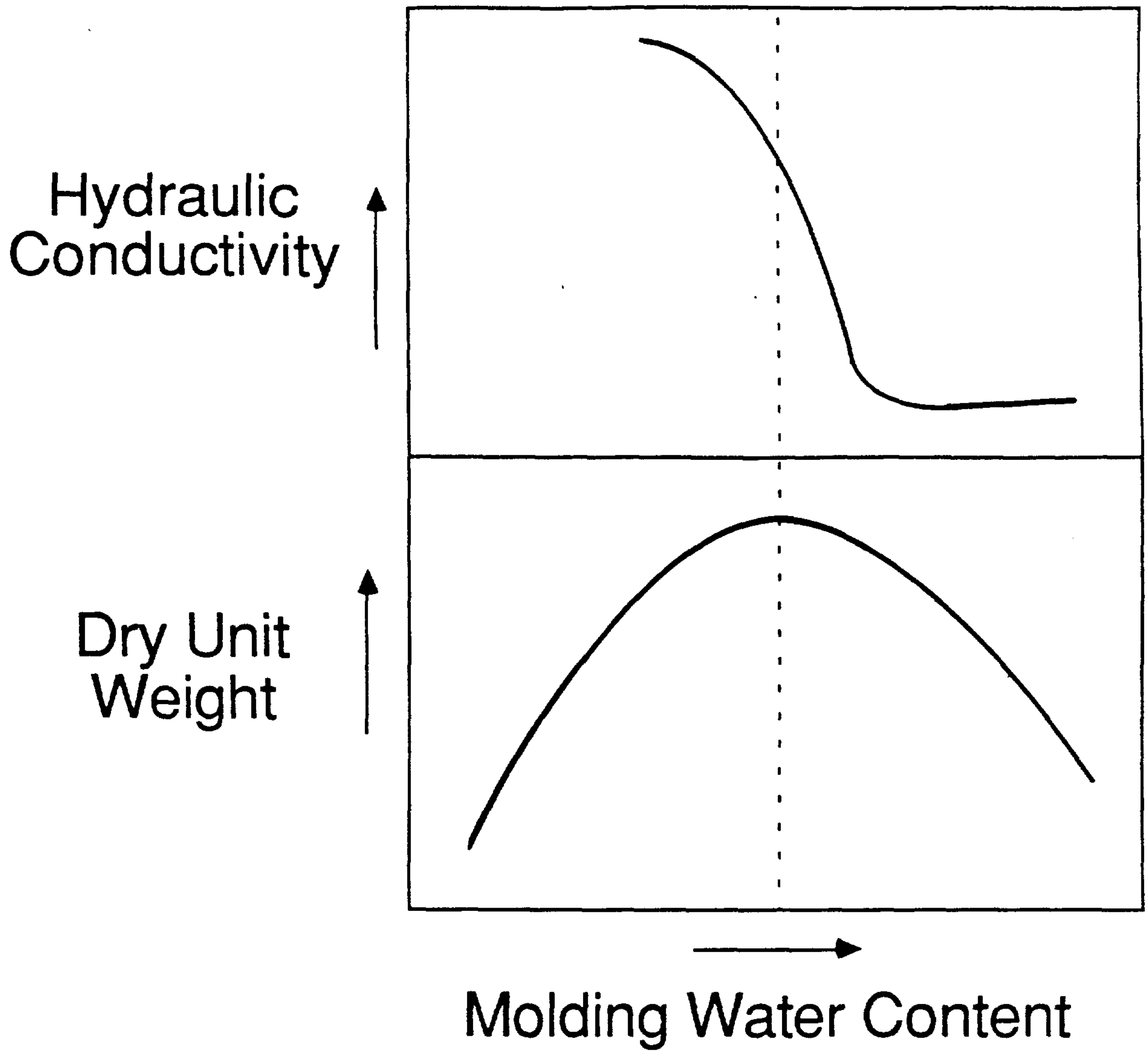


Figure 2.1. Dry Unit Weight and Hydraulic Conductivity Versus Molding Water Content for Typical Compacted, Low-permeability Soils.

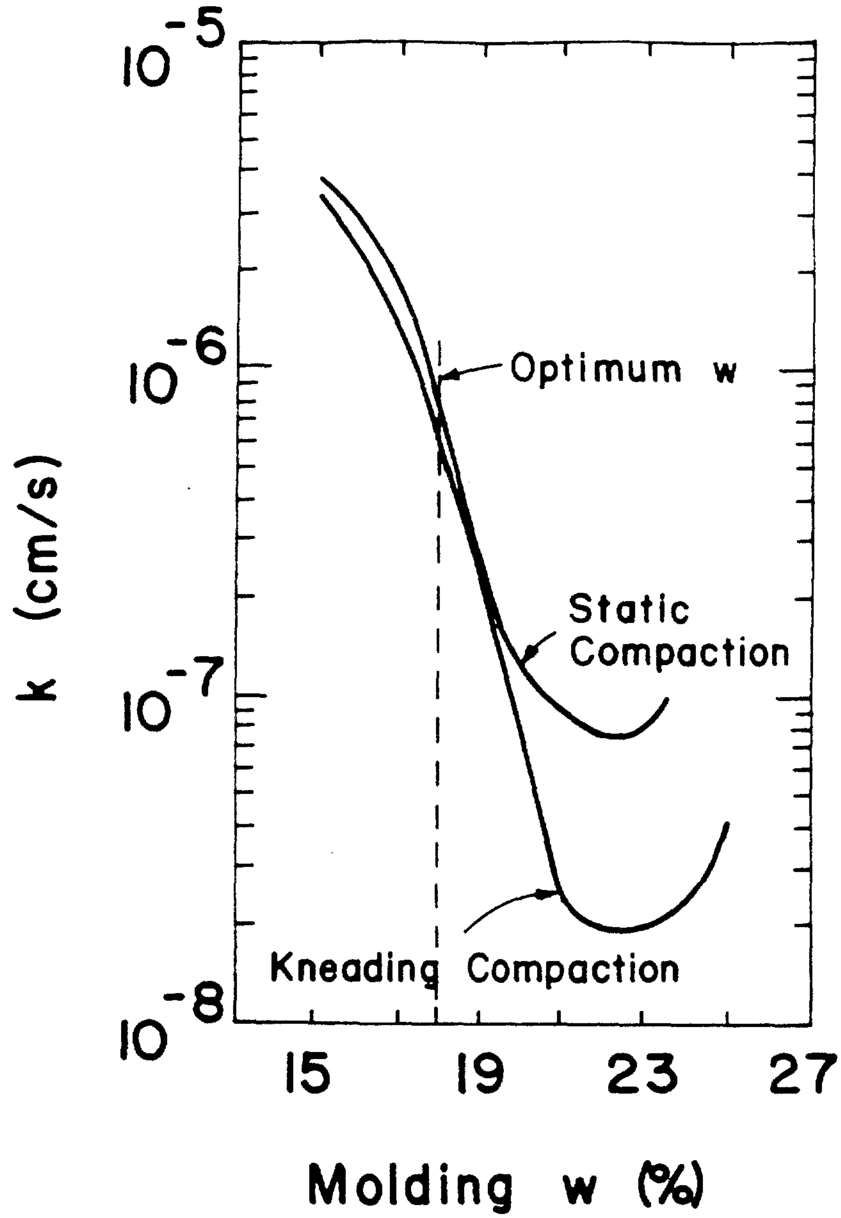


Figure 2.2. Effect of Method of Compaction Upon Hydraulic Conductivity of a Silty Clay Soil (from Mitchell, Hooper, and Campanella, 1965).

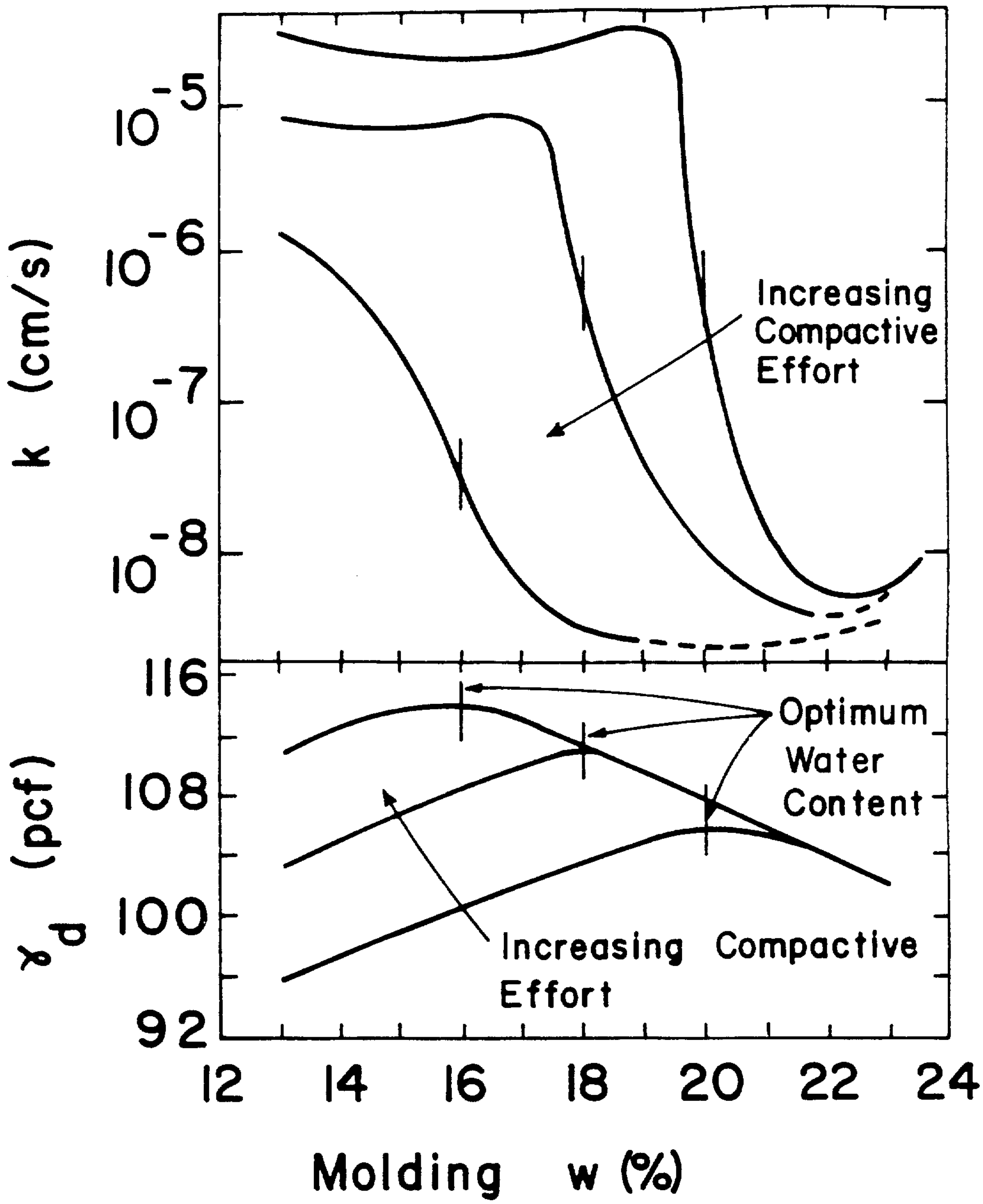


Figure 2.3. Effect of Compactive Energy Upon Hydraulic Conductivity of a Silty Clay Soil (from Mitchell, Hooper, and Campanella, 1965).

Experience has also demonstrated that lifts of soil must be bonded together to minimize highly permeable zones at lift interfaces. The problem is illustrated in Fig. 2.4 for a liner composed of 4 to 6 lifts. If each lift contains occasional hydraulic defects, liquid will permeate primarily through those defects. If there is a highly permeable inter-lift zone, liquid can spread laterally along the inter-lift zone until a hydraulic defect in the underlying lift is reached. Thus, permeable inter-lift zones provide hydraulic connection between the more permeable zones within adjacent lifts. If permeable inter-lift zones are eliminated, hydraulic connection between "defects" in each lift is destroyed and a lower overall hydraulic conductivity can be achieved. To maximize bonding between lifts, the surface of a previously-compacted lift is roughened ("scarified"), and the new lift of soil is compacted with rollers that have feet that fully penetrate the loose lift (to compact the new lift of soil into the surface of the previous lift).

2.3 Construction of Compacted Soil Liners

2.3.1 Processing of Soil

Some liner materials need to be processed to break down clods of soil, to sieve out stones and rocks, to moisten the soil, or to incorporate additives such as bentonite. Clods of soil can be broken down with mechanical tilling equipment such as a rototiller. Stones can be sieved out of the soil with large vibratory sieves or mechanized "rock pickers" passed over a loose lift of soil. Road reclaimers (also called road recyclers) can process soil in a loose lift and crush stones or large clods.

If the soil must be wetted or dried more than about 2 to 3 percentage points in water content, the soil should first be spread in a loose lift about 12 in. (300 mm) thick. Water can be added and mixed into the soil with agricultural tillage equipment or industrial mixers, or the soil can be disced or tilled to allow it to dry uniformly. It is essential that time be allowed for the soil to wet or dry uniformly. At least 1-3 days is usually needed for adequate hydration or dehydration. Frozen soil should never be used to construct a soil liner.

Additives such as bentonite can be introduced in two ways. One technique is to mix soil and additive in a pugmill. Water can also be added in the pugmill. Alternatively, the soil can be spread in a loose lift that is 9 to 12 in. (222 - 300 mm) thick, the additive spread over the surface, and a mechanical tiller or road reclaimer used to mix the materials. Several passes of the mixer over a given spot may be needed, and the mixer should be operated in at least 2 different directions to minimize the possibility of strips of unmixed material. Water can be added in the tiller during mixing or later, after mixing is complete. The pugmill is more

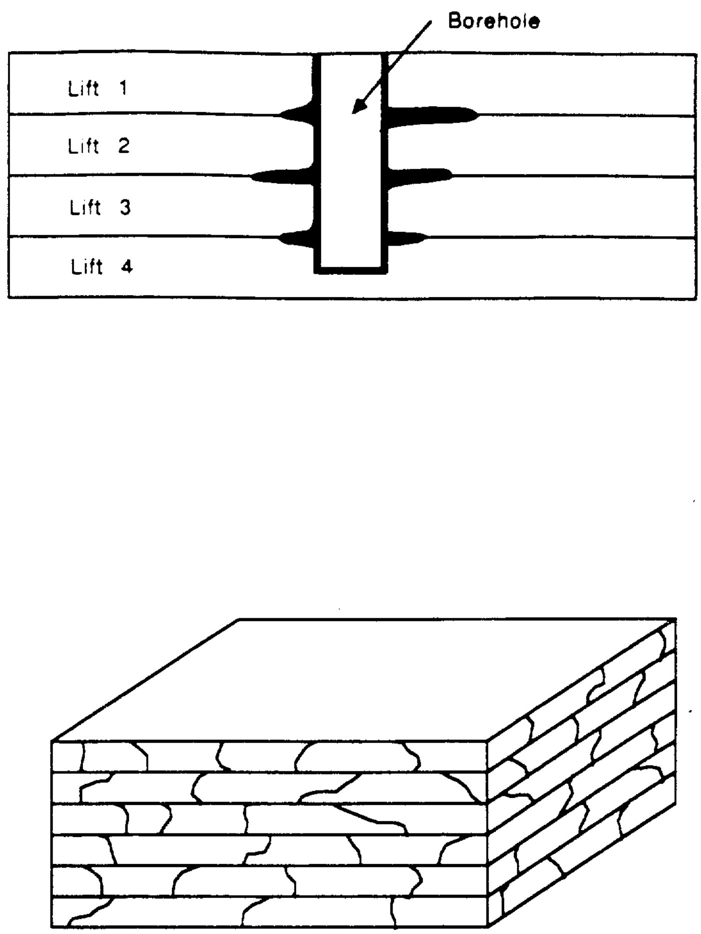


Figure 2.4. Permeable Inter-Lift Zones Providing Hydraulic Connection between High-Hydraulic-Conductivity Zones in Adjacent Lifts.

reliable in providing thorough mixing, but, done carefully, in-field methods can provide effective mixing.

2.3.2 Surface Preparation

It is crucial that each lift of a soil liner be effectively bonded to the overlying and underlying lifts. The surface of a previously-compacted lift must be rough rather than smooth. If the surface has been smoothed, e.g., with a finish roller at the end of a day's work shift, the surface should be excavated to a depth of about 1 in. (25 mm) with a disc or other suitable device before continuing placement of overlying lifts.

2.3.3 Placement

Soil is placed in a loose lift that is no thicker than about 9 in. (225 mm). If grade stakes are used to gauge thickness, the stakes must be removed and the holes left by the stakes sealed. Techniques that do not require penetration of the lift, e.g., laser controls, are preferable to grade stakes. After the soil is placed, a small amount of water may need to be added to offset evaporative losses, and the soil may be tilled one last time prior to compaction.

2.3.4 Compaction

Heavy, footed compactors with feet that fully penetrate a loose lift of soil are ideal. The weight of the compactor must be compatible with the soil: relatively dry soils with firm clods require a very heavy compactor whereas relatively wet soils with soft clods require a roller that is not so heavy that it becomes bogged down in the soil. Care should be taken to ensure that an adequate number of passes of the roller are made. Normal compaction specifications typically require 6 to 8 passes of a roller to achieve the required density. Since the soil liner is being build as a hydraulic containment structure, it is necessary to apply sufficient number of passes that every portion of compacted soil receives the compactive energy applied by the feet on the roller. The footprint area and the number of feet on the roller drum need to be taken into account to calculate the minimum number of passes required for complete coverage of an area. Additional passes beyond the theoretical minimum needed for 100 percent coverage should be provided to account for the footprint overlap likely to occur in field construction. Experience has shown that as many as 18 to 20 passes are required for some types of footed rollers to achieve complete coverage. Since a kneading compaction helps to provide minimal hydraulic conductivity, it is fallacious to use the common "walking out" endpoint to indicate that sufficient compaction has been achieved. Experience indicates that minimum hydraulic conductivity has been achieved while there remains some "waving" of the reworked soil ahead of the roller drum.

"Walking out" needs to be monitored carefully as it may indicate that the soil is too dry to achieve hydraulic conductivity objectives.

2.3.5 Protection

After compaction of a lift, the soil must be protected from desiccation and freezing. Desiccation can be minimized in several ways: the lift can be temporarily covered with a sheet of plastic, the surface can be smooth-rolled to form a relatively impermeable layer at the surface, or the soil can be periodically moistened. The compacted lift can be protected from damage by frost by avoiding construction in freezing weather or by temporarily covering the lift with an insulating layer of material.

2.3.6 Quality Control Tests

A critical component in construction quality assurance is quality control (QC) testing. For soil liners, the tests fall into two categories: (1) tests to verify that the materials of construction are adequate, and (2) tests and observation to verify that the compaction process is adequate. Great care must be taken to design an adequate program of QC testing and to repair holes left from destructive QC tests. Details on QC testing are given by EPA (1986), Goldman et al. (1988), and Daniel (1990).

2.3.7 Summary

Proper construction of soil liners is difficult. Materials must be carefully selected, the soil may require extensive processing, the moisture content must be in the correct range, the surface to receive a lift of soil must be prepared properly, the soil must be adequately compacted, and each compacted lift as well as the entire liner must be protected from damage caused by desiccation or freezing temperatures. Further information is provided by Daniel (1987, 1990), Herrmann and Elsbury (1987), and EPA (1986, 1989).

2.4 Test Pads

The construction of a test pad prior to building a full-sized liner has many advantages. By constructing a test pad, one can experiment with molding water content, construction equipment, number of passes of the equipment, lift thickness, and other construction variables. Most importantly, though, one can conduct extensive field-scale destructive testing, including QC testing and in-situ hydraulic conductivity testing, on the test pad. Test pads are recommended by the EPA (1985) for confirming that the materials and methods of construction will provide an adequately low hydraulic conductivity for the soil-liner component in RCRA hazardous waste landfills and surface impoundments.

The test pad usually has a width of at least 3 construction vehicles (>10 m), and an equal or greater length. The pad should ideally be the same thickness as the full-sized liner, but the test pad may be thinner than the full-sized liner. (The full-thickness liner should perform at least as well as, and probably better than, a thinner test section because defects in any one lift become less important as the number of lifts increases). The in-situ hydraulic conductivity may be determined in many ways, the large sealed double-ring infiltrometer is usually the best large-scale test (Daniel, 1989).

2.5 Chemical Compatibility

The compatibility of low-permeability soil liners with wastes to be retained must be assured. Daniel (1987) and Goldman et al. (1988) discuss the mechanisms of attack and summarize available data.

2.6 Reliability of Soil Liners

Examples can be cited of soil liners that had unacceptably large hydraulic conductivity and therefore failed to function effectively as hydraulic barriers (Daniel, 1987; and Goldman et al., 1988). Inadequate construction or construction quality control have been the main causes of problems. Good-quality soil liners can be constructed (Gordon et al., 1989) if construction is carried out very carefully and adequate construction quality is applied.

Section 3

Bentomat®

3.1 Description

Bentomat® is manufactured by American Colloid Company, 1500 West Shure Drive, Arlington Heights, Illinois 60004 (telephone 708-392-4600). The material consists of a minimum of one pound per square foot (4.9 kg/m^2) of dry (maximum 12% moisture), granular, sodium bentonite sandwiched between two polypropylene geotextiles (Fig. 3.1). The upper geotextile is woven while the lower geotextile is non-woven. The weights of the geotextiles can vary but are typically about 3 to 6 oz. per square yard (102 to 204 g/m^2). Fibers from the upper geotextile are needlepunched through the layer of sodium bentonite and into the lower geotextile (Fig 3.1). Variations of Bentomat® can be custom engineered to meet site-specific needs. Also, one of four basic types of sodium bentonite may be incorporated into Bentomat®. Each bentonite grade has different swelling properties and contaminant-resistant properties. The four bentonites available have the following designations and properties.

"CS"- CS-50 (untreated, granular bentonite)
"SG"- SG-40 (polymer-treated, high-swelling bentonite)
"PL"- PLS-50, (medium-containment-resistant bentonite)
"SS"- SS-100 (high-contaminant-resistant bentonite).

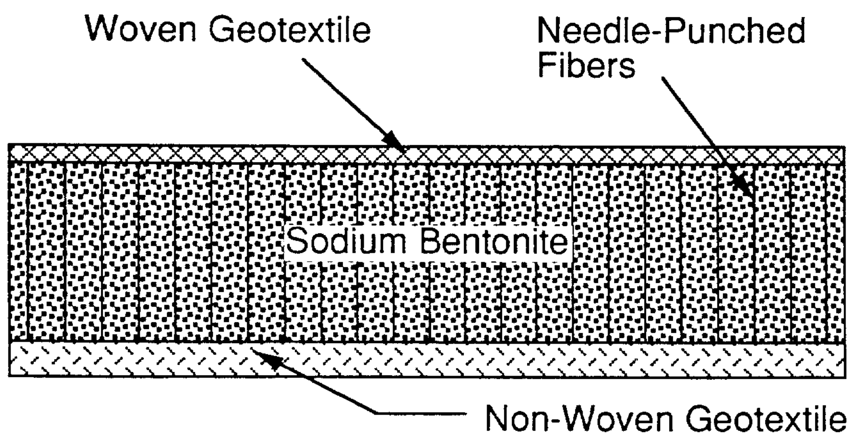


Figure. 3.1 Schematic Diagram of Bentomat®.

The standard roll size of Bentomat® is currently 12 ft (3.6 m) wide and 100 ft (30 m) long. The thickness of dry Bentomat® is approximately 1/4 in (6 mm).

Bemalux Inc., of Quebec, Canada, originated Bentomat® in 1980. The original Bentomat® was constructed in the field by laying a sheet of geotextile on a smooth surface, spreading a layer of sodium bentonite (about 3 lbs/ft^2, or 14.7 kg/m^2) over the geotextile, and covering the bentonite with another geotextile. The prefabricated, needlepunched version of Bentomat® was introduced in January, 1990, by American Colloid Company, which acquired the U.S. patent rights for the product from Bemalux, Inc., in 1989.

3.2 Installation

The following discussion summarizes the manufacturer's recommendations for installation. The subgrade should be compacted such that no rutting is caused by installation equipment or vehicles. Subgrade or fill material should be free of angular or sharp rocks larger than 1 inch in diameter. Organics or other deleterious materials should be removed. Prior to the placement of Bentomat®, the surface should be graded to fill all major voids and cracks.

Bentomat® is placed, beginning with the side slopes, by anchoring the panels in anchor trenches and then unrolling the material down the slope. Panels may also be pulled up from the bottom of the slope to the anchor trench. Seams at the base of the slope should be a minimum of 5 ft (1.5 m) away from the toe of the slope. Seams along the side slopes should be perpendicular to the toe of the slope. Panels on flat surfaces do not require any particular orientation.

Seams are formed by overlapping one panel on another. Seam overlaps should be a minimum of 6 in. (150 mm) wide with contacting surfaces that are flat and clear of any large rocks, dirt, or debris. The panels are printed with 6-in. and 9-in. (150 and 230 mm) guidelines along both edges to aid in assuring that the minimum overlap width is achieved. American Colloid Company recommends sprinkling granular bentonite at a rate of approximately 0.25 pounds per liner foot (35 g/m) over a 3-in. (76 mm) wide swath in the overlap zone. Fasteners, anchor pins, or adhesives may be used on seams to keep panels in place during backfilling operations.

For pipe penetrations, a small notch should be cut in the subbase around the circumference of the pipe. Bentonite should then be packed around the pipe in the area of the notch to form a thick bentonite seal. The Bentomat® panel should be slit with an "X" in the center, placed over the penetration, and sealed with bentonite to produce a seal. A second piece of Bentomat® should then be cut and fit around the pipe with bentonite applied between the overlap and to any gaps that may exist.

A 12-in. (300 mm) thick layer of protective soil should be placed over the Bentomat® liner, taking care to keep 12 in. (300 mm) of material between the liner and any machinery or equipment at all times. Sharp turns and quick stops or starts should be avoided to prevent pinching or moving the liner. When placing riprap on slopes, a layer of heavier geotextile should be incorporated into the liner for added puncture resistance (American Colloid Company, 1990).

3.3 Properties

3.3.1 Shear Strength

3.3.1.1 Direct Shear Tests

Direct shear tests were performed on soil/Bentomat® interfaces by J&L Testing Company (1990a). The frictional resistance between Bentomat® and sand and between Bentomat® and clay was measured in a direct shear device for both dry and hydrated samples. The tests were apparently designed to cause failure along the sand/Bentomat® or clay/Bentomat® interface and not to produce failure within the bentonite.

Samples of Bentomat® measuring 100 mm by 100 mm (3.9 in. by 3.9 in.) were placed against soil in a direct shear box and subjected to a constant rate of displacement of 0.009 in/min (0.24 mm/min). Normal stresses of 150, 300, and 450 psf (7.2, 14.4, and 21.5 kPa) were applied to each of the specimens. No standard method of testing these types of materials exists; apparatus of the type normally used for soils was apparently utilized. Failure occurred in most incidences at a horizontal displacement of approximately 0.2 in. (5 mm). The time to failure is calculated by the authors of this report to be about 20 minutes. It is doubtful that the rate of shear was slow enough to allow full dissipation of water pressures generated within hydrated clay or bentonite during shear. For this reason, the test results probably do not reflect the long-term performance of the materials or interfaces.

Results of direct shear tests are presented in Figs. 3.2 through 3.5 and are summarized in Table 3.1. The failure envelopes shown in Figs. 3.2 through 3.5 were calculated by linear regression. The calculated friction angles are between 28° and 41°.

For both the sand/Bentomat® and clay/Bentomat® tests, the friction angles were 7 to 10° higher when the bentonite was hydrated compared to dry bentonite. The authors of this report would have expected lower friction angles with hydrated bentonite, but the results of the tests were the opposite of this expectation. No explanation as to the cause for higher friction for hydrated versus dry bentonite is apparent, except that the tests may more nearly reflect short-

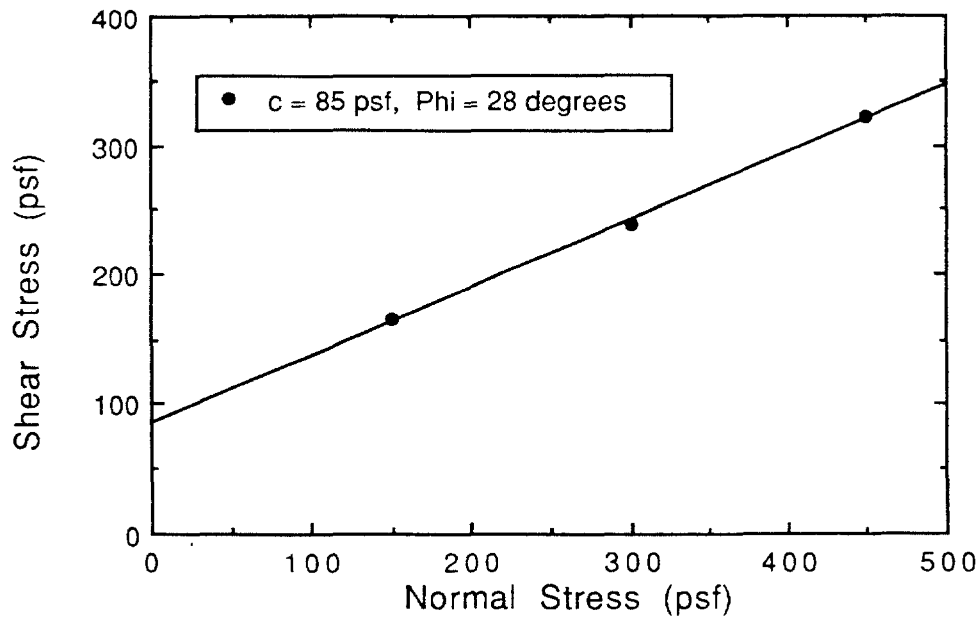


Figure 3.2. Results of Direct Shear Tests with Interfacial Shear Occurring between Dry Bentomat® and Sand (J & L Testing Company, 1990a).

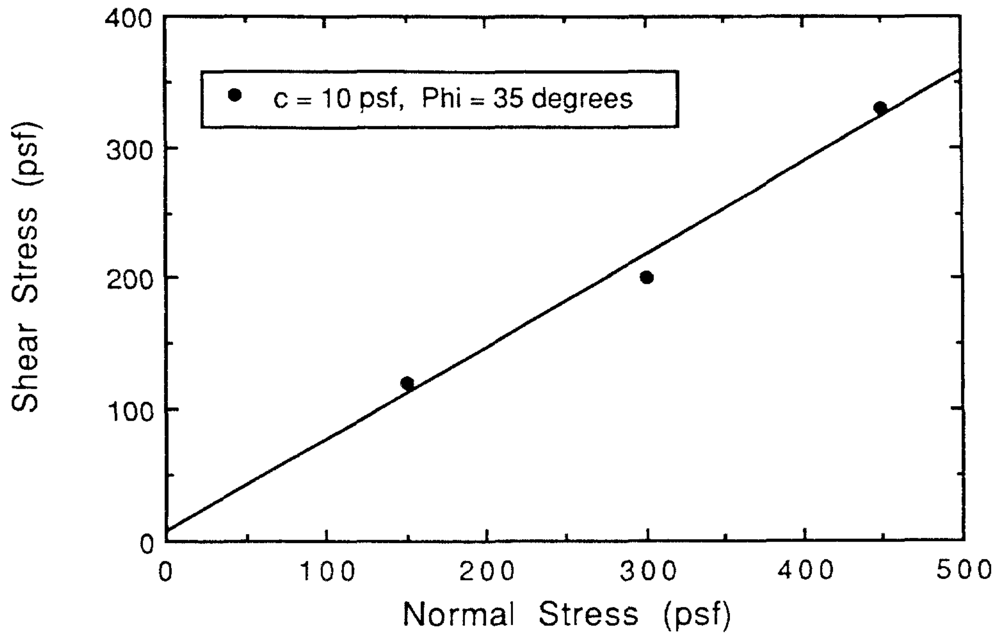


Figure 3.3. Results of Direct Shear Tests with Interfacial Shear Occurring between Hydrated Bentomat® and Sand (J & L Testing Company, 1990a).

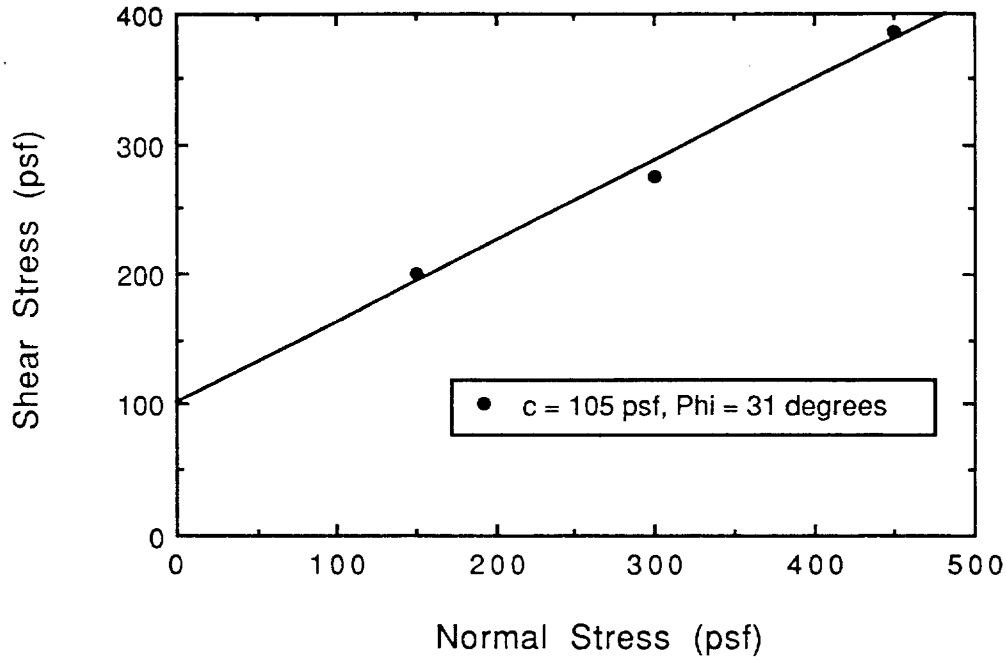


Figure 3.4. Results of Direct Shear Tests with Interfacial Shear Occurring between Dry Bentomat® and Clay (J & L Testing Company, 1990a).

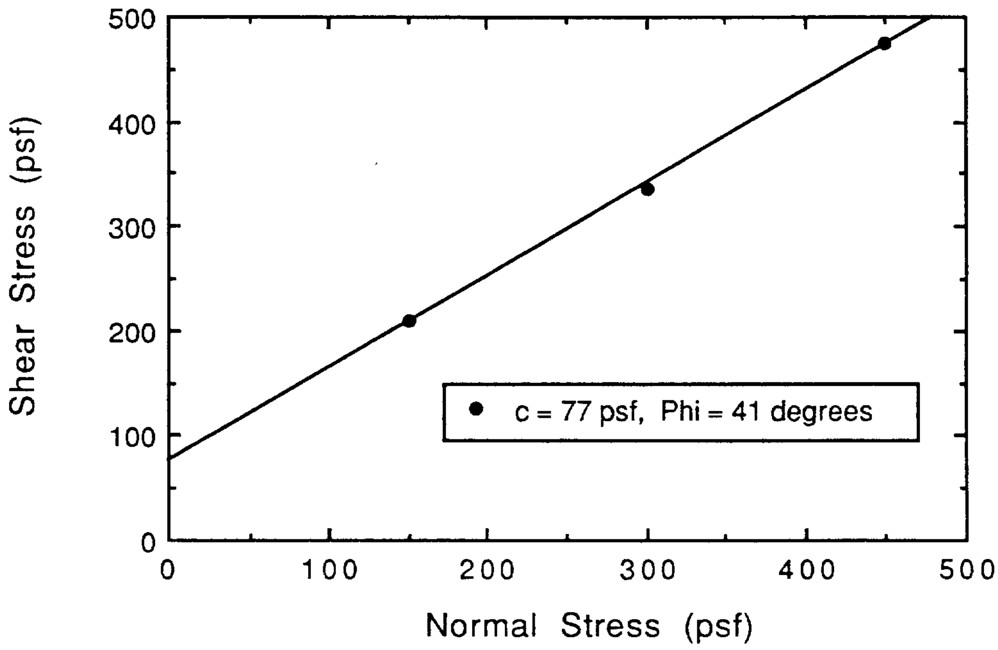


Figure 3.5. Results of Direct Shear Tests with Interfacial Shear Occurring between Hydrated Bentomat® and Clay (J & L Testing Company, 1990a).

term, undrained conditions rather than long-term, fully-drained conditions. These data are specific to the sand and clay soils tested. American Colloid Company recommends against extrapolating results to other soils; instead, site-specific testing is recommended.

Table 3.1 Summary of Results of Direct Shear Tests on Bentomat®

(J &L Testing Company, 1990a)

Sample	Cohesion (psf)	Friction Angle (degrees)
Dry Bentomat® with Sand	85	28
Hydrated Bentomat® with Sand	10	35
Dry Bentomat® with Clay	105	31
Hydrated Bentomat® with Clay	77	41

(Note: 100 psf = 4.8 kPa)

3.3.1.2 Tilt Table Tests

Tilt table tests were performed by J & L Testing Company (1990a). A multi-layered system composed of sand, high density polyethylene (HDPE) sheet, hydrated Bentomat®, and geonet was placed on a tilt table (Fig. 3.6) to measure the friction angle along the weakest interface (between smooth HDPE and hydrated Bentomat®). Normal stresses of 130 and 385 psf (6.2 and 18.4 kPa) were applied to the HDPE/Bentomat® interface, the table was inclined slowly, and the inclination at which sliding was first observed was recorded. No information on the time to failure was provided. Results of the tests are presented in Fig. 3.7. The friction angle between the smooth HDPE sheet and Bentomat® was 13.5°.

3.3.2 Hydraulic Properties

J & L Testing Company (1990b) conducted flexible-wall permeability tests on 6-in. (150-mm) diameter samples of Bentomat® containing either untreated granular bentonite

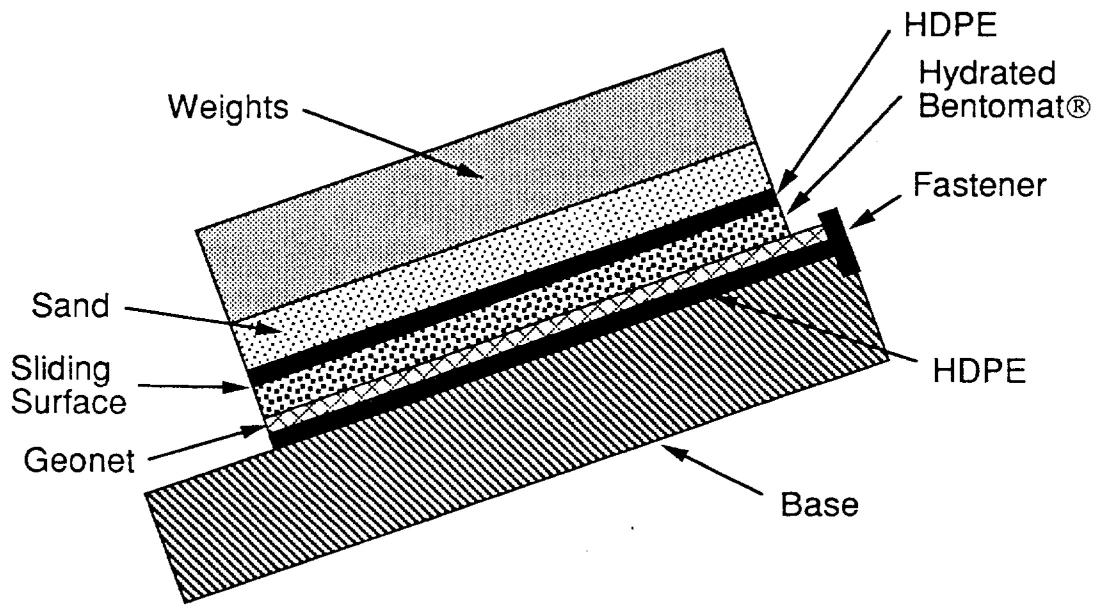


Figure 3.6. Schematic Diagram of Tilt Table Tests (J & L Testing Company, 1990a).

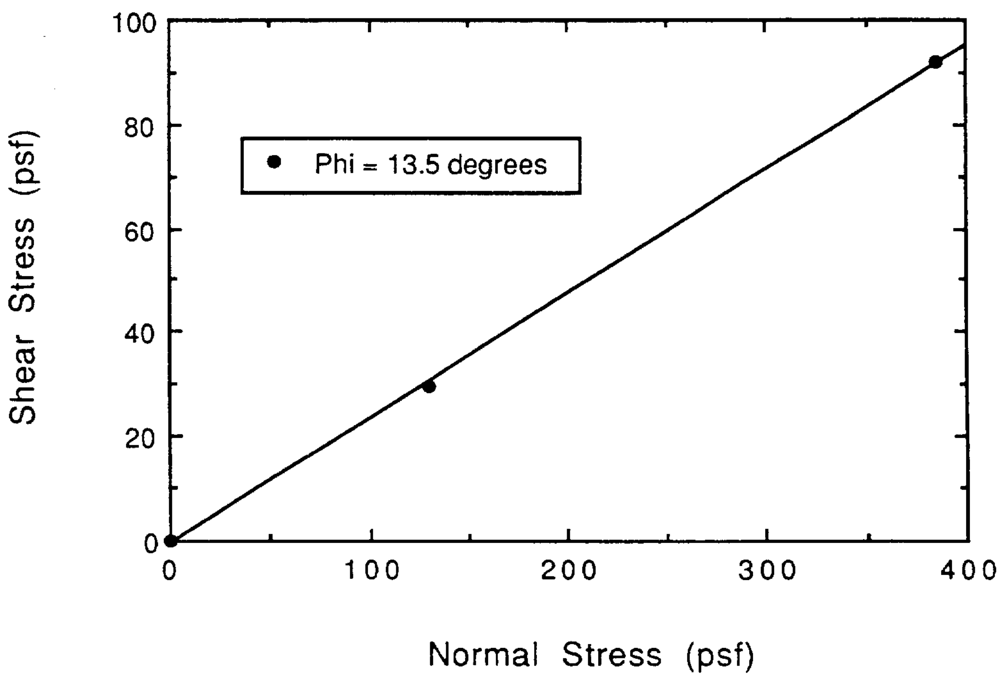


Figure 3.7. Results of Tilt Table Tests on Bentomat® (J & L Testing Company, 1990a).

("CS" grade) and high-contaminant-resistant bentonite ("SS" grade). Test conditions and results are summarized in Table 3.2. The duration of the tests was not reported. Figure 3.8 presents the relationship between hydraulic conductivity and maximum effective stress. Hydraulic conductivities ranged from 6×10^{-10} cm/s to 6×10^{-9} cm/s.

3.3.3 Seams

There have been no test results reported on the performance of Bentomat® seams. Bench-scale hydraulic conductivity tests on seam overlaps are in progress at the University of Texas at Austin, but no results were available at the time of this writing.

3.3.4 Mechanical Properties

Tests measuring grab strength, elongation, Mullen burst strength, wide width tensile strength and other mechanical properties of Bentomat® were conducted by J & L Testing Company, Inc. Tests were conducted according to ASTM standards, where available, on dry and hydrated Bentomat® as well as on the individual woven and nonwoven geotextile components of the liner material. Results of the test are summarized in Tables 3.3 through 3.6. Some slippage of the mat occurred on wide-width tensile testing. Tests will be repeated with modified grips.

3.4 Examples of Use

Bentomat® has been used in landfills, industrial and decorative lagoons, and as secondary containment liners in tank farms. However, because of its recent release in January, 1990, the applications of Bentomat® have been limited.

The largest Bentomat® installation to date in the U.S. was a lake liner for a residential development. A 9-acre (3.6 ha) lake was designed to be built in the midst of homes in the Cove on Herring Creek development in Delaware. Due to the existence of poor quality native soils, standing water, steep slopes and rough subgrade, the developer selected Bentomat® to line the lake. The liner was placed through water and over soft subgrade by placing each panel and then immediately following with a backhoe to place a foot of protective soil over the installed liner. The lake was filled in June, 1990.

A contaminant-resistant grade of Bentomat® ("PL" bentonite, which contains a polymer) was installed as a secondary containment barrier for petroleum tanks at a site in Oklahoma. Approximately 8400 ft^2 (780 m^2) of liner material was installed. Lysimeters were placed prior to all installations and the impoundments were flooded prior to their use to

Table 3.2 Summary of Results of Hydraulic Conductivity (K) Tests on Bentomat®
(J&L Testing Company, 1990b)

Grade of Bentonite	Stress (psi) Cell	Headwater	Tailwater	Maximum Effective	Hydraulic Conductivity (cm/s)
High-Contaminant-Resistant ("SS")	50	42.2	41.8	8.2	2.1×10^{-9}
	50	44.6	39.4	10.6	7.5×10^{-10}
	50	47.2	36.8	13.2	5.8×10^{-10}
Untreated Granular Bentonite ("CS")	50	42.2	41.8	8.2	5.6×10^{-9}
	50	44.6	39.4	10.6	1.1×10^{-9}
	50	47.2	36.8	13.2	9.8×10^{-10}

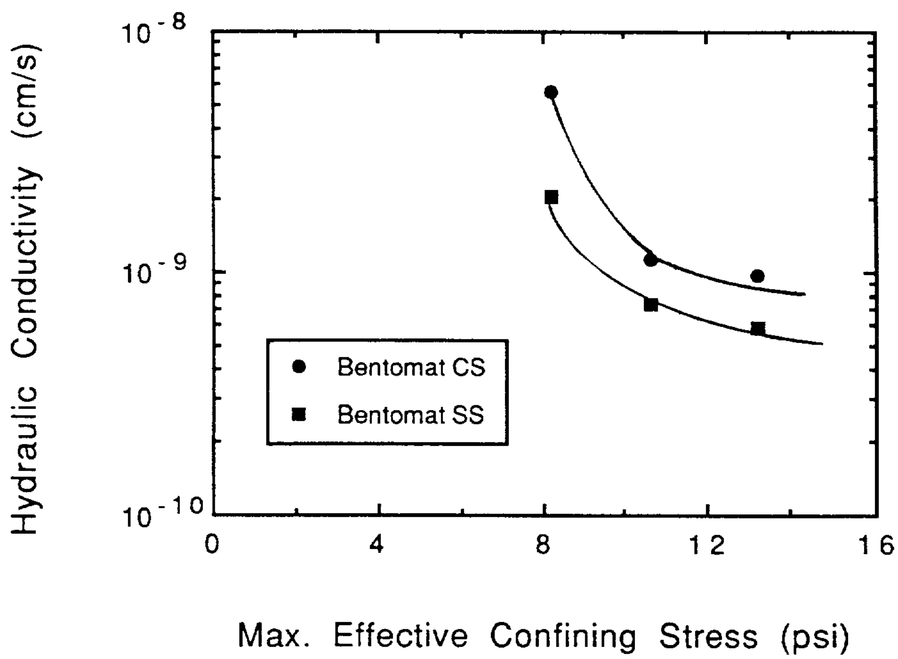


Figure 3.8. Results of Flexible-Wall Hydraulic Conductivity Tests on Bentomat® (J & L Testing Company, 1990b).

both activate the bentonite and check for defects. No leaks were apparent through the Bentomat® during this test.

Bentomat® has been installed in only one municipal landfill to date. A berm that was built across the center of a large landfill cell was lined with 20,000 ft^2 (1900 m^2) of Bentomat®. To insure the liner material would seal against an HDPE liner, granular bentonite was used at the HDPE/Bentomat® interface. The "PL" contaminant-resistant grade of Bentomat® was used on this project.

Table 3.3 Physical Property Test Results: Dry Bentomat® Containing High-Contaminant-Resistant Bentonite (J&L Testing Company, 1990a)

TEST	ASTM	UNITS	REPLICATE NO. 1	2	3	4	5	AVERAGE	STD DEV
GRAB STRENGTH	D-4632	lbs							
MD-Initial Peak			96.9	79.4	90.3	92.6	99.0	91.64	6.846
MD-Secondary Peak			125.1	N/A	96.7	134.9	100.9	114.40	16.049
GRAB ELONG.	D-4632	%							
MD-Initial Peak			9.3	12.0	11.7	15.0	14.7	12.54	2.108
MD-Secondary Peak			102.7	N/A	125.7	141.7	110.0	120.03	15.022
MULLEN BURST	D-3786	psi	270	300	324	333	395	324.4	41.505
TRAPEZOIDAL TEAR	D-4533	MD/lbs	64.9	44.3	51.5	50.7	63.5	54.98	7.943
		CD/lbs	62.4	61.5	48.0	75.9	77.9	65.13	10.901
PUNCTURE	D-4833	lbs	71.7	90.9	78.1	126.4	131.3	99.68	24.655
WIDE WIDTH TENSILE	D-4595	MD/lbs	316.5	317.3	339.8	293.1	345.5	322.44	18.739
INTERGEOTEXTILE SHEAR	D-3083(1)	lbs/in	17.2	17.6	22.1	26.0	27.9	22.16	4.315
INTERGEOTEXTILE PEEL	D-413(1)	lbs/in	6.6 Sep.(2)	5.7 Sep.(2)	4.4 Sep.(2)	2.4 Sep.(2)	7.4 Sep.(2)	5.30	1.760

NOTES: (1)Intergeotextile shear and peel performed using 4 inch wide specimens.
(2)Seam separated completely during test.

Table 3.4 Physical Property Test Results: Hydrated Bentomat® Containing High-Contaminant-Resistant Bentonite (J&L Testing Company, 1990a)

TEST	ASTM	UNITS	REPLICATE NO.					AVERAGE	STD DEV
			1	2	3	4	5		
GRAB STRENGTH	D-4632	lbs	88.6	82.5	84.7	87.1	103.0	90.86	7.147
GRAB ELONG.	D-4632	%	26.7	25.0	23.3	21.7	30.0	25.43	3.200
MULLEN BURST	D-3786	psi	130	120	75	135	145	121.0	24.372
TRAPEZOIDAL TEAR	D-4533	MD/lbs	44.6	59.6	51.5	51.0	73.5	56.02	9.932
PUNCTURE	D-4833	lbs	35.9	39.6	38.0	39.9	32.4	37.15	2.785
WIDE WIDTH TENSILE	D-4595	MD/lbs	276.8	238.1	288.2	295.7	264.2	272.58	20.260
INTERGEOTEXTILE SHEAR	D-3083(1)	lbs/in	42.0	49.1 Sep.(2)	67.0	49.1 Sep.(2)	46.0 Sep.(2)	50.63	8.584
INTERGEOTEXTILE PEEL	D-413(1)	lbs/in	13.4 Sep.(2)	14.0 Sep.(2)	20.6 Sep.(2)	11.9 Sep.(2)	8.9 Sep.(2)	13.74 Sep.(2)	3.835

NOTES: (1)Intergeotextile shear and peel performed using 4 inch wide specimens.
(2)Seam separated completely during test.

Table 3.5 Physical Property Test Results for NonWoven Geotextile Component of Bentomat® (J&L Testing Company, 1990a)

TEST	ASTM	UNITS	REPLICATE NO.					AVERAGE	STD DEV
			1	2	3	4	5		
GRAB STRENGTH	D-4632	MD/lbs	75.2	56.2	104.6	69.5	75.2	76.14	15.833
GRAB ELONGATION	D-4632	MD/%	188.4	120.0	126.6	116.6	175.0	145.32	30.177
MULLEN BURST	D-3786	psi	162	173	218	178	192	184.6	19.283
TRAPEZOIDAL TEAR	D-4533	MD/lbs	34.6	45.1	39.2	31.2	13.2	32.66	10.790
PUNCTURE	D-4833	lbs	54.0	49.2	25.0	42.4	36.9	41.49	10.085
WIDE WIDTH TENSILE	D-4595	MD/lbs	297.9	256.8	129.1	134.6	121.0	187.88	74.325

Table 3.6 Physical Property Test Results for Woven Geotextile Component of Bentomat® (J&L Testing Company, 1990a)

TEST	ASTM	UNITS	REPLICATE NO.					AVERAGE	STD DEV
			1	2	3	4	5		
GRAB STRENGTH	D-4632	MD/lbs	151.4	152.4	145.9	147.8	166.6	152.82	7.282
GRAB ELONGATION	D-4632	MD/%	20.0	23.3	23.8	22.5	21.7	22.26	1.337
MULLEN BURST	D-3786	psi	175	326	352	340	343	307.2	66.626
TRAPEZOIDAL TEAR	D-4533	MD/lbs	69.6	62.2	62.4	67.7	72.4	66.86	4.013
PUNCTURE	D-4833	lbs	9.2	12.4	13.3	16.0	11.0	12.38	2.279
WIDE WIDTH TENSILE	D-4595	MD/lbs	524.1	576.9	555.3	588.4	657.8	580.50	44.422

Section 4

Claymax®

4.1 Description

Claymax® is manufactured by the James Clem Corporation, 444 North Michigan, Suite 1610, Chicago, Illinois 60611 (telephone 312-321-6255). The material is a flexible mat consisting of granular sodium bentonite sandwiched between two geotextiles (Fig. 4.1). The primary backing, or top geotextile, is a slit-film, woven, polypropylene geotextile. The polypropylene geotextile typically weighs 3 oz. to 6 oz. per square yard (102 to 204 g/m^2), depending on the application, and provides durability and puncture resistance to protect and support the system during installation. The secondary backing, or bottom layer, is usually a spun-lace, open-weave polyester that weighs 3/4 oz. per square yard (25 g/m^2), although other materials can be substituted depending on specific requirements. The primary function of the secondary backing is to hold the bentonite in place during installation. In addition, the open weave of the backing allows the bentonite to expand when it hydrates and to ooze out between the openings so that a seal is formed. Information about the geotextiles is given in Table 4.1.

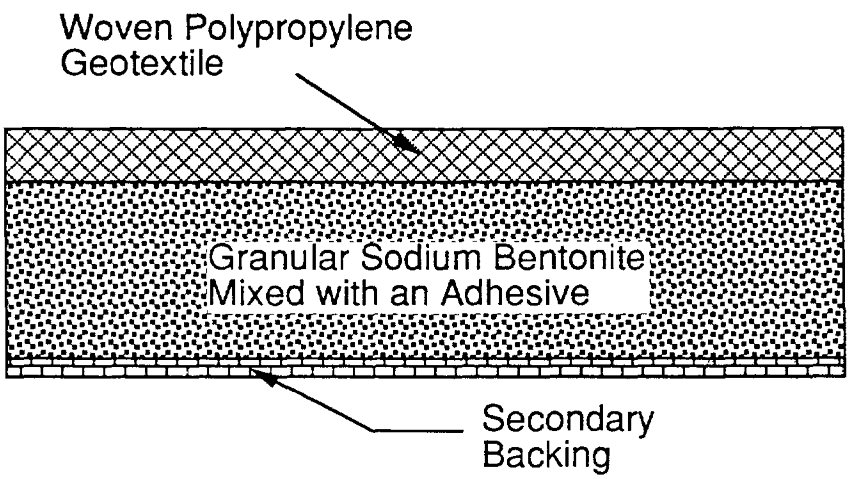


Figure 4.1 Schematic Diagram of Claymax® .

Table 4.1 Material Specifications (Supplied by Manufacturer)

(A) Primary Polypropylene

Substrate	Non-Biodegradable, Non-Toxic, Porous, Woven, Slit-Film, Polypropylene Geotextile
Tensile Strength (ASTM D4632)	78 lbs. per inch (1,390 kg/m) minimum
Burst Strength (ASTM D3786)	250 psi (1720 kPa)
Puncture Strength (ASTM D3787 mod.)	70 lbs (32 kg)
Elongation (ASTM D4632)	15%

(B) Secondary Backing

Description	Highly porous, non-structural, non-woven fabric that protects and contains the granular bentonite during installation

(C) Bentonite

Material	Natural Sodium Bentonite Containing a minimum of 90% Montmorillonite
Gradation of Bentonite	Two gradations: 6 Mesh and 16 Mesh Granules
Amount of Bentonite	Minimum of 1 lb/ft^2 (4.9 kg/m^2) Measured at Final Moisture Content
Final Moisture Content	15 to 18% (Typical)
Minimum Volumetric Increase (ASTM E946-83)	900%
Minimum Swell Index (USP NF XVII, "Bentonite Swelling Power")	25 ml
Gradation of Raw Bentonite	

Sandwiched between the two geotextiles is 1 lb per square foot (4.9 kg/m^2) of sodium bentonite adhered to the geotextiles with a water soluble, non-toxic, organic adhesive. The sodium bentonite consists of a minimum of 90% montmorillonite and is specially graded to have fine grained and coarse grained granules. Claymax® is manufactured in sheets that measure approximately 1/4 in. (6 mm) in thickness, 13.5 ft (4.1 m) in width, and 100 ft (30.5 m) in length. The sheets are placed on rolls, each of which weighs approximately 1400 lbs (635 kg).

4.2 Installation

The following discussion details the installation procedures recommended by the manufacturer. Before installation, the surface should be prepared by removing all angular rocks, roots, grass, vegetation, and foreign materials or protrusions. All cracks and voids should be filled. The surficial soils should be compacted to at least 90% of modified Proctor density (ASTM D1557). The prepared surface should be free from loose earth, fully-exposed rocks larger than 3/4 in. (19 mm) in diameter, rubble, and other foreign matter.

Claymax® is rolled out with the polypropylene side facing upward and with adjoining rolls overlapping at least 6 in. (150 mm). No soil should be between the rolls in the overlapped area. In hot, arid conditions, shrinkage may occur soon after placement; to account for shrinkage, the longitudinal seam overlap should be increased to 9 in. (230 mm) and the transverse overlaps increased to 4% of the run length plus 6 in. (150 mm). Seams should run up and down a slope and never horizontally on slopes. Claymax® should not be installed in rain or standing water; the material must be dry when installed and when covered. The liner should be installed in a relaxed condition and should be free of tensile stress upon completion of the installation. The liner may be pulled tight to smooth out creases or irregularities but should not be stretched to force the liner to fit.

In windy areas, installation should commence at the upwind side of the project area. The leading edge of the liner should be secured with sandbags or other means to hold the material in position during installation. Only material that can be anchored and covered in the same day should be unpackaged and placed in position. A trench should be used at the top of all slopes to lock the liner in place by placing the end of the roll of Claymax® in the trench and backfilling it. Irregular shapes or areas to be patched should be covered with sufficient material to provide a 6-in. (150 mm) overlap in all directions. Patch repairs should not be allowed on slopes steeper than 10%.

Claymax® must be protected from ultraviolet light and unrestrained hydration by covering the material with a geomembrane and/or by placing 6 to 12 in. (150 to 300 mm) of backfill or aggregate on top of Claymax®. If backfill is used, it should be compacted with wheeled rollers. Sheepsfoot compactors should not be used since the feet on the roller might damage the Claymax®.

4.3 Properties

4.3.1 Shear Strength

Geoservices Consulting Engineers (1989a) performed three sets of direct shear tests on selected Claymax® interfaces for the James Clem Corporation. The purpose of the tests was to evaluate internal shearing and frictional characteristics of fully-hydrated Claymax® placed against a silty sand and a smooth polyvinyl chloride (PVC) geomembrane. The tests were performed in 12-in. by 12-in. (300 mm by 300 mm) direct shear boxes which consisted of, from top to bottom: (1) a layer of silty sand soil, dense sand, or, dense sand and 40 mil (1.0 mm) PVC geomembrane, depending on the specific test being conducted; (2) Claymax® that had been fully hydrated for 24 hours under 500 psf (24 kPa) normal stress; and (3) a layer of dense sand. Vertical stresses ranging from 100 to 575 psf (5 to 24 kPa) were used, and shearing took place at a rate of 0.02 in/min (0.5 mm/min) with the upper half of the shear box in motion and the lower half fixed. Failure was forced through the bentonite layer for one series of tests; for the other series, failure was forced through the contact between the Claymax® and the overlying material (PVC geomembrane or silty sand). It appears that the rate of shearing may have been too rapid for long-term, fully-drained conditions to have been ensured.

Mohr-Coulomb diagrams are shown in Figs. 4.2, 4.3, and 4.4 for Claymax®, Claymax®/PVC, and Claymax®/sand, respectively. Results of the tests are summarized in Table 4.2. The friction angles were 12° for Claymax® alone, 15° for Claymax®/PVC, and 17° for Claymax®/sand.

Chen-Northern (1988) performed direct shear tests on the bentonite layer of samples of saturated Claymax® for a uranium mill tailings remedial action project (UMTRA) in Durango, Colorado. Two consolidated-undrained tests and two consolidated-drained tests were performed by applying strain rates of 0.047 and 0.00013 in/min (1.1 and .003 mm/min), respectively, under normal stresses of 3, 6, and 12 psi (20.7, 31.3, and 82.7 kPa). The test specimens were allowed to hydrate for 2 to 3 days prior to shearing. Results are plotted in Figs. 4.5 and 4.6 for undrained and drained tests, respectively. One data point for the consolidated-undrained tests was left off of Fig. 4.5 because the point was inconsistent with the overall trend of data (beyond the ordinary limits of variability of test data). The cohesion and friction angles computed by least-squares regression are summarized in Table 4.3. With undrained conditions, Claymax® had an average angle of internal friction of 16°. When sheared under drained conditions, Claymax® samples had an angle of internal friction of 14° and negligible cohesion.

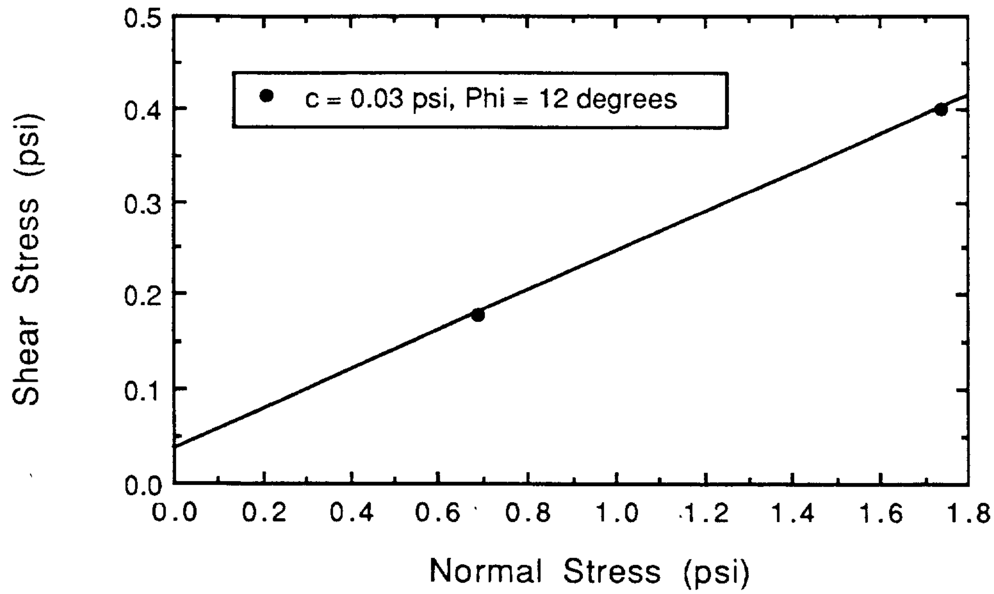


Figure 4.2. Mohr-Coulomb Failure Envelope for Direct Shear Tests Performed on Hydrated Bentonite with Shear Plane Passing through the Bentonite within Claymax® (Geoservices, 1989a).

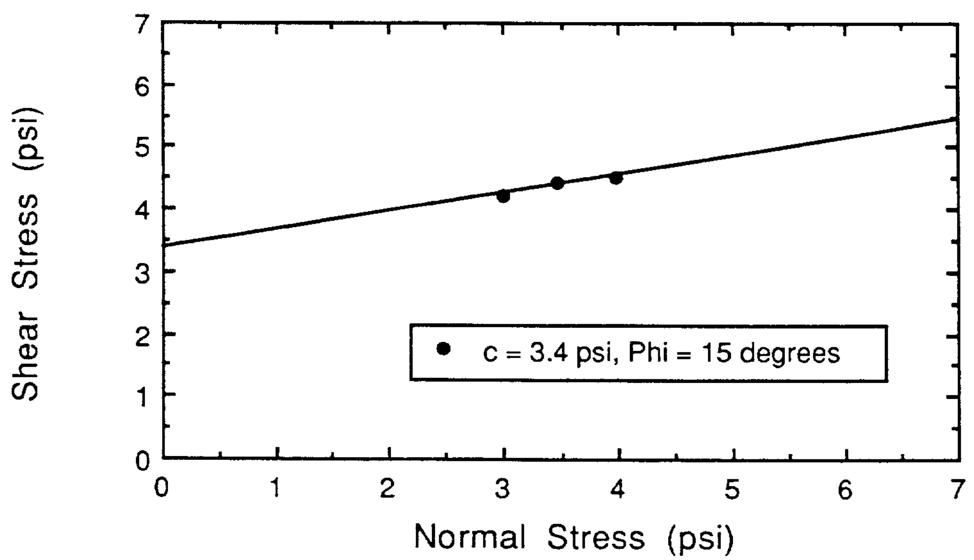


Figure 4.3. Mohr-Coulomb Failure Envelope for Direct Shear Tests Performed on Hydrated Bentonite with Shear Plane Passing through the Interface between Polypropylene Geotextile on Claymax® and a 40-mil PVC Geomembrane (Geoservices, 1989a).

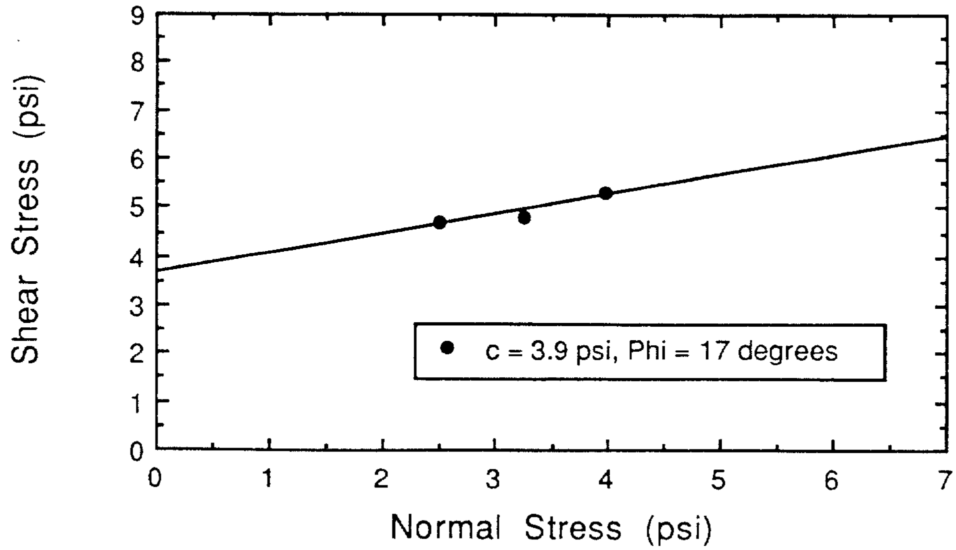


Figure 4.4. Mohr-Coulomb Failure Envelope for Direct Shear Tests Performed on Hydrated Bentonite with Shear Plane Passing through the Interface between Polypropylene Geotextile on Claymax® and a Silty Sand (Geoservices, 1989a).

Table 4.2 Summary of Results of Direct Shear Tests on Claymax® (Geoservices, 1989a)

Sample	Cohesion (psf)	Friction Angle (degrees)
Hydrated Claymax® Alone	5	12
Hydrated Claymax® Against PVC	490	15
Hydrated Claymax® Against Silty Sand	560	17

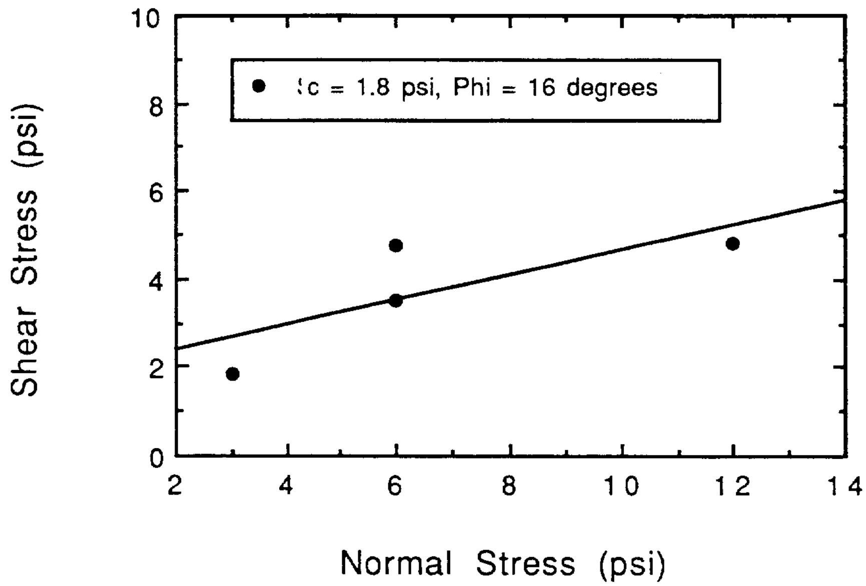


Figure 4.5. Results of Consolidated-Drained Direct Shear Tests on Claymax® (Chen Northern, 1988).

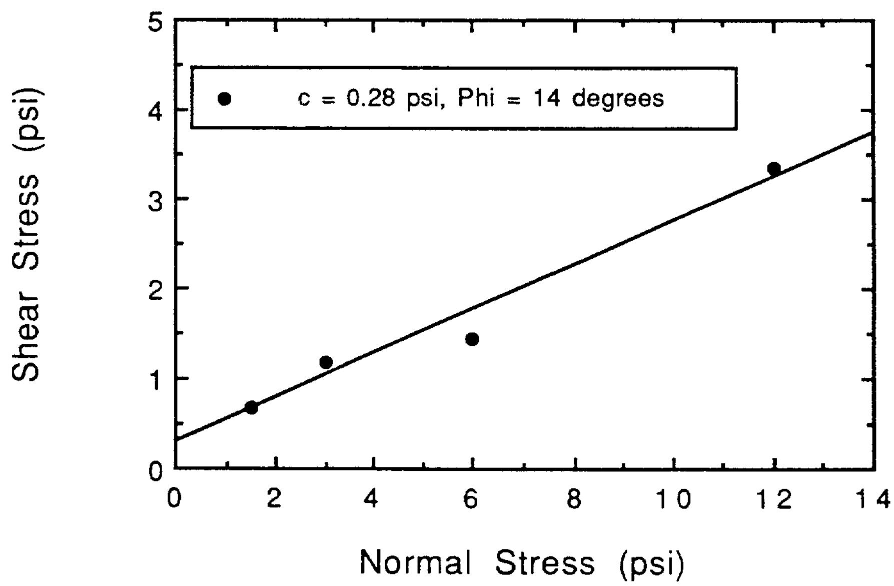


Figure 4.6. Results of Consolidated-Drained Direct Shear Tests on Claymax® (Chen Northern, 1988).

Table 4.3 Results of Direct Shear Tests on Claymax® (Chen-Northern, 1988)

Drainage Conditions	Cohesion (psf)	Friction Angle (degrees)
Consolidated-Undrained Conditions	260	16
Consolidated-Drained Conditions	40	14

Direct shear tests were also conducted by Shan (1990). Consolidated-drained tests were conducted on 2.5-in. (64-mm) diameter samples of both dry and fully saturated Claymax® with constant strain rates of 0.63 and 0.0008 in. per hour (16 and 0.02 mm/hr), respectively The soaking period was typically 2 to 3 weeks and the time to failure was approximately 3 to 5 days for the saturated Claymax®. The rate of shearing used by Shan (1990) appears to have been slow enough to ensure fully-drained failure. Normal stresses ranged from 575 to 2880 psf (28 and 138 kPa). Results are summarized in Table 4.4. The Mohr-Coulomb diagrams are shown in Figs. 4.7 and 4.8 for dry and hydrated bentonite, respectively. The internal angle of friction was found to be 28° for the dry Claymax®, and 9° for the hydrated Claymax®.

Table 4.4 Results of Direct Shear Tests on Claymax® (Shan, 1990)

Hydration Condition	Cohesion (psf)	Friction Angle (degrees)
Dry Bentonite	550	28
Hydrated Bentonite	90	9

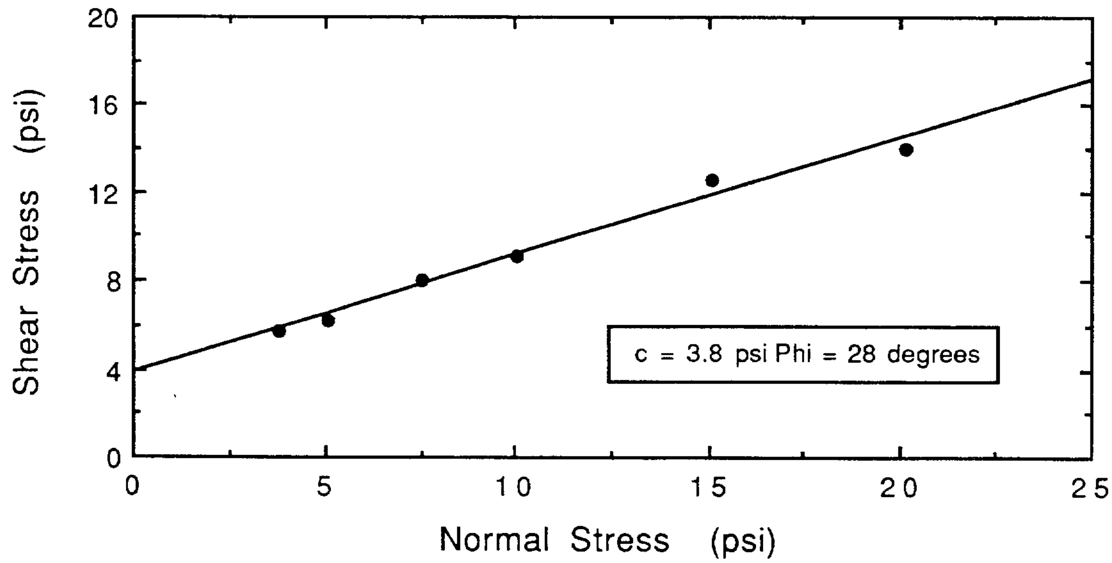


Figure 4.7. Results of Direct Shear Tests on Dry Samples of Claymax® (Shan, 1990).

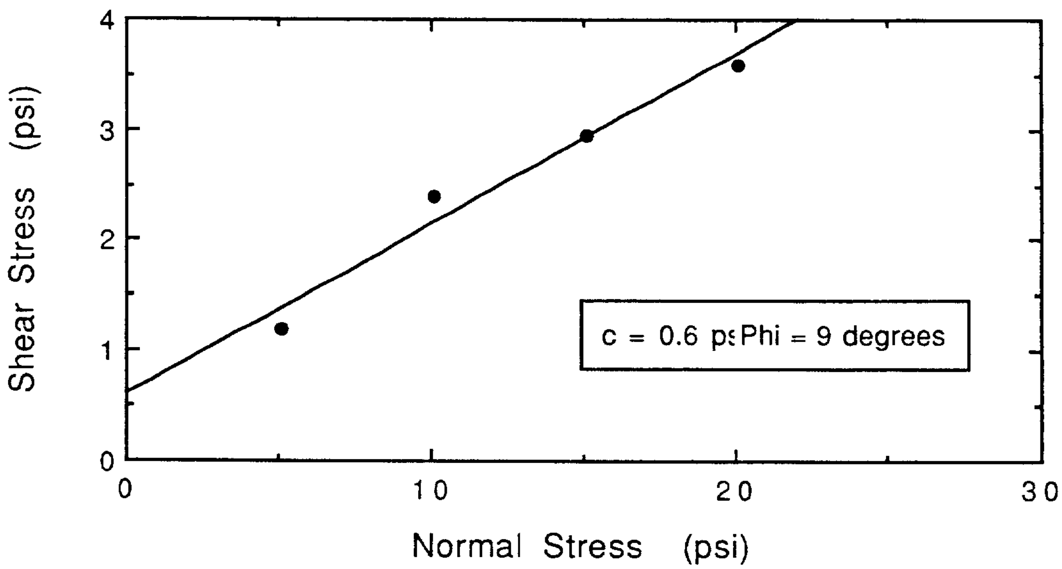


Figure 4.8. Results of Direct Shear Tests on Hydrated Samples of Claymax® (Shan, 1990).

4.3.2 Hydraulic Properties

4.3.2.1 Tests with Water

Literature published by the James Clem Corporation lists 2 x 10^{-10} cm/s as the hydraulic conductivity of Claymax® permeated with deaired water. A summary of published measurements of the hydraulic conductivity of Claymax® to water is given in Table 4.5. Results are plotted in Fig. 4.9 in terms of hydraulic conductivity versus effective confining stress. The results show that the hydraulic conductivity to water varies from just under about 1 x 10^{-8} cm/s at low effective stress to just above 1 x 10^{-10} cm/s at high effective stress.

Estornell (unpublished) permeated an 8 ft by 4 ft (2.4 m by 1.2 m) piece of Claymax® in a large tank, which is described more fully in Section 4.3.3. This data point is also shown in Fig. 4.9 and is similar, though slightly larger than, the trend of the other data.

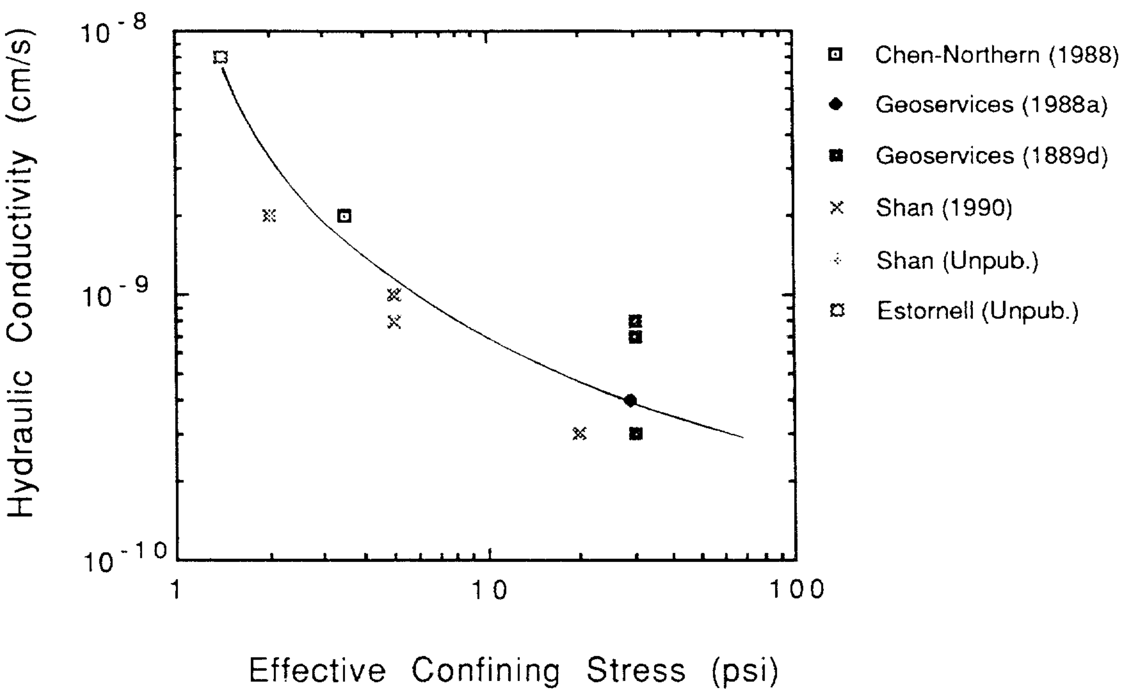


Figure 4.9 Results of Hydraulic Conductivity Tests on Claymax® Permeated with Water.

Table 4.5 Results of Hydraulic Conductivity Tests on Claymax® Permeated with Water

Source of Information	Permeameter	Backpressure Saturation?	Permeant Water	Diameter of Sample (in.)	Effective Stress (psi)	Hydraulic Conductivity (cm/s)
Clem Corp. Literature	- -	- -	Deaired Water	- -	-	2×10^{-10}
Chen-Northern (1988)	Flex. Wall	Yes	- -	2.5	3.5	2×10^{-9}
Geoservices (1988a)	Flex. Wall	Yes	Deaired Tap Water	2.8	29	4×10^{-10}
Geoservices (1989d)	Flex. Wall	Yes	Deaired Tap Water	2.8	30	8×10^{-10}
Geoservices (1989d)	Flex. Wall	Yes	Deaired Tap Water	2.8	30	8×10^{-10}
Geoservices (1989d)	Flex. Wall	Yes	Deaired Tap Water	2.8	30	3×10^{-10}
Geoservices (1989d)	Flex. Wall	Yes	Deaired Tap Water	2.8	30	7×10^{-10}
Shan (1990)	Flex. Wall	No	Distilled Water	4.0	2	2×10^{-9}
Shan (1990)	Flex. Wall	No	Tap Water	4.0	2	2×10^{-9}
Shan (1990)	Flex. Wall	No	Distilled Water	4.0	5	1×10^{-9}
Shan (1990)	Flex. Wall	No	Tap Water	4.0	5	8×10^{-10}
Shan (1990)	Flex. Wall	No	Distilled Water	4.0	10	6×10^{-10}
Shan (1990)	Flex. Wall	No	Distilled Water	4.0	20	3×10^{-10}
Shan (Unpub.)	Flex. Wall	Yes	Tap Water	12	2	2×10^{-9}

4.3.2.2 Various Liquid and Chemical Leachates

The information available concerning hydraulic conductivity of Claymax® permeated with liquids other than water is summarized in Table 4.6. All of the test specimens that were hydrated with water and then permeated with chemicals maintained a hydraulic conductivity ≤ 1 x 10^{-8} cm/s, even for compounds such as diesel fuel and heptane that would normally be very aggressive to soil liner materials. Brown, Thomas, and Green (1984), for example, found that the hydraulic conductivity of a compacted, micaceous soil was 1 to 4 orders of magnitude higher to kerosene, diesel fuel, and gasoline than it was to water. The inconsistency of results reported in Table 4.6 to the research conducted by Brown and his co-workers may be related to either a small cumulative pore volumes of flow in the tests on Claymax® or application of a high compressive stress to the test specimens. The cumulative pore volumes of flow of permeant liquid was not reported in many of the test referenced in Table 4.6; in many cases, there was probably an insufficient quantity of flow to determine the full effects of the permeant liquids. In some tests, a large effective confining stress was used. Broderick and Daniel (1990) found that one compacted clay was vulnerable to significant alterations in hydraulic conductivity when compressive stresses were ≤ 5 - 10 psi (34 - 69 kPa) but did not undergo an increase in hydraulic conductivity when the specimens were permeated with compressive stresses larger than 5 to 10 psi (34 to 69 kPa). Brown and his co-workers applied no compressive stress to their test specimens.

Tests on specimens of Claymax® that were hydrated with the same liquid as the eventual permeant liquid (rather than water) showed mixed results. For leachates, a paper pulp sludge, and simulated seawater, the hydraulic conductivity was found to be < 1 x 10^{-9} cm/s. However, the significance of these results is questionable because the duration of the tests was short, the cumulative pore volumes of flow was not reported, and the applied compressive stress was not reported. In as-yet unpublished tests by Shan, markedly different results were obtained when Claymax® was not prehydrated with water. Shan found that when dry Claymax® was permeated directly with a 50% mixture of water and methanol, with pure methanol, or with heptane, the bentonite did not hydrate even after several pore volumes of flow, and the hydraulic conductivity did not drop below 1 x 10^{-6} cm/s. Shan used a compressive stress of 5 psi (34 kPa). Thus, with concentrated organic liquids, the conditions of hydration appear to play an important role in determining the ability of the bentonitic blanket to resist the deleterious action of organic chemicals. The bentonite appears to be more chemically resistant if hydrated with fresh water before exposure to concentrated organic chemicals.

Table 4.6 Hydraulic Conductivity of Claymax® Permeated with Various Liquids

Source of Information	Permeant Liquid	Hydration Liquid	Pore Volumes of Flow	Effective Confining Stress (psi)	Hydraulic Conductivity (cm/sec)
STS Consultants (1988b)	Sewage Leachate	Sewage Leachate	- -	- -	8×10^{-10}
STS Consultants (1988c)	Paper Pulp Sludge	Paper Pulp Sludge	- -	- -	2×10^{-10}
Geoservices (1988b)	Simulated Seawater	Simulated Seawater	- -	30	2×10^{-10}
STS Consultants (1989a)	Landfill Leachate	Landfill Leachate	- -	- -	4×10^{-10}
STS Consultants (1989b)	Ash-Fill Leachate	Ash-Fill Leachate	- -	- -	1×10^{-10}
Geoservices (1989d)	Diesel Fuel	Water	1.5	30	9×10^{-10}
Geoservices (1989d)	Jet Fuel	Water	2.5	30	9×10^{-10}
Geoservices (1989d)	Unleaded Gasoline	Water	1.6	30	3×10^{-10}
Geoservices (1989d)	Gasahol	Water	0.5	30	3×10^{-10}
Shan (1990)	50% (Vol) Methanol	Water	2.2	5	9×10^{-10}
Shan (1990)	Heptane	Water	0.2	5	1×10^{-10}
Shan (1990)	Sulfiric Acid	Water	3.1	5	6×10^{-11}
Shan (1990)	0.01 N $CaSO_4$	Water	2.2	5	1×10^{-9}
Shan (1990)	0.5 N $CaCl_2$	Water	24	5	8×10^{-9}
Shan (Unpublished)	50% (Vol) Methanol	50% Methanol	4	5	5×10^{-6}
Shan (Unpublished)	Methanol	Methanol	5.4	5	3×10^{-5}
Shan (Unpublished)	Heptane	Heptane	4.3	5	5×10^{-5}

4.3.2.3 Effects of Desiccation

The effects of desiccation were investigated by Geoservices (1989e). Three hydrated samples of Claymax® were placed in a temperature- and humidity-controlled chamber. The chambers operated on a timed cycle to simulate day and night conditions. The temperature and humidity during the day cycle were 95°F and 30%, respectively, while the temperature and humidity during the night cycle were 70°F and 50%, respectively. Samples of Claymax® were buried below 8 in. (200 mm) and 18 in. (450 mm) of sand, while a third sample was not buried beneath any sand. Water content samples were obtained from the Claymax® regularly throughout the 3-month test period.

Results of the Geoservices tests are summarized in Table 4.7. The Claymax® sample left exposed with no sand overburden underwent severe drying. In comparison, little or no desiccation appeared to have occurred during the testing period when Claymax® was buried beneath sand. The sand appeared to provide an adequate buffer to the extremes of temperature and humidity to protect the Claymax® from desiccation.

Table 4.7 Results of Desiccation Studies on Sand Overlying Claymax® (Geoservices, 1989e)

Depth Below		Water Content (%)					
Top of Sand (in.)	Elapsed Time (Days):	0	4	21	25	47	90
0		1300	690	15	-	6	1
8.5		260	- -	- -	280	- -	260
18.5		300	- -	- -	265	- -	248

Shan (1990) studied the effects of desiccation on the hydraulic properties of Claymax® in a different way. Shan measured the hydraulic conductivity of 4-in (100-mm) diameter samples of Claymax® that had been subjected to several wet-dry cycles. His experiment involved permeating Claymax® specimens in a flexible-wall permeameter using an effective stress of 2 psi (14 kPa) and a hydraulic gradient of about 50. The specimens were then removed from the permeameter and allowed to air dry. At first the specimens were dried on a laboratory table top with no overburden, but the specimens shrank to much smaller-diameter circular discs and did not undergo significant cracking. To force desiccation cracks to develop, a vertical stress of about 0.2 psi (1 kPa) was applied to the specimens while they dried. The stress was applied by placing a steel cylinder on top of the sample to be desiccated. Numerous large (2-mm-wide) desiccation cracks were seen in the dried specimens that had the small overburden stress. The desiccated specimens containing cracks were set up again in a flexible-wall permeameter and were permeated under the same conditions. After permeation, the specimens were removed and desiccated/permeated again. It was found that after 3 wet-dry cycles, the hydraulic conductivity of Claymax® did not change; it remained approximately 2 x 10^{-9} cm/s. Shan reported that at the beginning of repermeation after drying, the hydraulic conductivity was on the order of 10^{-4} cm/s as water flowed through the cracks very easily. But the cracks closed within a few hours and the flow stopped as bentonite hydrated and took in water from both influent and effluent ends. It was not until bentonite was fully hydrated that flow started again.

Chen-Northern (1988) conducted hydraulic conductivity tests in flexible-wall permeameters on samples of Claymax® that had undergone 0, 3, and 10 cycles of wetting and drying. The hydraulic conductivity increased approximately 2.6 times after three cycles of wetting/drying but underwent no further increase with additional wet/dry cycles. Hydraulic conductivities were 1 x 10^{-9} cm/s, 2.6 x 10^{-9} cm/s, and 2.3 x 10^{-9} cm/s, respectively, for samples subjected to 0, 3, and 10 cycles of wetting and drying.

4.3.2.4 Hydraulic Properties of Damaged Claymax®

Hydraulic conductivity of Claymax® was measured by STS Consultants (1988a) on a specimen of Claymax® that had been subjected to 15% elongation. The purpose of the experiment was to study the hydraulic integrity of Claymax® after a specimen had undergone deformation. The Claymax® specimen (evidently in a dry condition) was first stretched to 15% elongation. A 2.5-in. (64 mm) diameter piece of the stretched Claymax® was trimmed from the larger piece that had been stretched and was then placed above approximately 5-1/2 in. (140 mm) of silica sand in a flexible-wall permeameter. The test specimen was hydrated, saturated, and permeated with de-aired water. The effective consolidation stress was 0.15 kg/cm^2 (125 kPa) and the backpressure was 4.0 kg/cm^2 (392 kPa). The test was allowed to continue until steady state was reached. The hydraulic conductivity of the material that had been subjected to a 15% elongation was determined to be 4 x 10^{-10} cm/s. Elongation appeared to have a negligible effect upon the hydraulic characteristics of the material.

The effects of punctures on the hydraulic conductivity of Claymax® was investigated by Shan (1990). Punctures were simulated by cutting three holes, each 0.5, 1.0, or 3.0 in. (13, 25, or 75 mm) in diameter, in dry Claymax® specimens. The specimens were permeated with tap water in a flexible-wall permeameter under an effective stress of 2 psi (14 kPa) and a hydraulic gradient of about 50. The test specimens that had been punctured with holes 0.5 and 1.0 in. (13 and 25 mm) in diameter had hydraulic conductivities of 3 x 10^{-9} and 5 x 10^{-9} cm/s, respectively, which is only slightly larger than the value of 2 x 10^{-9} cm/s measured on an intact specimen. With the specimen containing 3 holes each 3-in. (75 mm) in diameter, 2 of the 3 holes did not seal themselves and were left with openings of about 0.5 in. (13 mm) in diameter. These tests, in conjunction with the tests on desiccated specimens, demonstrate that the swelling nature of bentonite gives this material the capability of self-healing small defects or punctures when the material is hydrated with water.

4.3.2.5 Composite Action

Shubert (1987) described various tests on composites of Claymax® placed adjacent to defective HDPE geomembrane liners. In the first series of tests, Claymax® was placed between two defective HDPE sheets in a configuration that simulated the usage of the material at a landfill in the Chicago area. The four separate tests, the upper and lower HDPE sheet were slit over a length of 1 in. (25 mm) with razor blade or punctured with a large nail, but the Claymax® was left intact. The composites were tested in a flexible-wall permeameter with a maximum effective confining stress of 30 psi (207 kPa). Leachate from a hazardous waste landfill was used as the permeant liquid and was pressurized with 10 psi (69 kPa) to induce permeation into the Claymax®. No inflow or outflow was recorded after initial pressurization of the system over the 3-day test duration.

A second series of tests is described by Shubert (1987). Three samples were tested:

Sample 1: Top HDPE: punctured with 0.84-in. (21 mm) diameter hole
Bottom HDPE: punctured with 16-penny nail
Sample 2: Top HDPE: punctured with 0.84-in. (21 mm) diameter hole
Bottom HDPE: Slit with razor blade for 1 in. (25 mm) length
Sample 3: Top HDPE: punctured with 0.84-in. (21 mm) diameter hole
Bottom HDPE: punctured with 0.84-in. (21 mm) diameter hole.

Test conditions were the same as in the first series of tests. After 5 days of permeation, the quantity of inflow was 1.6, 0.8, and 0.8 mL for Samples 1, 2, and 3, respectively. Visual observations after the tests revealed that the bentonite was significantly wetted near the proximity of each membrane defect. The wetting of the bentonite, in turn, is reported to have caused significant swelling of the bentonite, which caused plugging of the defect.

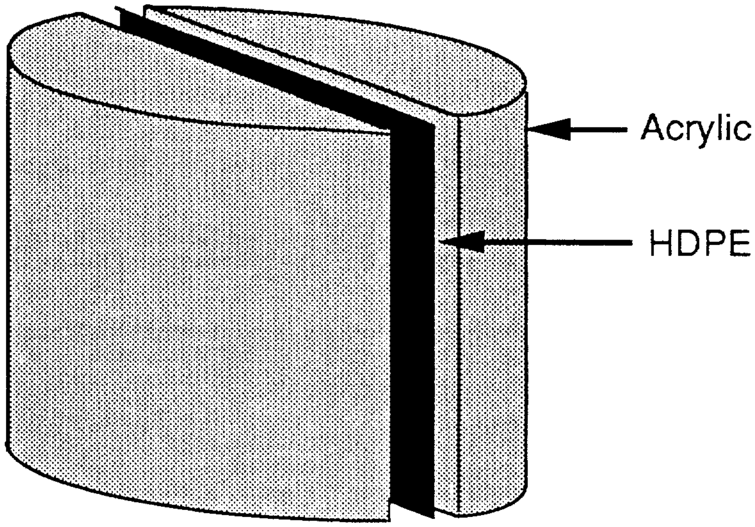
A third series of experiments was conducted on two samples that had holes of 0.84 in. (21 mm) diameter drilled into the top and bottom HDPE sheets. Hazardous waste landfill leachate was introduced under the same testing conditions described earlier. The "apparent" hydraulic conductivities of the samples were approximately 1 x 10^{-9} cm/s after more than 100 pore volumes of flow. A control test on Claymax® alone was not performed.

Shan (1990) conducted a test using a flexible-wall permeameter in order to measure the in-plane hydraulic conductivity of Claymax® in contact with two sheets of high density polyethylene (HDPE). The test set-up is shown in Fig. 4.10. The effective confining stress was 2 psi and a head of water of 1 ft was applied to one end of Claymax®. No outflow occurred for about 2 months. When steady flow was finally reached, the computed hydraulic conductivity was 2 x 10^{-6} cm/s. At least some seal was obtained between the Claymax® and the HDPE because there was no outflow for two months. However, in view of the high in-plane conductivity, the seal was evidently imperfect.

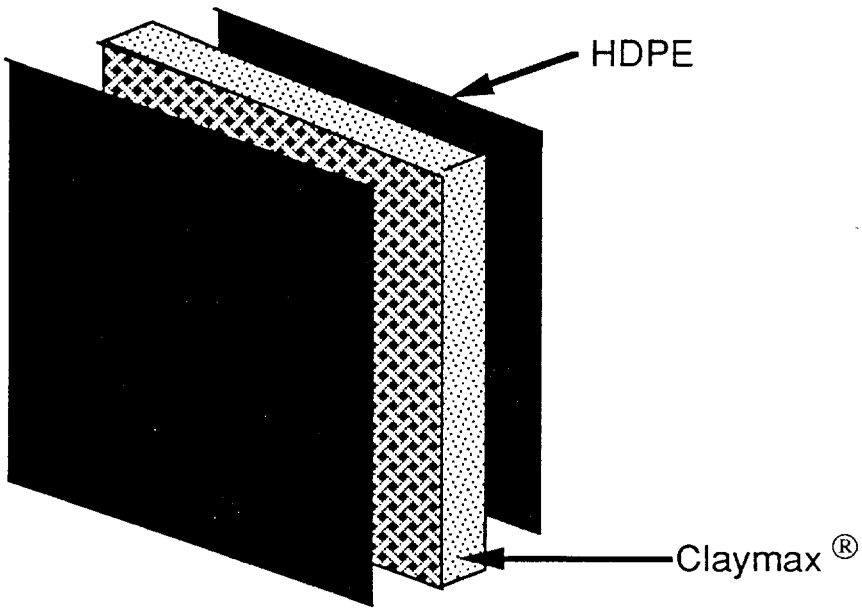
Shan (unpublished) permeated 3 samples of 12-in. (300-mm) diameter Claymax® in flexible-wall permeameters using an effective stress of 2 psi (14 kPa) and backpressure saturation. Two of the three Claymax® samples were overlain by a sheet of defective HDPE sheet (the sheet was placed against the polypropylene geotextile, which would normally be the upper geotextile in the field); the third sample was a control with no HDPE. One of the two HDPE sheets was punctured with 3 holes, each 1 in. (25 mm) in diameter, and the second was slit with a 1-mm-wide slit having a length of 6 in. (150 mm). The hydraulic conductivities were as follows:

Control (No HDPE):	Hydraulic Conductivity = 2 x 10^{-9} cm/s
Composite (3 Holes in HDPE):	Hydraulic Conductivity = 4 x 10^{-9} cm/s
Composite (Slit in HDPE):	Hydraulic Conductivity = 4 x 10^{-9} cm/s

It is not known why the hydraulic conductivities of the composites were slightly greater than those of the control -- the conductivities of the composites should have been less or equal to that of the control. Nevertheless, the data do not indicate that a particularly good seal developed between the HDPE and the bentonite. Liquid evidently spread laterally through the geotextile and



(A) Control Test for Determining Baseline Flowrate



(B) Test on Composite Action between Claymax® and HDPE

Figure 4.10. Schematic Diagram of Test to Evaluate In-Plane Flow (from Shan, 1990).

permeated a large percentage of the area of the Claymax®. Better contact might have been achieved if the other side of the Claymax® (the side with the light-weight, spun-lace polyester) was placed against the HDPE. This possibility is being evaluated by the authors.

4.3.3 Seams

STS Consultants Ltd. (1984) performed a hydraulic conductivity test on a 2-in. (50 mm) wide overlapped seam of Claymax®. The test arrangement is depicted in Fig. 4.11. The specimens were placed above a 6-in. (150-mm) thick layer of silica sand in a flexible-wall permeameter and were backpressure-saturated prior to permeation with deaired water. The overlapped materials were hydrated and permeated from bottom to top (with flow from the underlying sand to the Claymax®. No details on the effective compressive stress, hydraulic gradient, magnitude of backpressure, or duration of test were given. The hydraulic conductivity of the test materials with the overlapped seam was 7×10^{-10} cm/s using a weighted average specimen thickness of 0.4 in. (10 mm).

Bench-scale hydraulic conductivity tests on seam overlaps are currently being conducted at the University of Texas at Austin. The experiments are being performed in three rectangular steel tanks, shown schematically in Fig. 4.12, that measure 8 ft (2.4 m) in length, 4 ft (1.2 m) in width, and 3 ft (0.9 m) in height. A 1/2-in. (13 mm) diameter drain hole has been drilled at the center of the base. To conduct a test, a geotextile/geonet/geotextile composite drainage layer is placed over the bottom of the tank (except that a 3-in. or 75 mm wide gap is left between the drainage material and edge of the tank to accommodate a bentonite seal that seals the material being tested to the bottom of the tank). Next, dry bentonite is placed in the 3-in. (75 mm) wide gap left between the drainage material and the walls of the tank. The bentonitic blanket being tested is placed over the drainage material and bentonite edge seal, with the edges of the material going to the edges of the steel tank. Next, a 1-ft (0.3 m) or 2-ft (0.6 m) thick layer of gravel is placed over the bentonitic blanket. The tank is slowly filled with a depth of water above the bentonitic blanket of 1 to 2 ft (0.3 to 0.6 m). Effluent water passing through the drain hole is collected and weighed to determine the flux of water through the material being tested. The thickness of the material is estimated based on laboratory measurements. Hydraulic conductivity is calculated from the measured flux and known head and known area and thickness of the bentonitic blanket.

Tests have recently been completed on three samples of Claymax®. The tests involve: (1) a 6-in. (150-mm) wide overlap (in accordance with the manufacturer's recommended minimum overlap width); (2) a 3-in (75-mm) wide overlap (intended to evaluate whether the recommended 6-in. or 150-mm wide overlap includes a generous factor of safety); and (3) a

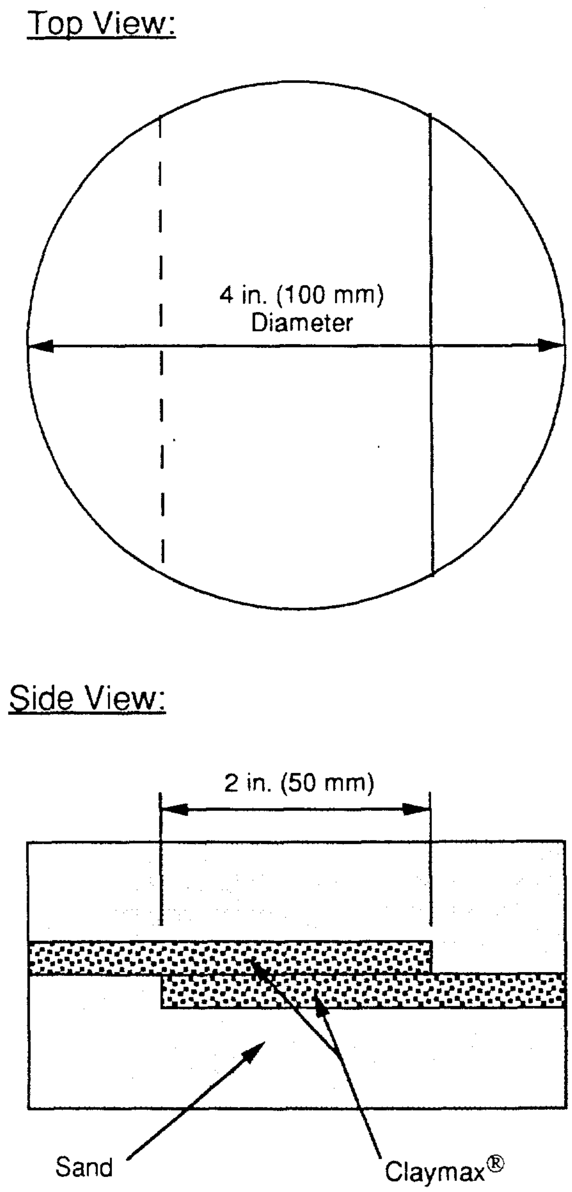


Figure 4.11. Schematic Diagram of Laboratory Test Designed to Evaluate the Hydraulic Conductivity of Overlapped Seam (from STS Consultants, 1984).

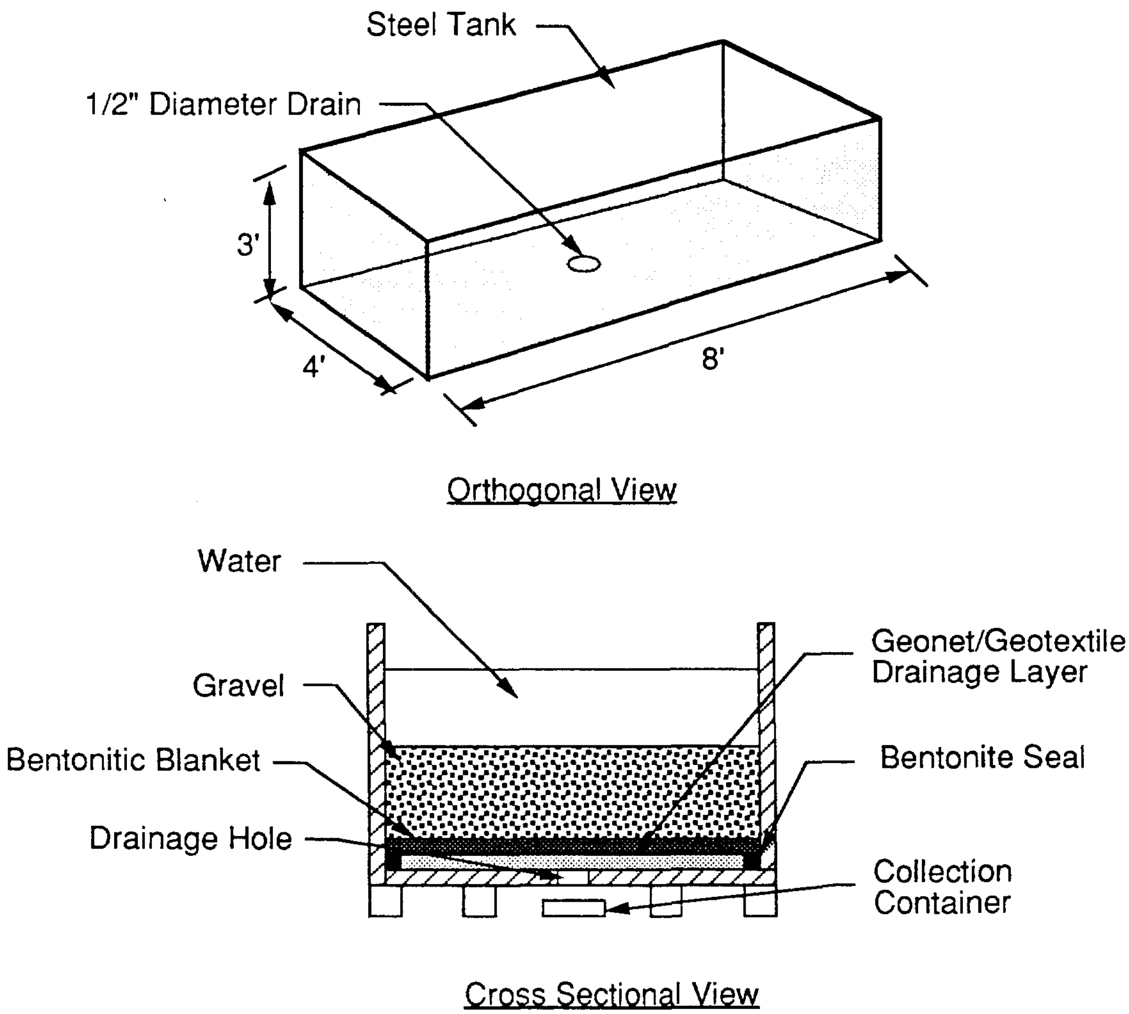


Figure 4.12. Schematic Diagram of Tanks Being Used to Measure Hydraulic Conductivity of Bentonitic Blankets Containing Overlapped Seams.

control with no overlap. In the first tests, the gravel layer was 1-ft (0.3 m) thick, and the depth of water above the Claymax® was 2 ft (0.6 m). These conditions were estimated to produce an effective vertical stress of approximately 75 psf (3.6 kPa) at the top of the Claymax® and 200 psf (9.6 kPa) at the bottom of the Claymax®. Steady state flow was achieved after approximately 1.5 months, and the hydraulic conductivities were 9×10^{-9} cm/s, 2×10^{-8} cm/s, and 6×10^{-8} cm/s for the control, 6-in. (150 mm) wide seam, and 3-in. (75 mm) wide seam, respectively. Hydraulic conductivity was calculated assuming a thickness of 0.5 in. (13 mm) for the material based on data published by Shan (1990). Effective overburden stress was later increased to approximately 220 psf (10.5 kPa) at the top of the Claymax® but steady state conditions have not been reached as of this writing.

4.3.4 Swelling Characteristics

Shan (1990) measured the swelling characteristics of Claymax® as follows. A 2.5-in. (64-mm) diameter specimen was trimmed, placed in a consolidation ring, compressed with a controlled vertical stress, and then hydrated with water. The percentage change in height was monitored until the sample ceased to swell or compress. The test was repeated for several different levels of stress. Results are plotted in Fig. 4.13. The stress at which no compression or swelling occurred was found to be approximately 3,000 psf (144 kPa).

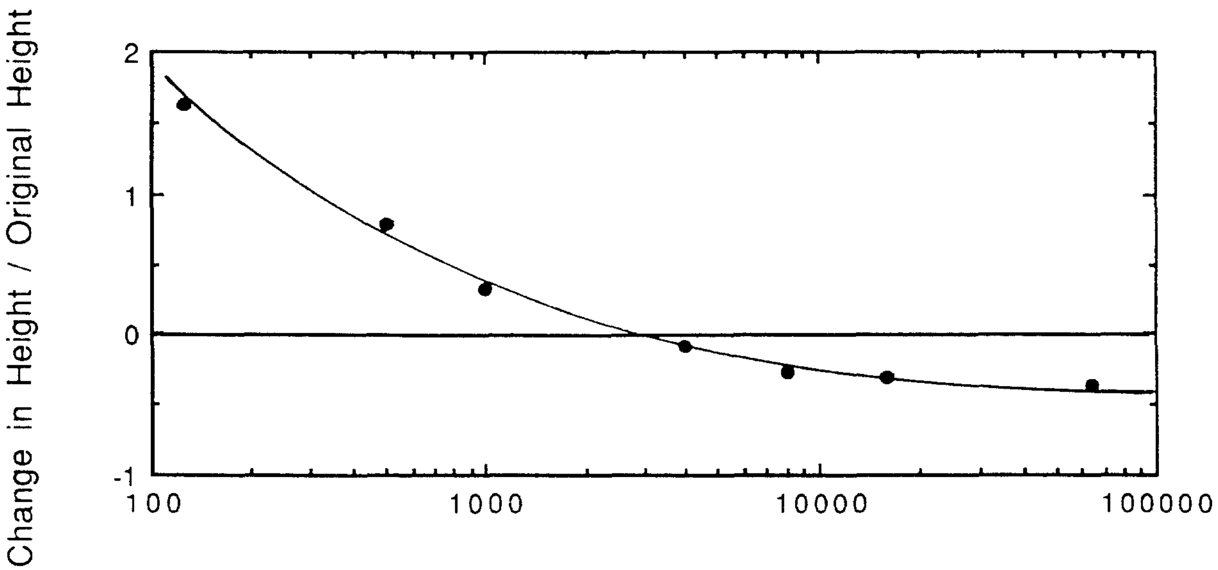


Figure 4.13. Results of Swelling Tests on Samples of Claymax® (from Shan, 1990).

4.4 Examples of Use

Lutz (1990) describes examples of the use of Claymax® based on information supplied by the James Clem Corporation. The following was taken from Lutz's discussion.

The Broward County Landfill in Fort Lauderdale, Florida, is a 20 acre (8 ha) incinerator ash monofill. County officials wanted an alternative to importing clay to construct a liner for the landfill so they decided to use Claymax®. The components, from bottom to top, are as follows: 6 in. (150 mm) of bedding sand; a 60 mil (1.5 mm) HDPE liner; 12 in. (300 mm) of drainage sand; Claymax®; an 80 mil (2 mm) HDPE liner; and 24 in. (600 mm) of drainage sand.

A 3 acre (1.2 ha) commercial hazardous waste landfill in Calumet City, Illinois, contains a double composite liner system with a secondary leachate collection and removal system. The components, from bottom to top, are as follows: 3 ft (0.9 m) of compacted clay; a 60 mil (1.5 mm) HDPE liner; a secondary leachate collection and removal system; Claymax®; and a 100 mi (2.5 mm) HDPE liner. This landfill was completed in March of 1986, and there has been no accumulation of leachate in the secondary leachate collection system.

St. Paul Island is in the Bering Sea and acts as a refueling site for fishing vessels. The fuel storage tank farm needed to be enlarged and relocated. Since the island is a primary breeding ground for the northern fur seal and has a large seabird population, the environmental sensitivity of the island was a major concern. The cold, wet, and windy climate of St. Paul Island makes construction difficult. Claymax® was used because of its ability to form a barrier to fuel oils as well as its ease of installation. The liner was installed in sections between periods of inclement weather. Pipe penetrations were sealed by wrapping Claymax® around the pipe at the penetration. For tank farm applications, the Claymax® must be saturated with water after it has been covered with the bedding material, otherwise the Claymax® will neither hydrate properly nor impede the flow of a hydrocarbon spill.

During the open discussion session of the Alternative Barriers Workshop, Steve Walker of Polyfelt, Inc., and John Boschuk of J & L Testing Company, Inc., described two other cases in which Claymax® was used as a liquid barrier. A 60-acre (24 ha) ravine in the Hudson Valley area was proposed as a site to contain PCB's. The existing subbase was a weak material (standard penetration test N-value of 2) with organic deposits that generated gas. The requirements for the liner were that it be impervious, collect gas, and act as reinforcement. Instead of using compacted clay that would have been difficult to impossible to compact on the existing subbase and would have required 22,000 truck loads of clay, a custom-made Claymax® product was used which required only 90 tractor trailers at two-thirds the cost. The custom made Claymax® liner was made using a 10 ounce per square yard (340 g/m^2) geotextile cover fabric instead of the typical 1 ounce per square yard (34 g/m^2) in order to meet the requirements of collecting gas and providing reinforcement.

The second case involved a 5 acre (2 ha) pond at a campground. Two feet (600 mm) below existing mudline was a layer of decomposed rock which was causing water to drain out of the pond. Campground owners decided to line the pond with Claymax®. An anchor trench was deemed unnecessary because the maximum side slope at the site was 12:1. A friction anchor was used instead, consisting of 1 foot (300 mm) of soil placed above the Claymax® liner at the top of the slope. Some time shortly after construction, a slope failure occurred. On the 16:1 (4°) slopes, the Claymax® liner slid only a few inches; on the 12:1 (5°) slopes, the liner slid all the way down the embankment. The sliding surface was between the geotextile and the ground surface. It is hypothesized that movement of the slightly viscous bentonite in the Claymax® caused slippage. It is evident from this failure that anchor trenches are important and that more information is needed concerning the frictional characteristics of Claymax®.

In addition to the examples listed above, Claymax® liners have been used to waterproof building foundations and to line waste lagoons and irrigation canals. Claymax® has also been used as an alternative to cutoff walls and slurry walls and has been used as part of the core material in a dam. This type of information is available in a series of publications supplied by the manufacturer.

Section 5.0

Gundseal

5.1 Description

Gundseal will be manufactured by the team of Gundle Lining Systems, Inc. (18100 Gundle Road, Houston, Texas 77073, telephone 713-443-8564), and Paramount Technical Products, Inc. (2600 Paramount Drive, Spearfish, South Dakota 57783, telephone 605-642-4787). The manufacturing facility will be located in Spearfish, South Dakota. Gundseal will be similar to an existing product, Paraseal, which is manufactured by Paramount Technical Products, Inc. Paraseal consists of one pound per square foot (4.5 kg/m^2) of sodium bentonite glued to a 20 mil (0.5 mm) HDPE geomembrane (Fig. 5.1), although the liner can be manufactured with other thicknesses of HDPE. Paraseal can be supplied with or without a light-weight fabric backing, which helps to prevent spalling of small granules of bentonite. Paraseal is manufactured in 24-ft (7.3 m) long by 4-ft (1.2 m) wide rolls. Paraseal can be installed with the HDPE facing upward or downward. The material is available with different grades of bentonite, depending upon whether the bentonite is to retain fresh water or saline water. Paraseal is seamed in the field with simple overlaps; although no mechanical seam is necessary, mechanical seaming of HDPE to HDPE is possible. To date, Paraseal has been used primarily for waterproofing basement walls, basement slabs, water-retention structures, and small reservoirs and ponds.

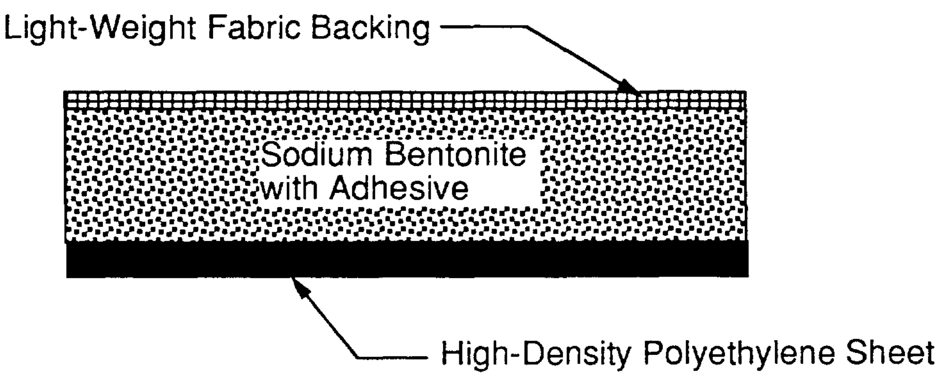


Figure 5.1. Schematic Diagram of Paraseal and Gundseal.

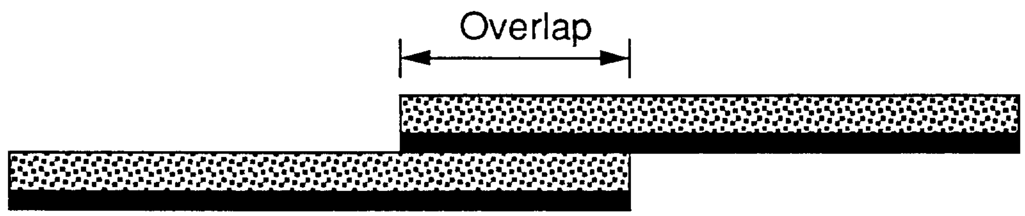
Gundseal will be composed of the same materials as Paraseal but will be produced in rolls that are approximately 17.5-ft (5.3 m) wide and approximately 200-ft (60 m) long. Each roll will weigh just under 4,000 lbs (1800 kg) and will have a diameter of approximately 3 ft (1 m). The thickness of HDPE sheet in Gundseal will initially be 20 mil (0.5 mm), but greater thicknesses are also expected to be available. Whereas Paraseal is intended for use primarily in structural waterproofing, Gundseal is designed for applications involving landfill liners and covers, liquid containment ponds, waste water lagoons, tank farms, etc. Gundle Lining Systems, Inc., anticipates utilizing Gundseal as a back-up liner for conventional HDPE geomembrane liners. For such an application, Gundseal would be installed with the bentonite facing upward, as shown in Fig. 5.1. A conventional geomembrane liner would then be placed directly on the bentonite. If there are any defects in the geomembrane liner, such as a pinhole or defective seam, the leakage through the geomembrane would be minimized by the bentonite layer within the Gundseal.

There are no technical data currently available on Gundseal because the product has not yet been produced. The following section presents information about the established Paraseal liner, which is similar to Gundseal.

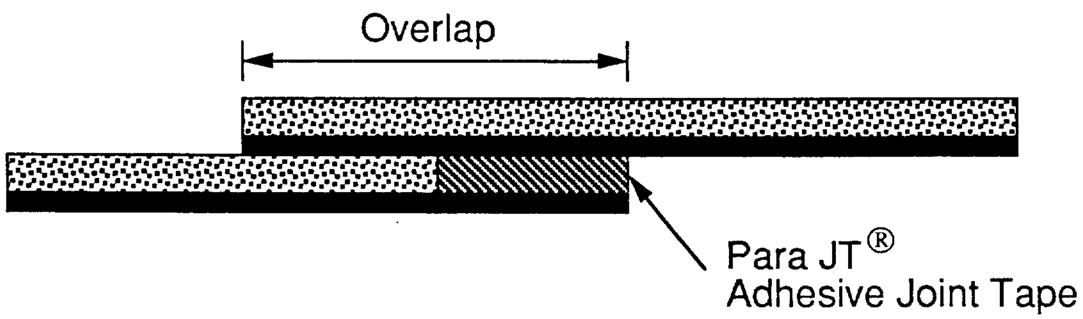
5.2 Installation

The following discussion summarizes the manufacturer's recommended installation procedures. The area to be covered by Gundseal or Paraseal must be graded level. All rocks, sticks, other sharp objects and loose soil should be removed. The product can be installed with the HDPE side facing either up or down. If used by itself as a composite liner, the HDPE would normally face up. If the material is used to backup a geomembrane liner that is placed on top of the Paraseal, the product is installed with the bentonite facing up.

Paraseal is unrolled and placed on the area to be covered, with adjacent rolls overlapping at least 1.5 to 3 in. (38 to 75 mm). The material is said to be self-seaming; when the bentonite is hydrated and swells, the bentonite/HDPE contact is hydraulically sealed. Thus, no mechanical joining of the seams is necessary (Fig. 5.2a), although the overlapped sheets of HDPE can be mechanically joined with a double-sided tape called Para JT® (Fig. 5.2b). Para JT® is a proprietary adhesive joint tape compounded from a family of partially cross-linked polymeric elastomers. Para JT® is placed on the HDPE along a strip where the bentonite has been removed from the edge of the roll. This configuration results in a double seal: one seal is made by the Para JT® between two pieces of HDPE and the second between the bentonite and the HDPE. Other methods for joining the HDPE sheets, e.g., fillet extrusion welding, could probably



(A) Overlap of Paraseal



(B) Overlap of Paraseal with Adhesive Joint Tape

Figure 5.2. Overlap of Paraseal.

be employed. Paraseal should be anchored in trenches around the perimeter of the site. Upon completion of the liner, the Paraseal must be covered with soil or other protective material.

5.3 Properties

5.3.1 Physical Properties

The physical properties of Paraseal reported by the manufacturer are summarized as follows. The HDPE membrane has a tensile strength (ASTM D412) of 4,000 psi (27 MPa). Elongation at failure (ASTM D638) is reported to be 700%. Puncture resistance (Federal Test Method Standard No. 101B) is 95 lbs (43 kg). Permeance is reported to be 2.7 x 10^{-13} cm^3/cm^2 when the membrane is applied to a porous stone and placed in a permeameter with a pressure head equivalent to 150 ft (45 m) of water.

5.3.2 Shear Strength

No information is available on the shear strength of Paraseal.

5.3.3 Hydraulic Properties

Pittsburgh Testing Laboratory (1985) conducted a hydraulic conductivity test on a 2.5-in. (64-mm) diameter sample of Paraseal. A 15-ft (4.6-m) head of water was applied to the sample, which was soaked for 5 days prior to permeation. A single, falling-head test was performed, which yielded a hydraulic conductivity reported to be 4 x 10^{-10} cm/s. Further details of the test procedures are not available. However, because the direction of flow was apparently through the HDPE membrane, the test may have provided a measure of sidewall leakage rather than flow through the material.

5.3.4 Seams

Twin City Testing Corporation (1986) measured the hydraulic conductivity of the bentonite in overlapped pieces of Paraseal with flow taking place parallel to the HDPE sheets. A schematic diagram of the test arrangement is shown in Fig. 5.3. Two 1 in. by 4 in. (25 by 100 mm) pieces of Paraseal were placed against one another and clamped between two half-cylinders of lucite. The assembly was placed in a flexible-wall permeameter. The overlapped pieces of Paraseal were compressed with a stress of 24 psi (165 kPa), hydrated under a 6-in. (150-mm) head of water for 17 days, and permeated with a head of 40 ft (12 m) for 12 days. The hydraulic conductivity for in-plane flow with this arrangement was 2 x 10^{-10} cm/s.

Bench-scale hydraulic conductivity tests on seam overlaps are currently being conducted at The University of Texas at Austin. A description of the apparatus was given in section 4.3.3 and a diagram of the test apparatus is provided in Fig. 4.12. The three tests currently underway on Paraseal liner are: (1) a 3-in. (75-mm) wide overlap; (2) a 1.5-in. (38-mm) wide overlap; and (3) a control with no seam overlap. One foot (300 mm) of gravel was placed over the Paraseal sheets, and 2 ft (0.6 m) of water was ponded on top of the sheets. There was no outflow from any of the three test specimens over the entire 5-month testing period.

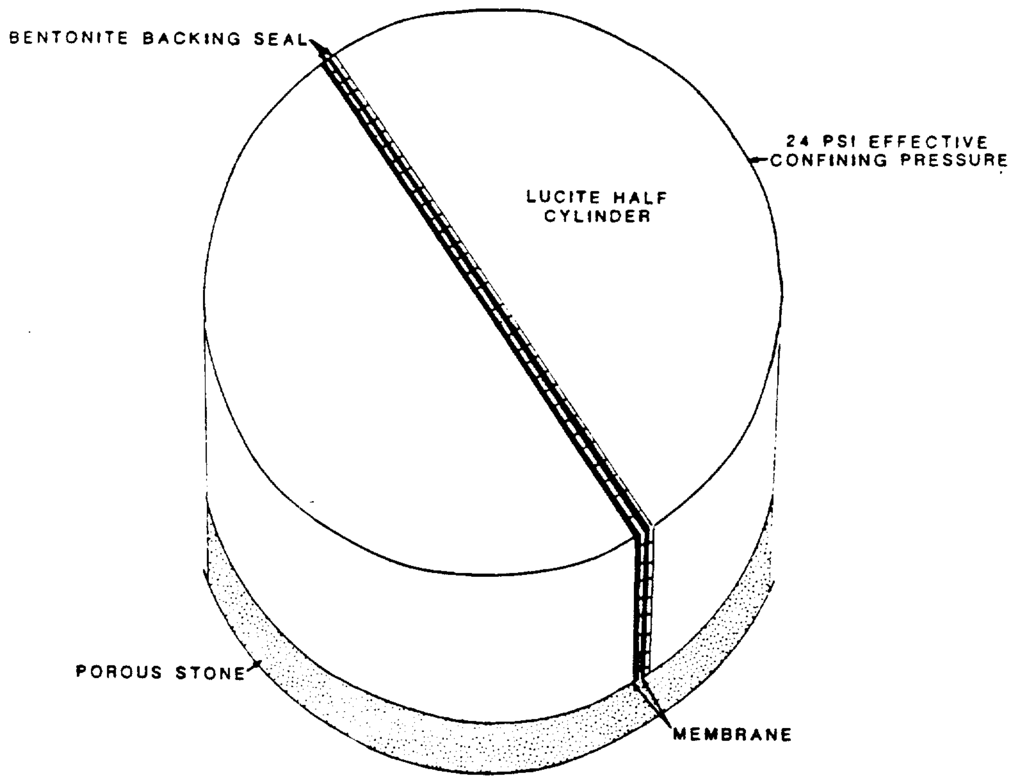


Figure 5.3. Schematic Diagram of Hydraulic Conductivity Test on Overlapped Seam of Paraseal (from Twin City Testing Corporation, 1986).

5.4 Examples of Use

Paraseal has been used primarily as a waterproofing material for building basements and, to a lesser extent, to line water-retention ponds.

Section 6.0

Bentofix

6.1 Description

Bentofix is manufactured by Naue-Fasertechnik Co. in Lubbecke, Germany. Information on the material was obtained from Scheu et al. (1990) and from personal communication with Robert M. Koerner.

Bentofix is a fiber-reinforced, bentonitic mat composed of a needlepunched, nonwoven geotextile as a cover, bentonite as a sealing element, and a needlepunched, nonwoven geotextile as a base layer (Fig. 6.1). The bentonite used for the manufacture of Bentofix is an activated sodium bentonite (a calcium-bentonite modified to a sodium bentonite) containing 70% montmorillonite. The geotextile layers are needlepunched together through the bentonite layer with a large amount of single stitches per square inch to form the Bentofix mat (Fig. 6.1).

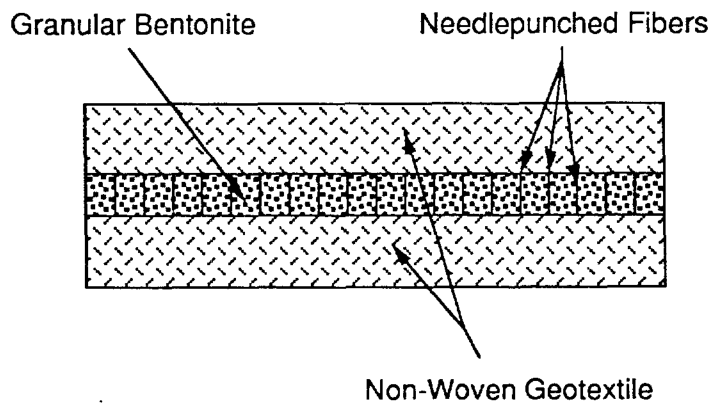


Figure 6.1. Schematic Diagram of Bentofix (from Scheu et al., 1990).

6.2 Installation

Bentofix may be placed on irregular surfaces, like slightly eroded embankments and channel beds. Larger pot holes must be filled with concrete and exposed buckles must be removed. Joints are made by overlapping the material. An overlap width of at least 12 in. (300 mm) is recommended. Wet granular bentonite is placed along the edge of the overlap sheet using a "U" shaped device that applies bentonite to the overlapped section in order to increase the integrity of the seam. A ballast layer of underwater concrete or crushed stones must be placed over the Bentofix mats in order to protect the material and keep the mats in place. The system can be applied throughout the year without any seasonal restrictions. Underwater installations are possible, as well.

6.3 Properties

6.3.1 Shear Strength

Direct shear tests were conducted by the Franzius Institute for Hydraulic Research and Coastal Engineering, University of Hannover, to determine the frictional behavior of Bentofix (Scheu et al., 1990). Bentofix specimens were placed between two sand layers in a direct shear box. The specimens were then saturated for three days under normal stresses of 50, 100, and 200 kPa and sheared until a total displacement of approximately 30 mm had been achieved. The time to failure was not reported by Scheu et al. Sliding took place within the bentonite layer, which caused the needle-punched threads to align themselves according to the direction of displacement. The Mohr-Coulomb diagram is shown in Figure 6.2. The cohesion was found to be 8 kPa (1.2 psi), and the angle of internal friction was 30°.

6.3.2 Hydraulic Properties

Hydraulic conductivity tests were carried out at the Institute for Foundation Engineering, Soil and Rock Mechanics of the Technical University, Munich (Scheu et al., 1990). Bentofix specimens were placed in triaxial cells, back-pressure saturated, and permeated with de-aired water. The hydraulic conductivity of water through the Bentofix sample was determined to be 1×10^{-9} cm/s under an unspecified effective confining stress.

6.3.3 Seams

The Franzius Institute (Scheu et al. 1990) conducted hydraulic conductivity tests on overlapped seams and overlaps with an intermediate bentonite layer. A large box with a drain at the bottom was used to contain the overlapped Bentofix samples. Water from an upper overflow reservoir was fed into the box where it then permeated through the overlapped samples and collected in a measuring glass located beneath the drain. A set of piezometer tubes were used to measure the change in head through the sample. From these experiments, the hydraulic conductivity of water through the Bentofix seams was determined to be 1×10^{-8} cm/s for overlapped sections of Bentofix containing bentonite between the sheets of Bentofix. The compressive stress applied to the overlapped area was not specified.

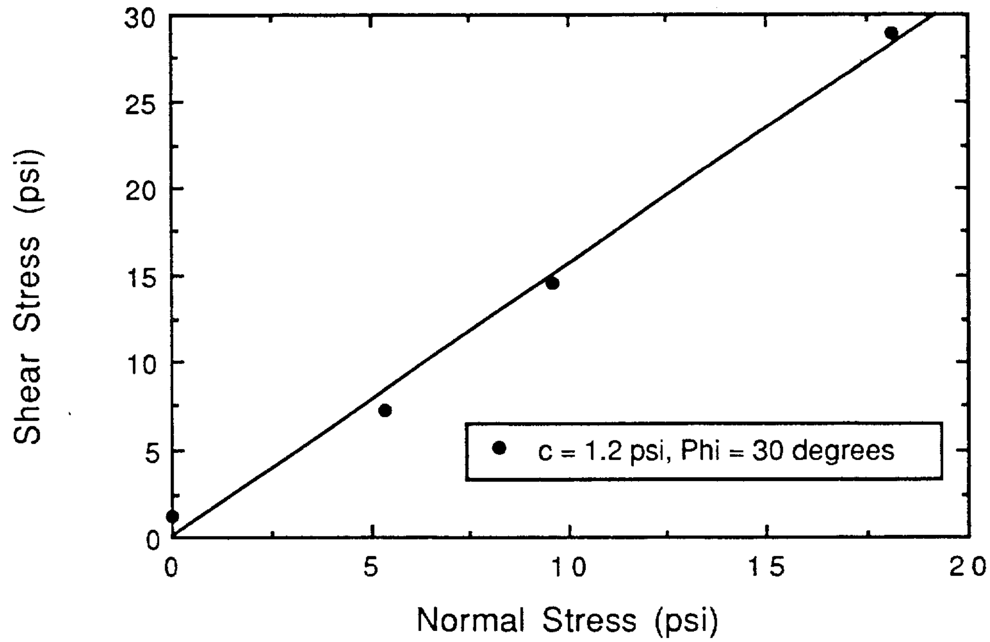


Figure 6.2 Results of Direct Shear Tests on Bentofix (from Scheu et al., 1990)

6.3.4. Mechanical Properties

Data on the mechanical properties of Bentofix could not be located.

6.4. Examples of Use

Bentofix can be used for many lining applications such as water reservoirs, channels, artificial lakes, dams, and landfills. The installation examples cited by Scheu et al. (1990) include a dam rehabilitation project and a chemical containment project.

The Lechkanal is a diversion canal built in the 1920's. The canal runs parallel to the river Lech, in Germany. The weirs and locks integrated in the diversion canal are used to generate electricity. Some portions of the 70-year-old canal are lined with man-made levees. Surface erosion and minor piping channels have developed along sections of the levees over a long period of time. Instability was solved using a double lining system that consisted of 140 mm of asphalt with a filter fabric drainage layer as the primary liner and Bentofix as the secondary liner.

A purification network was recently constructed at the Munich II Airport in Germany in order to collect and purify the runoff from runways. Deicing of airplanes with a mixture of glycol and hot water creates a hazardous runoff that has a potential to contaminate the underlying groundwater. The purification network at the Munich Airport consists of underground granular filters that support bacteria used to biologically purify the glycol and water mixture. The sealing element between the purification system and the groundwater is Bentofix.

Section 7.0

Other Alternative Barrier Materials

Some additional barrier materials that were identified during the workshop include:

1. Flyash-bentonite-soil mixtures;
2. Super-absorbant geotextiles (e.g., Fibersorb®);
3. Sprayed-on geomembranes (intended to form a composite with a compacted soil layer or bentonitic blanket);
4. Custom-made bentonite composites with geomembranes or geotextiles.

Fly-ash was not discussed because it did not appear to fit within the theme of thin, manufactured materials, which was the main focus of attention at the Workshop. Few details were presented concerning spayed-on products or custom-made bentonite composites, other than to indicate that sprayed-on products are promising and that all of the bentonitic blankets can be custom-designed and fabricated to meet particular needs, e.g., by using a thicker geotextile as a gas venting medium.

Information concerning Fibersorb® was supplied via a manufacturer's brochure and a technical report (STS Consultants, 1990). Fibersorb® is a lightweight geotextile containing thin, nonwoven, superabsorbent fibers and is manufactured by ARCO Chemical Company (3901 West Chester Pike, Newton Square, Pennsylvania 19073, telephone 215-359-5616). When water contacts the fibers, the resultant swelling fills voids and impedes water flow. Fibersorb® has been used primarily in protective clothing, in packaging, filters, or humidity control systems to seal out water or moisture, as industrial wipes, and has even been used as an emergency heat barrier in the case of a fire due to it's heat absorption capabilities.

STS Consultants (1990) conducted constant-head hydraulic conductivity tests on 4-in. (100-mm) diameter samples of Fibrosorb®. Four samples were tested:

Sample 1: Fibersorb® alone.

Sample 2: Fibersorb® overlying a 40 mil (1 mm) HDPE membrane that had a 1/8 in. (3 mm) diameter hole punched in it.

Sample 3: No Fibersorb®; a 40 mil (1 mm) HDPE membrane with a 1/8 in. (3 mm) diameter hole (the membrane placed between two porous discs and tested in the permeameter).

Sample 4: A 2-in. (50-mm) wide strip of Fibersorb® was placed between two sheets of 40 mil (1 mm) HDPE sheets to simulate use as a field seaming material.

The samples were back-pressure saturated and permeated at an effective confining stress of 0.15 kg/cm2 (15 kPa) with a head difference of 50 mm of water. Results are summarized in Table 7.1.

Table 7.1. Summary of Results of Hydraulic Conductivity Tests on Fibersorb® (from STS Consultants, 1990).

Sample	Test Conditions	Hydraulic Conductivity (cm/s)
1	Fibersorb®	3×10^{-8}
2	Fibersorb® with Defective HDPE	6×10^{-10}
3	Defective HDPE	1×10^{-6}
4	Fibersorb® between HDPE Overlap	5×10^{-9}

Section 8.0

Equivalency

The main function of low-permeability, compacted soil is either to restrict infiltration of water into buried waste (in cover systems) or to limit seepage of leachate from the waste (in liner systems). Other objectives may include enhancement of the efficiency of an overlying drainage layer, development of composite action with a flexible membrane liner (FML), adsorption and attenuation of leachate, restriction or gas migration, and others. In the case of a cover system, compacted soil must also have the ability to withstand subsidence and must be repairable if damaged by freezing, desiccation, or burrowing animals. For liner systems, the ability of the liner to withstand chemical degradation from the liquids to be contained. In addition, low-permeability compacted soil must have adequate shear strength to support itself on slopes and to support the weight of overlying materials or equipment.

An alternative barrier material, in order to be fully equivalent to a compacted soil layer, must serve the same functions as compacted soil. Due to inherent differences in the composition and construction of compacted soil and alternative barriers, the two categories of materials can never be "equivalent" in all possible respects. For example, compacted soil is usually from 2 to 5 ft (0.6 to 1.5 m) thick whereas the alternative barriers discussed in this report are typically no thicker than approximately 1/2 in. (13 mm). Due to differences in thickness, the alternative barrier material is bound to be more vulnerable to puncture than the much thicker layer of compacted soil.

Fundamental differences between compacted, low-permeability soil and the alternative barriers discussed in this report create inevitable differences in hydraulic properties, attenuation capacity, time of travel of chemicals, strength, desiccation resistance, freeze/thaw resistance, reaction to settlement, ease of repair, and useful life. Table 8.1 presents a qualitative list addressing the differences between compacted soil and alternative barrier materials.

When the potential use of an alternative barrier is evaluated for a particular project, the critical functions of the barrier should be identified. "Equivalency" should be evaluated on the basis of the critical parameters and not necessarily upon all potential areas of comparison. Further, it should be kept in mind that all liner materials have inherent advantages and disadvantages -- no one type of liner (including low-permeability, compacted soil) is a panacea. Some of the potential advantages of alternative barriers over low-permeability, compacted soil are as follows:

Table 8.1 Comparison of Differences in Alternative Barrier Materials

Compacted Soil	Alternative Barrier Materials
Thick (2 ft - 5 ft, or 0.6 - 1.5 m)	Thin (≤ 10 mm)
Field Constructed	Manufactured
Hard to Build Correctly	Easy to Build (Unroll & Place)
Impossible to Puncture	Possible to Damage and Puncture
Constructed with Heavy Equipment	Light Construction Equip. Can Be Used
Often Requires Test Pad at Each Site	Repeated Field Testing Not Needed
Site-Specific Data on Soils Needed	Manufactured Product; Data Available
Large Leachate-Attenuation Capacity	Small Leachate-Attenuation Capacity
Relatively Long Containment Time	Shorter Containment Time
Large Thickness Takes Up Space	Little Space Is Taken
Cost Is Highly Variable	More Predictable Cost
Soil Has Low Tensile Strength	Higher Tensile Strength
Can Desiccate and Crack	Can't Crack Until Wetted (after Construction)
Difficult to Repair	Not Difficult to Repair
Vulnerable to Freeze/Thaw Damage	Probably Less Vulnerable to Freeze/Thaw Damage
Performance Is Highly Dependent Upon Quality of Construction	Hydraulic Properties Are Less Sensitive to Construction Variabilities
Slow Construction	Much Faster Construction

- The installation of alternative barriers proceeds rapidly and relatively simply (construction of low-permeability, compacted soil is slower and requires a much higher level sophistication in construction technique);
- Because of the higher level of sophistication required for proper construction and protection of low-permeability, compacted soil liners, the alternative barriers may provide a more predictable end-product for situations in which the quality of construction of a compacted soil liner cannot be assured;
- Alternative barriers, which cost approximately $0.50 to $1.00 per square foot ($5.50 to $11 per square meter) installed, are often less expensive than compacted soil and can be installed at a more predictable cost than compacted soil liners;
- Alternative barriers occupy much less volume than compacted soil, which has three ramifications: (1) more space is available in landfills for waste with the thin, alternative barrier; (2) fewer truckloads of delivered material are needed for the alternative barrier compared to compacted soil liners, which can have important implications for transportation impacts when soil must be obtained from off-site; and (3) because the alternative barrier weighs less than the thicker compacted soil, less settlement of underlying waste (for cover applications) would result with alternative barriers;
- Alternative barriers can be installed with light-weight equipment, which is particularly advantageous for placing liners on top of geosynthetic components, e.g., a primary liner placed on top of a secondary leachate collection and removal system;
- Once an alternative barrier material is thoroughly characterized and field tested, there should be no need to retest it unless the materials or installation procedures change;
- Some alternative barrier materials possess unique self-healing characteristics derived from the expansive nature of bentonite.

The alternative barriers are not without caveats. Some of the potential disadvantages of alternative barriers include the following:

- There is a general lack of data and independent research on the alternative barrier materials;
- Field experience is very limited for most of the alternative barrier materials, and field performance data is virtually nonexistent;
- Because the alternative barriers are thin, they are vulnerable to damage from puncture, e.g., from traffic or construction equipment such as bulldozers, over unprotected or improperly protected sections or during placement of cover materials;
- Sodium bentonite is more vulnerable to adverse chemical reactions from leachate than the clay minerals found in most compacted soil liners;
- The effects of settlement of underlying waste upon the hydraulic integrity of the materials has not been evaluated;
- The effects of cyclic wetting and drying of the materials upon bulk shrinkage has not been adequately investigated;
- Characterization of performance of overlapped seams under actual field conditions is incomplete;
- The low shear strength of bentonite raises questions about the stability of alternative barrier materials containing bentonite when such materials are placed on slopes.

One of the areas of application that was relatively uncontroversial was the use of alternative barrier materials as a back-up to a flexible membrane liner in the primarily liner of a double liner system. The EPA does not require a clay liner in the uppermost liner for doubly-lined, hazardous waste landfills; an alternative barrier used in this situation involves placing an extra component beyond the minimum requirements.

Section 9.0

Concerns

During a concluding open discussion session of the Workshop, attendees voiced their concerns regarding the behavior of alternative barriers and discussed informational needs that would provide a better understanding of the characteristics of these manufactured materials. The major concerns, expressed in question form, were as follows:

- Concerning stability -- Should alternative barriers be used in landfill caps having slopes equal to or greater that 10°? And if so, would reinforcing the cover soil, e.g., with a geogrid, and construction of an anchor eliminate the instability problem? Do engineers have sufficient experience with and knowledge of these materials to allow building on slopes with a high level of confidence?
- Concerning temporary versus permanent use in a cap -- Alternative materials should be seriously considered as a temporary cap for some RCRA or CERCLA sites for which settlement that would damage a final cover is anticipated. How would the alternative barrier material react to significant settlement? Although the alternative barrier would appear to be easy to repair, are there practical problems in repairing the materials that have not been anticipated?
- Concerning application in dry climates -- Compacted soils have limited self-healing capability, especially at low stress, and are vulnerable to damage from desiccation after they are constructed. Alternative barrier materials are less vulnerable to damage from desiccation after they are installed because they are installed dry. Should alternative barriers be given stronger consideration for applications in arid regions? If so, are there other problems with use of alternative barriers in arid regions, such as bulk shrinkage upon drying, that might prove to be significant?
- Concerning installation -- What happens when it rains during construction? What happens to hydraulic conductivity if the material is wetted before overburden is placed? How much overburden is needed to form an adequate seam? What if the alternative barrier is placed on a small pebble; will the bentonite be pushed aside and cause and increase in permeability? A great deal of

care would appear to be necessary to install the alternative barrier material correctly -- is it reasonable to assume that the necessary degree of care will be exercised in the installation of the alternative barrier material?

- Concerning application in the field -- The following concerns were expressed. There is a strong need to see real performance; laboratory data alone are not enough. Possibly a slow approach to the use of alternative barrier materials would be wise. The least controversial applications of alternative barrier materials appear to be landfill covers on reasonably flat surfaces, primary lining systems for which there is a conventional FML/compacted clay secondary liner located beneath the primary liner, and liner or cover systems in arid regions. In these situations, there is less doubt about performance, less risk involved, and the performance may be easier to assess.
- Concerning field performance -- Routine methods to monitor actual performance of field installations is badly needed. Installation of large (e.g., 2 m) diameter collection lysimeters underneath these barrier materials is feasible and is encouraged to provide a credible base of data on field performance.

Section 10.0

Informational Needs

The workshop was concluded by compiling a list of issues for which more information is needed. The "needs" were established with the goal of generating the data that design engineers, owner/operators, and regulatory personnel require to have a high level of confidence that alternative barrier materials will provide the required environmental protection functions in waste management applications. The following is a condensed version of this list.

1. Shear Strength
   a. Interfacial friction with other liner/cover components
   b. Long term performance
   c. Water diffusion effects; point wetting
   d. Standardized testing procedures
   e. Laboratory versus field scale
2. Hydraulic Properties
   a. Hydraulic conductivity
   b. Attenuation capacity
   c. Hydration with water versus leachate
   d. Composite action; does a composite seal form?
   e. Migration of bentonite
   f. Laboratory versus field scale
3. Environmental Effects
   a. Freeze/thaw resistance
   b. Desiccation resistance
   c. Effects of settlement
   d. Self healing capabilities
   e. Effects of rock beneath alternative barrier
   f. Laboratory versus field scale
4. Seams
   a. Hydraulic properties
   b. Strength
   c. Effects of settlement

   d. Wrinkled seam (wrinkle of liner above or below and of alternative barrier itself)
   e. Laboratory versus field scale
5. Quality Assurance/Quality Control
   a. Manufacture
   b. Transportation
   c. Installation
6. Applications
   a. Caps versus liners
   b. Humid versus arid regions
   c. Compressible and incompressible waste
7. Thermal Effects
   a. Differential expansion of alternative barrier with other liner materials, especially with HDPE/bentonite composite, that could cause wrinkles or delamination of materials
   b. Shrinkage of materials upon drying, causing a reduction in overlap width
8. Mechanical Properties
9. Comparison with Compacted Soil
10. Useful Life; Aging.

The list was prioritized in the following order:

1. Shear strength
2. Hydraulic properties
3. Seams
4. Useful life.

More information about these research needs, plus other issues not listed above, is expected to become available over the next few months and years. Individuals with information are encouraged to pass that information along to David E. Daniel, University of Texas, Department of Civil Engineering, Austin, TX 78712, or to Walter E. Grube, Jr., U. S. Environmental Protection Agency, Risk Reduction Engineering Laboratory, Cincinnati, OH 45268. Of particular interest are unpublished data, for example, developed for a particular project such as a DOE cover project, that might otherwise not be widely disseminated.

Many individuals attending the Workshop expressed a desire to hold a similar Workshop in 1 to 2 years to present and to discuss new information. If a significant base of new data is developed, the new information would likely be the focal point of discussions in the next Workshop.

Section 11.0

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