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Design and Construction of RCRA/CERCLA Final Covers

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CHAPTER 1

OVERVIEW OF COVER SYSTEMS FOR WASTE MANAGEMENT FACILITIES

INTRODUCTION

Proper closure is essential to complete a filled hazardous waste landfill. Research has established minimum requirements needed to meet the stringent, necessary, closure regulations in the United States. In designing the landfill cover, the objective is to limit the infiltration of water to the waste so as to minimize creation of leachate that could possibly escape to ground-water sources.

Minimizing leachates in a closed waste management unit requires that liquids be kept out and that the leachate that does exist be detected, collected, and removed. Where the waste is above the ground-water zone, a properly designed and maintained cover can prevent (for practical purposes) water from entering the landfill and, thus, minimize the formation of leachate.

The cover system must be devised at the time the site is selected and the plan and design of the landfill containment structure is chosen. The location, the availability of soil with a low permeability or hydraulic conductivity, the stockpiling of good topsoil, the availability and use of geosynthetics to improve performance of the cover system, the height restrictions to provide stable slopes, and the use of the site after the postclosure care period are typical considerations. The goals of the cover system are to minimize further maintenance and to protect human health and the environment.

Subparts G, K, and N of the Resource Conservation and Recovery Act (RCRA) Subtitle C regulations form the basic requirements for cover systems being designed and constructed today. Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) regulations refer to the RCRA Subtitle C regulations but other criteria, primarily approved state requirements, also have to be evaluated for applicability. The proposed RCRA Subtitle D regulations base cover requirements primarily on the hydraulic conductivity of the bottom liner.

RECOMMENDED DESIGN FOR SUBTITLE C FACILITIES

After the hazardous waste management unit is closed, the U.S. Environmental Protection Agency (EPA) recommends (1) that the final cover (Figure 1-1) consist of, from bottom to top:

1. *A Low Hydraulic Conductivity Geomembrane/Soil Layer.* A 60-cm (24-in.) layer of compacted natural or amended soil with a hydraulic conductivity of 1×10^{-7} cm/sec in intimate contact with a minimum 0.5-mm (20-mil) geomembrane liner.
2. *A Drainage Layer.* A minimum 30-cm (12-in.) soil layer having a minimum hydraulic conductivity of 1×10^{-2} cm/sec, or a layer of geosynthetic material having the same characteristics.
3. *A Top, Vegetation/Soil Layer.* A top layer with vegetation (or an armored top surface) and a minimum of 60 cm (24 in.) of soil graded at a slope between 3 and 5 percent.

Because the design of the final cover must consider the site, the weather, the character of the waste, and other site-specific conditions, these minimum recommendations may be altered providing the alternative design is equivalent to the EPA-recommended design or will meet the intent of the regulations. EPA encourages design innovation and will accept an alternative design provided the owner or operator demonstrates the new design's equivalency. For example, in extremely arid regions, a gravel top surface might compensate for reduced vegetation, or the middle drainage layer might be expendable. Where burrowing animals might damage the geomembrane/low hydraulic conductivity soil layer, a biotic barrier layer of large-sized cobbles may be needed above it. Where the type of waste may create gases, soil or geosynthetic vent structures would need to be included.

Settlement and subsidence should be evaluated for all covers and accounted for in the final cover plans. The current operating procedures for RCRA Subtitle C facilities (e.g., banning of liquids and partially filled drums of liquids) usually do not present major settlement or subsidence issues. For RCRA Subtitle D facilities, however, the normal decomposition of the waste will invariably result in settlement and subsidence. Settlement and subsidence can be significant, and special care may be required in designing the final cover system. The cover design process should consider the stability of all the waste layers and their intermediate soil covers, the soil and foundation materials beneath the landfill site, all the

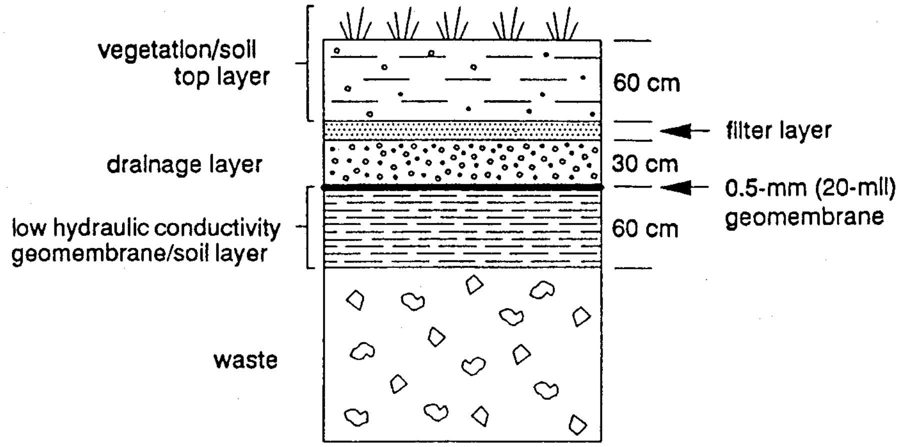

Figure 1-1. EPA-recommended landfill cover design (1).

liner and leachate collection systems, and all the final cover components. When a significant amount of settlement and subsidence is expected within 2 to 5 years of closure, an interim cover that protects human health and the environment might be proposed. Then when settlement/subsidence is essentially complete, the interim cover could be replaced or incorporated into a final cover.

Low Hydraulic Conductivity Layer

The function of the composite low hydraulic conductivity layer, composed of soil and a geomembrane, is to prevent moisture movement downward from the overlying drainage layer.

Compacted Soil Component

EPA recommends a test pad be constructed before the low hydraulic conductivity soil layer is put in place to demonstrate that the compacted soil component can achieve a maximum hydraulic conductivity of 1×10^{-7} cm/sec. To ensure that the design specifications are attainable, a test pad uses the same soil, equipment, and procedures to be used in constructing the low hydraulic conductivity layer. For Subtitle D facilities, the test fill should be constructed on part of the solid waste material to determine the impact of compacting soil on top of less resistive municipal solid waste.

The low hydraulic conductivity soil component placed over the waste should be at least 60-cm (24-in.) deep; free of detrimental rock, clods, and other soil debris; have an upper surface with a 3 percent maximum slope; and be below the maximum frost line. The surface should be smooth so that no small-scale stress points are created for the geomembrane.

In designing the low hydraulic conductivity layer, the causes of failure—subsidence, desiccation cracking, and freeze/thaw cycling—must be considered. Most of the settling will have taken place by the time the cover is put into place, but there is still a potential for further subsidence. Although estimating this potential is difficult, information about voids and compressible materials in the underlying waste will aid in calculating subsidence.

A soil with low cracking potential should be selected for the soil component of the low hydraulic conductivity layer. The potential for desiccation cracking of compacted clay depends on the physical properties of the compacted clay, its moisture content, the local climate, and the moisture content of the underlying waste.

Because freeze/thaw conditions can cause soil cracking, lessen soil density, and lessen soil strength, this entire low hydraulic conductivity/geomembrane layer should be below the depth of the maximum frost penetration. In northern areas, then, the maximum depth of the top vegetation/soil layer would be greater than the recommended minimum of 60 cm (24 in.).

Penetrating this low hydraulic conductivity/geomembrane soil layer with gas vents or drainage pipes should be kept to a minimum. Where a vent is necessary, there should be a secure, liquid-tight seal between the vent and the geomembrane. If settlement or subsidence is a major concern, this seal must be designed for flexibility to allow for vertical movement.

Geomembrane

The geomembrane placed on the smooth, even, low hydraulic conductivity layer should be at least 0.5-mm (20-mils) thick. The minimum slope surface should be 3 percent after any settlement of the soil layer or sub-base material. Stress situations such as bridging over subsidence and friction between the geomembrane and other cover components (i.e., compacted soil, geosynthetic drainage material, etc.), especially on side slopes, will require special laboratory tests to ensure the design has incorporated site-specific materials.

Drainage Layer

The drainage layer should be designed to minimize the time the infiltrated water is in contact with the bottom, low hydraulic conductivity layer and, hence, to lessen the potential for the water to reach the waste (see Figure 1-1). Water that filters through the top layer is intercepted and rapidly moved to an exit drain, such as by gravity flow to a toe drain.

If the granular material in the drainage layer is sand, the minimum requirements are that it should be at least 30-cm (12-in.) deep with a hydraulic conductivity of 1 x 10^{-2} cm/sec or greater. Drainage pipes should not be placed in any manner that would damage the geomembranes.

If geosynthetic materials are used in the drainage layer, the same physical and hydraulic requirements should be met, e.g., equivalency in hydraulic transmissivity, longevity, compatibility with geomembrane, compressibility, conformance to surrounding materials, and resistance to clogging. Geosynthetic materials are gaining increased use and understanding of their performance. Manufacturers are also continuing to improve the basic resin properties to improve their long-term durability. The net result is that organizations such as the American Society of Testing Materials (ASTM) and the Geosynthetic Research Institute (GRI), Drexel University, Philadelphia, Pennsylvania, are continually developing new evaluation procedures to better correlate with design and field experiences.

Between the bottom of the top-layer soil and the drainage-layer sand, a granular or geosynthetic filter layer should be included to prevent the drainage layer from clogging by top-layer fines. The criteria established for the grain size of granular filter sand are designed to minimize the migration of fines from the overlying top layer into the drainage layer. (For information on filter criteria, refer to the EPA Technical Guidance Document [1].) ASTM test procedures have also been established to evaluate particulate clogging potential of geosynthetics.

Vegetation/Soil Top Layer

Vegetation Layer

The upper layer of the two-component top layer (Figure 1-1) should be vegetation (or another surface treatment) that will allow runoff from major storms while inhibiting erosion. Vegetation over soil (part of which is topsoil) is the preferred system, although, in some areas, vegetation may be unsuitable.

The temperature- and drought-resistant vegetation should be indigenous; have a root system that does not extend into the drainage layer; need no maintenance; survive in low-nutrient soil; and have sufficient density to control the rate of erosion to the recommended level of less than 5.5 MT/ha/yr (2 ton/acre/yr).

The surface slope should be the same as that of the underlying soils; at least 3 percent but no greater than 5 percent. To support the vegetation, this top layer should be at least 60-cm (24-in.) deep and include at least 15-cm (6-in.) of topsoil. To help the plant roots develop, this layer should not be compacted. In some northern climates, this top layer may need to be more than the minimum 60 cm (24 in.) to ensure that the bottom low hydraulic conductivity layer remains below the frost zone.

Where vegetation cannot be maintained, particularly in arid areas, other materials should be selected to prevent erosion and to allow for surface drainage. Asphalt and concrete are apt to deteriorate because of thermal-caused cracking or deform because of subsidence. Therefore, a surface layer 13 to 25-cm (5 to 10-in.) deep of 5 to 10-cm (2 to 4-in.) stones or cobbles would be more effective. Although cobbles are a one-way valve and allow rain to infiltrate, this phenomenon would be of less concern in arid areas. In their favor, cobbles resist wind erosion well.

Soil Layer

The soil in this 60-cm (24-in.) top layer should be capable of sustaining nonwoody plants, have an adequate water-holding capacity, and be sufficiently deep to allow for expected, long-term erosion losses. A medium-textured soil such as a loam would fit these requirements. If the landfill site has sufficient topsoil, it should be stockpiled during excavation for later use.

The final slopes of the cover should be uniform and at least 3 percent, and should not allow erosion rills and gullies to form. Slopes greater than 5 percent will promote erosion unless controls are built in to limit erosion to less than 5.5 MT/ha/yr (2 ton/acre/yr). The U.S. Department of Agriculture's (USDA's) Universal Soil Loss Equation is recommended as the tool to evaluate erosion potential.

Optional Layers

Although other layers may be needed on a site-specific basis, the common optional layers are those for gas vents and for a biotic barrier layer (Figure 1-2).

Gas Vent Layer

The gas vent layer should be at least 30-cm (12-in.) thick and be above the waste and below the low hydraulic conductivity layer. Coarse-grained porous material, similar to that used in the drainage layer or equivalent-performing synthetic material, can be used.

The perforated, horizontal venting pipes should channel gases to a minimum number of vertical risers located at a high point (in the cross section) to promote gas ventilation. To prevent clogging, a granular or geotextile filter may be needed between the venting and the low hydraulic conductivity soil geomembrane layers.

As an alternative, vertical, standpipe gas collectors can be built up as the landfill is filled with waste. These standpipes, which may be constructed of concrete, can be 30 cm (12 in.) or more in diameter and may also be used to provide access to measure leachate levels in the landfill.

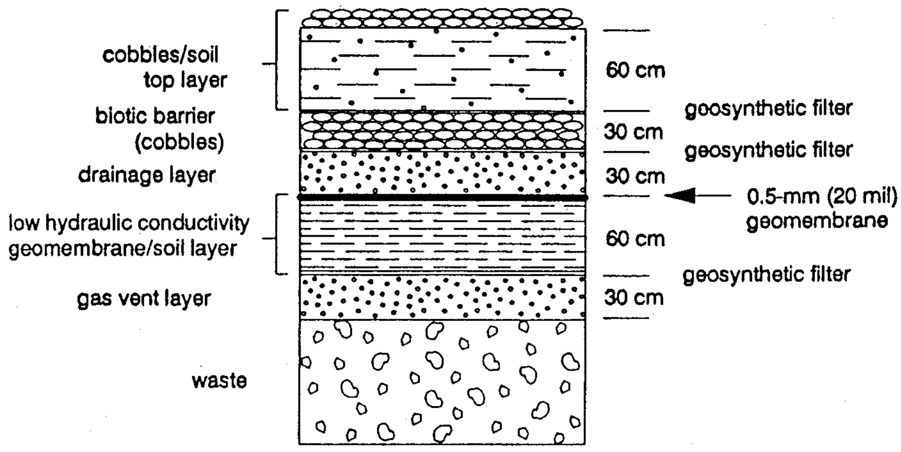

Figure 1-2. EPA-recommended landfill cover with options (1).

Biotic Layer

Plant roots or burrowing animals (collectively called biointruders) may disrupt the drainage and the low hydraulic conductivity layers to interfere with the drainage capability of the layers. A 90-cm (3-ft.) biotic barrier of cobbles directly beneath the top vegetation layer may stop the penetration of some deep-rooted plants and the invasion of burrowing animals. Most research on biotic barriers has been done in, and is applicable to, arid areas. Geosynthetic products that incorporate a time-released herbicide into the matrix or on the surface of the polymer may also be used to retard plant roots. The longevity of these products requires evaluation if the cover system is to serve for longer than 30 to 50 years.

SUBTITLE D COVERS

The cover system in nonhazardous waste landfills (Subtitle D) will be a function of the bottom liner system and the liquids management strategy for the specific site. If the bottom liner system contains a geomembrane, then the cover system should contain a geomembrane to prevent the "bathtub" effect. When the bottom liner is less permeable than the cover system, e.g., geomembrane on the bottom and natural soil on the top, the facility will "fill up" with infiltration water (through the cover) unless an active leachate removal system is in place. Likewise, if the bottom liner system is a natural soil liner, then the cover system barrier should be hydraulically equivalent to or less than the bottom liner system. A geomembrane used in the cover will prevent the infiltration of moisture to the waste below and may contribute to the collection of waste decomposition gases, therefore necessitating a gas-vent layer.

There are at least two options to consider under a liquids management strategy, mummification and recirculation. In the *mummification* approach the cover system is designed, constructed, and maintained to prevent moisture infiltration to the waste below. The waste will eventually approach and remain in a state of "mummification" until the cover system is breached and moisture enters the landfill. A continual maintenance program is necessary to maintain the cover system in a state of good repair so that the waste does not decompose to generate leachate and gas.

The *recirculation* concept results in the rapid physical, chemical, and biological stabilization of the waste. To accomplish this, a moisture balance is maintained within the landfill that will accelerate these stabilization processes. This approach requires geomembranes in both the bottom and top control systems to prevent leachate from getting out and excess moisture from getting in. In addition, the system needs a leachate collection and removal system on the bottom and a leachate injection system on the top, maintenance of this system for a number of years (depending on the size of the facility), and a gas collection system to remove the waste decomposition gases. In a modern landfill facility, all of these elements, except the leachate injection system, would probably be available. The benefit of this approach is that, after stabilization, the facility should not require further maintenance. A more important advantage is that the decomposed and stabilized waste may be removed and used like compost, the plastics and metals could be recycled, and the site used again. If properly planned and operated in this manner, several cells could serve all of a community's waste management needs.

A natural soil material may be used in a cover system when the bottom liner system is also natural soils and the regulatory requirements will permit. A matrix of soil characteristics (using either USDA or USCS) and health, aesthetics, and site usage characteristics can be developed to provide information on which soil or combination of soils will be the most beneficial.

Health considerations demand the evaluation of each soil type to minimize vector breeding areas and attractiveness to animals. The soil should minimize moisture infiltration (best accomplished by fine grain soils) while allowing gas movement (coarse grain soils are best). This desired combination of seemingly opposite soil properties suggests a layered system. The soil should also minimize fire potential.

Aesthetic considerations include minimizing blowing of paper and other waste, controlling odors, and providing a sightly appearance. All landfill operators strive to be good neighbors and these considerations are very important for community relations.

The landfill site may be used for a variety of activities after closure. For this reason, cover soils should minimize settlement and subsidence, maximize compaction, assist vehicle support and movement, allow for equipment workability under all weather conditions, and allow healthy vegetation to grow. The future use of the site should be considered at the initial landfill design stages so that appropriate end-use design features can be incorporated into the cover during the active life of the facility.

CERCLA COVERS

The Superfund Amendments and Reauthorization Act of 1986 (SARA) adopts and expands a provision in the 1985 National Contingency Plan (NCP) that remedial actions must at least attain applicable or relevant and appropriate requirements (ARARs). Section 121(d) of CERCLA, as amended by SARA, requires attainment of federal ARARs and of state ARARs in state environmental or facility siting laws when the state requirements are promulgated, more stringent than federal laws, and identified by the state in a timely manner.

CERCLA facilities require information on whether or not the site is under the jurisdiction of RCRA regulations. The cover system design can then be developed based on appropriate regulations.

RCRA Subtitle C requirements for treatment, storage, and disposal facilities (TSDFs) will frequently be ARARs for CERCLA actions, because RCRA regulates the same or similar wastes as those found at many CERCLA sites, covers many of the same activities, and addresses releases and threatened releases similar to those found at CERCLA sites. When RCRA requirements are ARARs, only the *substantive requirements* of RCRA must be met if a CERCLA action is to be conducted on site. Substantive requirements are those requirements that pertain directly to actions or conditions in the environment. Examples include performance standards for incinerators (40 CFR 264.343), treatment standards for land disposal of restricted waste (40 CFR 268), and concentration limits, such as maximum contaminant levels (MCLs). Onsite actions do not require RCRA permits or compliance with *administrative requirements*. Administrative requirements are those mechanisms that facilitate the implementation of the substantive requirements of a statute or regulation. Examples include the requirements for preparing a contingency plan, submitting a petition to delist a listed hazardous waste, recordkeeping, and consultations. CERCLA actions to be conducted off site must comply with both substantive and administrative RCRA requirements.

APPLICABILITY OF RCRA REQUIREMENTS

RCRA Subtitle C requirements for the treatment, storage, and disposal of hazardous waste are applicable for a Superfund remedial action if the following conditions are met (2):

1. The waste is a RCRA hazardous waste, and either:
2. The waste was initially treated, stored, or disposed of after the effective date of the particular RCRA requirement

   or

   The activity at the CERCLA site constitutes treatment, storage, or disposal, as defined by RCRA.

For RCRA requirements to be applicable, a Superfund waste must be determined to be a listed or characteristic hazardous waste under RCRA. A waste that is hazardous because it once exhibited a characteristic (or a media containing a waste that once exhibited a characteristic) will not be subject to Subtitle C regulation if it no longer exhibits that characteristic. A listed waste may be delisted if it can be shown not to be hazardous based on the standards in 40 CFR 264.22. If such a waste will be shipped off site, it must be delisted through a rulemaking process. To delist a RCRA hazardous waste that will remain on site at a Superfund site, however, only the substantive requirements for delisting must be met.

Any environmental media (i.e., soil or ground water) contaminated with a listed waste is not a hazardous waste, but must be managed as such until it no longer contains the listed waste—generally when constituents from the listed waste are at health-based levels. Delisting is not required.

To determine whether a waste is a listed waste under RCRA, it is often necessary to know the source of that waste. For any Superfund site, if determination cannot be made that the contamination is from a RCRA hazardous waste, RCRA requirements will not be applicable. This determination can be based on testing or on best professional judgment (based on knowledge of the waste and its constituents).

A RCRA requirement will be applicable if the hazardous waste was treated, stored, or disposed of after the effective date of the particular requirement. The RCRA Subtitle C regulations that established the hazardous waste management system first became effective on November 19, 1980. Thus, RCRA regulations will not be applicable to wastes disposed of before that date, unless the CERCLA action itself constitutes treatment, storage, or disposal (see below). Additional standards have been issued since 1980; therefore, applicable requirements may vary somewhat, depending on the specific date on which the waste was disposed.

RCRA requirements for hazardous wastes will also be applicable if the response activity at the Superfund site constitutes treatment, storage, or disposal, as defined under RCRA. Because remedial actions frequently involve grading, excavating, dredging, or other measures that disturb contaminated material, activities at Superfund sites may constitute disposal, or placement, of hazardous waste. Disposal of hazardous waste, in particular, triggers a number of significant requirements, including closure requirements and land disposal restrictions, which require treatment of wastes prior to land disposal. (See *Guides on Superfund Compliance with Land Disposal Restrictions*, OSWER Directives 9347.3-01FS through 9237.3-06FS, for a detailed description of these requirements.)

EPA has determined that disposal occurs when wastes are placed in a land-based unit. However, movement within a unit does not constitute disposal or placement, and at CERCLA sites, an area of contamination (AOC) can be considered comparable to a unit. Therefore, movement within an AOC does not constitute placement.

Relevant and Appropriate RCRA Requirements

RCRA requirements that are not applicable may, nonetheless, be relevant and appropriate, based on site-specific circumstances. For example, if the source or prior use of a CERCLA waste is not identifiable, but the waste is similar in composition to a known, listed RCRA waste, the RCRA requirements may be potentially relevant and appropriate, depending on other circumstances at the site. The similarity of the waste at the CERCLA site to RCRA waste is not the only, nor necessarily the most important, consideration in the determination. An in-depth, constituent-by-constituent analysis is generally neither necessary nor useful, since most RCRA requirements are the same for a given activity or unit, regardless of the specific composition of the hazardous waste.

The determination of relevance and appropriateness of RCRA requirements is based instead on the circumstances of the release, including the hazardous properties of the waste, its composition and matrix, the characteristics of the site, the nature of the release or threatened release from the site, and the nature and purpose of the requirement itself. Some requirements may be relevant and appropriate for certain areas of the site, but not for other areas. In addition, some RCRA requirements may be relevant and appropriate at a site, while others are not, even for the same waste. For example, at one site minimum technology requirements may be considered relevant and appropriate for an area receiving waste because of the high potential for migration of contaminants in hazardous levels to ground water, but not for another area that contains relatively immobile waste. Land disposal restrictions at the same site may not be relevant and appropriate for either area because the required treatment technology is not appropriate, given the matrix of the waste. Only those requirements that are determined to be both relevant and appropriate must be attained.

State Equivalency

A state may be authorized to administer the RCRA hazardous waste program in lieu of the federal program provided the state has equivalent authority. Authorization is granted separately for the basic RCRA Subtitle C program, which includes permitting and closure of TSDFs; for regulations promulgated pursuant to the Hazardous and Solid Waste Amendments (HSWA), such as land disposal restrictions; and for other programs, such as delisting of hazardous wastes. If a site is located in a state with an authorized RCRA program, the state's promulgated RCRA requirements will replace the equivalent federal requirements as potential ARARs.

An authorized state program may also be more stringent than the federal program. For example, a state may have more stringent test methods for characteristic wastes, or may list more wastes as hazardous than the federal program does. Therefore, it is important to determine whether laws in an authorized state go beyond the federal regulations.

Closure

For each type of unit regulated under RCRA, Subtitle C regulations contain standards that must be met when a unit is closed. For treatment and storage units, the closure standards require that all hazardous waste and hazardous waste residues be removed. In addition to the option of closure by removal, called *clean closure*, units such as landfills, surface impoundments, and waste piles may be closed as disposal or landfill units with waste in place, referred to as *landfill closure*. Frequently, the closure requirements for such land-based units will be either applicable or relevant and appropriate at Superfund sites.

Applicability of Closure Requirements

The basic prerequisites for applicability of closure requirements are (1) the waste must be hazardous waste; and (2) the unit (or AOC) must have received waste after the RCRA requirements became effective, either because of the original date of disposal or because the CERCLA action constitutes disposal. When RCRA closure requirements are applicable, the regulations allow only two types of closure:

- *Clean Closure.* All waste residues and contaminated containment system components (e.g., liners), contaminated subsoils, and structures and equipment contaminated with waste leachate must be removed and managed as hazardous waste or decontaminated before the site management is completed [see 40 CFR 264.111, 264.228(a)].

- *Landfill Closure*. The unit must be capped with a final cover designed and constructed to:
  - Provide long-term minimization of migration of liquids.
  - Function with minimum maintenance.
  - Promote drainage and minimize erosion.
  - Accommodate settling and subsidence.
  - Have a hydraulic conductivity less than or equal to any bottom liner system or natural subsoils present.

Clean closure standards assume the site will have unrestricted use and require no maintenance after the closure has been completed. These standards are often referred to as the "eatable solid, drinkable leachate" standards. In contrast, disposal or landfill closure standards require postclosure care and maintenance of the unit for at least 30 years after closure. Postclosure care includes maintenance of the final cover, operation of a leachate and removal system, and maintenance of a ground-water monitoring system [see 40 CFR 264.117, 264.228(b)].

EPA has prepared several guidance documents on closure and final covers (1, 3). These guidance documents are not ARARs, but are to be considered for CERCLA actions and may assist in complying with these regulations. The performance standards in the regulation may be attained in ways other than those described in guidance, depending on the specific circumstances of the site.

Relevant and Appropriate Closure Requirements

If they are not applicable, RCRA closure requirements may be determined to be relevant and appropriate. There is more flexibility in designing closure for relevant and appropriate requirements because the Agency has the flexibility to determine which requirements in the closure standards are relevant and appropriate. Under this scenario, a hybrid closure is possible. Depending on the site circumstances and the remedy selected, clean closure, landfill closure, or a combination of requirements from each type of closure may be used.

The proposed revisions to the NCP discuss the concept of hybrid closure (53 FR 51446). The NCP illustrated the following possible hybrid closure approaches:

- *Hybrid-Clean Closure*. Used when leachate will not impact the ground water (even though residual contamination and leachate are above health-based levels) and contamination does not pose a direct contact threat. With hybrid-clean closure:
  - No covers or long-term management are required.
  - Fate and transport modeling and model verification are used to ensure that ground water is usable.
  - A property deed notice is used to indicate the presence of hazardous substances.
- *Hybrid-Landfill Closure*. Used when residual contamination poses a direct contact threat, but does not pose a ground-water threat. With hybrid-landfill closure:
  - Covers, which may be permeable, are used to address the direct contact threat.
  - Limited long-term management includes site and cover maintenance and minimal ground-water monitoring.
  - Institutional controls (e.g., land-use restrictions or deed notices) are used as necessary.

The two hybrid closure alternatives are constructs of applicable laws but are not themselves promulgated at this time. These alternatives are possible when RCRA requirements are relevant and appropriate, but not when closure requirements are applicable.

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CHAPTER 2
SOILS USED IN COVER SYSTEMS

INTRODUCTION

This chapter describes several important aspects of soils design for cover systems over waste disposal units and site remediation projects. The chapter focuses on three critical components of the cover system: composite action of soil with a geomembrane liner; design and construction of low hydraulic conductivity layers of compacted soil; and mechanisms by which low hydraulic conductivity layers can be damaged. In addition, types of soils used for liquid drainage or gas collection also will be discussed.

TYPICAL COVER SYSTEMS

Cover systems perform many functions. One of the principal objectives of a cover system is to reduce leaching of contaminants from buried wastes or contaminated soils by minimizing water infiltration. Cover systems also promote good surface drainage and maximize runoff. In addition, they restrict or control gas migration, or, at some sites, enhance gas recovery. Finally, cover systems provide a physical separation between buried wastes or contaminated materials and animals and plant roots. When designing a cover system, all of these requirements, plus others, typically must be considered.

As presented and discussed in Chapter 1, Figures 1-1 and 1-2 illustrate two typical cover profiles (see pages 1-3 and 1-7). Figure 1-1 illustrates the minimum cover profile recommended by EPA for hazardous waste. Many of the layers shown in the figure are composed of soils or have soil components. Each layer has a different purpose and the materials must be selected and the layer designed to perform the intended function:

- *Topsoil* - The topsoil supports vegetation (which minimizes erosion and maximizes evapotranspiration), separates the waste from the surface, stores water that infiltrates the cover system, and protects underlying materials from freezing during winter and from desiccation during dry periods.
- *Filter* - The filter separates the underlying drainage material from the topsoil so that the topsoil will not plug the drainage material. The filter is often a geotextile, but also can be soil.
- *Drainage Layer* - The drainage layer (which is not needed in arid climates) serves to drain away water that infiltrates the topsoil.
- *Geomembrane Liner and Low Hydraulic Conductivity Soil Layer* - The geomembrane and low hydraulic conductivity soil layer form a composite liner that serves as a hydraulic barrier to impede water infiltration through the cover system.

Figure 1-2 illustrates an alternative cover profile recommended by EPA for hazardous waste. In Figure 1-2, *cobbles* are placed on the topsoil to provide protection from erosion. Cobbles, which are normally used only at very arid sites, allow precipitation to infiltrate underlying materials, but do not promote evapotranspiration (since there are no plants present). Figure 1-2 also depicts a *biobarrier* between two filters. The biobarrier is usually a layer of cobbles, approximately 30- to 90-cm (1- to 3-ft) thick. The biobarrier stops animals from burrowing into the ground, and, if the cobbles are dry, prevents the penetration of plant roots. The *gas vent layer* facilitates removal of gases that could accumulate in the waste layer.

The cover profiles shown in Figures 1-1 and 1-2 provide general guidance only. Depending on the specific circumstances at a particular site, some of the layers shown in these figures may not be necessary. For example, at an extremely arid site, a cover system placed over non-hazardous, nonputrescible waste may simply consist of a single layer of topsoil with no drainage layer, no hydraulic barrier, and no gas vent layer. Conversely, some situations may require more layers than those shown in these figures. For example, radioactive waste such as uranium mill tailings may require a radon-emission-barrier layer. In addition, the designer may need to include several components or layers within the cover system to satisfy multiple objectives. When such objectives lead to conflicting technical requirements, tradeoffs are frequently necessary.

FLOW RATES THROUGH LINERS

Figure 2-1 illustrates three types of hydraulic barriers (liners) for cover systems: 1) a low hydraulic conductivity, compacted soil liner; 2) a geomembrane liner; and 3) a geomembrane/soil composite liner. Flow rates for each of these types of liners are calculated below for the purpose of comparing the effectiveness of the barriers.

Flow rates through *compacted soil liners* are calculated using Darcy's law, the basic equation used to describe the flow of fluids through porous materials. Darcy's law states:

$$q = k_s\, i\, A$$

where q is the flow rate (m^3/s); k_s represents the hydraulic conductivity of the soil (m/s); i is the dimensionless hydraulic gradient; and A is the area (m^2) over which flow occurs. If the soil is saturated and there is no soil suction, the hydraulic gradient (i) is:

$$i = (h + D) / D$$

where the terms are defined in Figure 2-1 (h is the depth of liquid ponded above a liner with thickness D). For example, if 30 cm (1 ft) of water is ponded on a 90-cm (3-ft) thick liner that has a hydraulic conductivity of 1×10^{-9} m/s (1×10^{-7} cm/s), the flow rate is 120 gal (454 L)/acre/day. If the hydraulic conductivity is increased or decreased, the flow rate is changed proportionally (Table 2-1).

The second liner depicted in Figure 2-1 is a *geomembrane liner*. It is assumed that the geomembrane has one or more circular holes (defects) in the liner, that the holes are sufficiently widely spaced that leakage through each hole occurs independently from the other holes, that the head of liquid ponded above the liner (h) is constant, and that the soil that underlies the geomembrane has a very large hydraulic conductivity (the subsoil offers no resistance to flow through a hole in the geomembrane). Giroud and Bonaparte (1) recommend the following equation for estimating flow rates through holes in geomembranes under these assumptions:

$$q = C_B\, a\, (2gh)^{0.5}$$

where q is the rate of flow (m^3/s); C_B is a flow coefficient with a value of approximately 0.6; a is the area (m^2) of a circular hole; g is the acceleration due to gravity (9.81 m/s^2); and h is the head (m) above the liner. For example, if there is a single hole with an area of 1 cm^2 (0.0001 m^2) and the head is 30 cm (1 ft) (0.305 m), the calculated rate of flow is 3,300 gal (12,491 L)/day. If there is one hole per acre, then the flow rate is 3,300 gal (12,491 L)/acre/day.

Flow rates for other circumstances are calculated in Table 2-2. Giroud and Bonaparte report that with good quality control, one hole per acre is typical (1). With poor control, 30 holes per acre is typical. They also note that most defects are small (<0.1 cm^2), but that larger holes are occasionally observed. In calculating the rate of flow for "No Holes" in Table 2-2, it was assumed that any flux of liquid was controlled by water vapor transmission; a

Table 2-1. Calculated Flow Rates through Soil Liners with 30 cm of Water Ponded on the Liner

Hydraulic Conductivity (cm/s)	Rate of Flow (gal/acre/day)[a]
1×10^{-6}	1,200
1×10^{-7}	120
1×10^{-8}	12
1×10^{-9}	1

[a] L = gal x 3.785

Table 2-2. Calculated Flow Rates Through a Geomembrane with a Head of 30 cm of Water above the Geomembrane

Size of Hole (cm^2)	Number of Holes Per Acre	Rate of Flow (gal/acre/day)[a]
No holes	--	0.01
0.1	1	330
0.1	30	10,000
1	1	3,300
1	30	100,000
10	1	33,000

[a] L = gal x 3.785

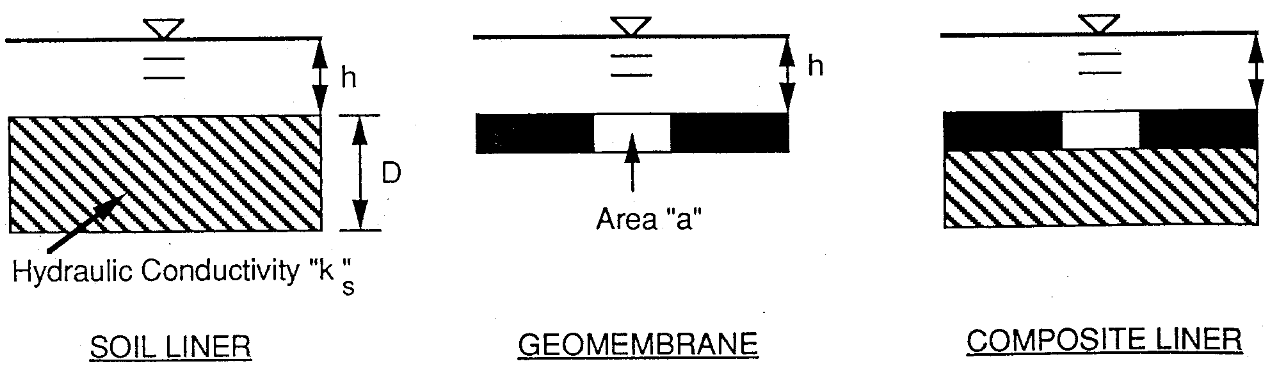

Figure 2-1. Soil liner, geomembrane liner, and composite liner.

flux of 0.01 gal/acre/day corresponds to a typical water vapor transmission rate of geomembrane liner materials.

The third type of liner depicted in Figure 2-1 is a *composite liner.* Giroud and Bonaparte (2) and Giroud et al. (3) discuss seepage rates through composite liners. They recommend the following equation for computing seepage rates for cases in which the hydraulic seal between the geomembrane and soil is poor:

$$q = 1.15\, h^{0.9}\, a^{0.1}\, k_s 0.74$$

where all the parameters and units are as indicated previously. This equation assumes that the hydraulic gradient through the soil is 1. If there is a good hydraulic seal between the geomembrane liner and underlying soil, the flow rate is approximately one-fifth the value computed from the equation shown above; the constant in the equation is 0.21 rather than 1.15 for the case of a good seal. For example, suppose the geomembrane component of a composite liner has one hole/acre with an area of 1 cm^2 per hole, the hydraulic conductivity of the subsoil is 1×10^{-7} cm/s (1×10^{-9} m/s), the head of water is 30 cm (1 ft) and a poor seal exists between the geomembrane and soil. The calculated flow rate is 0.8 gal (3 L)/acre/day. Table 2-3 shows other calculated flow rates for composite liners with a head of water of 30 cm (1 ft.)

It is useful to compare the three types of liners under a variety of assumed conditions, as illustrated in Table 2-4. For discussion purposes, each liner type is classified as poor, good, or excellent. EPA requires that low permeability compacted soil liners used for hazardous wastes have a hydraulic conductivity no greater than 1×10^{-7} cm/s; therefore, a soil liner with a hydraulic conductivity of 1×10^{-7} cm/s is described in Table 2-4 as a "good" liner. A compacted soil liner with a 10-fold higher hydraulic conductivity is described as a "poor" liner, and a soil liner with a 10-fold lower hydraulic conductivity is described as an "excellent" liner.

For geomembrane liners, a liner with a large number of small holes (30 holes/acre, with each hole having an area of 0.1 cm^2) is described as a "poor" liner because Giroud and Bonaparte suggest that such a large number of defects would be expected only with minimal construction quality control (1). A "good" geomembrane liner was assumed to have been constructed with good quality assurance and an "excellent" geomembrane liner was assumed to have one small hole/acre (1). For all of the seepage rates computed for composite liners in Table 2-4, it was assumed that there was poor contact between the geomembrane and soil.

As Table 2-4 illustrates, a composite liner (even one built by poor to mediocre standards) significantly outperforms a soil liner or a geomembrane liner alone. For this reason, a composite liner is recommended when there is enough rainfall to warrant a very low-permeability hydraulic barrier in the cover system.

Table 2-3. Calculated Flow Rates for Composite Liners with a Head of Water of 30 cm

Hydraulic Conductivity of Subsoil (cm/s)	Size of Hole in Geomembrane (cm^2)	Number of Holes/Acre	Rate of Flow (gal/acre/day)[a]
1×10^{-6}	0.1	1	3
1×10^{-6}	0.1	30	102
1×10^{-6}	1	1	4
1×10^{-6}	1	30	130
1×10^{-6}	10	1	5
1×10^{-7}	0.1	1	0.6
1×10^{-7}	0.1	30	19
1×10^{-7}	1	1	0.8
1×10^{-7}	1	30	24
1×10^{-7}	10	1	1.0
1×10^{-8}	0.1	1	0.1
1×10^{-8}	0.1	30	3
1×10^{-8}	1	1	0.1
1×10^{-8}	1	30	4
1×10^{-8}	10	1	0.2
1×10^{-9}	0.1	1	0.2
1×10^{-9}	0.1	30	0.6
1×10^{-9}	1	1	0.03
1×10^{-9}	1	30	0.8
1×10^{-9}	10	1	0.03

[a]L = gal x 3.785

To maximize the effectiveness of a composite liner, the geomembrane must be placed to achieve a good hydraulic seal with the underlying layer of low hydraulic conductivity soil. As shown in Figure 2-2, the composite liner works by limiting the flow of fluid in the soil to a very small area. Fluid must not be allowed to spread laterally along the interface between the geomembrane and soil. To ensure good hydraulic contact, the soil liner should be smooth-rolled with a steel-drummed roller before the geomembrane is placed, and the geomembrane should have a minimum number of wrinkles when it is finally covered. In addition, high-permeability material, such as a sand bedding layer or geotextile, should not be placed between the geomembrane and low hydraulic conductivity soil (Figure 2-2) because this will destroy the composite action of the two materials.

If there are concerns that rocks or stones in the soil material may punch holes in the geomembrane, the stones should be removed, or a stone-free material with a low hydraulic conductivity placed on the surface. Vibratory screens also can be used to sieve stones prior to placement. Alternatively, mechanical devices that sieve stones or move them to a row in a loose lift of soil may be used. A different material, or a differently

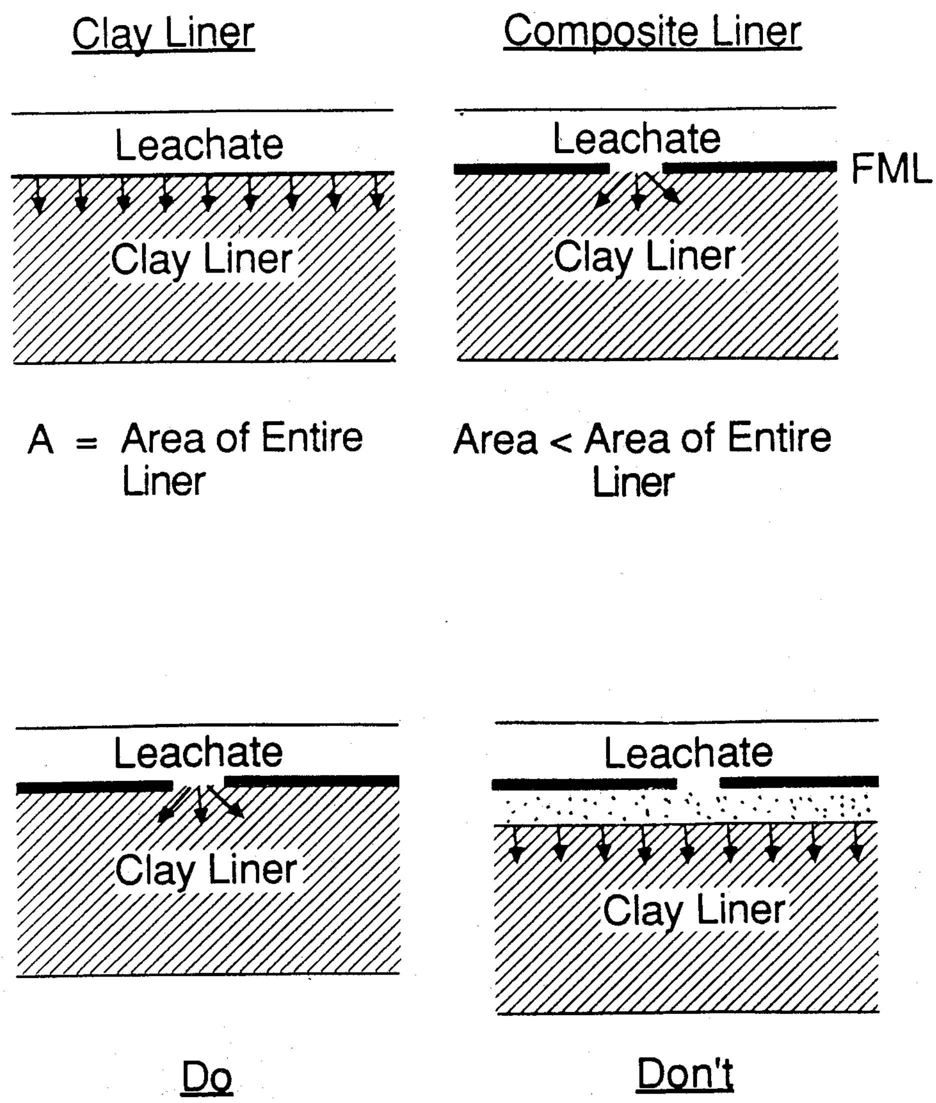

Figure 2-2. Soil liner and composite liner.

processed material that has fewer and smaller stones, may be used to construct the uppermost lift of the soil liner (i.e., the lift that will serve as a foundation for the geomembrane).

CRITICAL PARAMETERS FOR SOIL LINERS

Materials

The primary requirement for a soil liner material is that it be capable of being compacted to produce a suitably low hydraulic conductivity. To meet this requirement, the following conditions should be met:

- *Fines* - The soil should contain at least 20 percent fines (fines are defined as the percentage, on a dry-weight basis, of material passing the No. 200 sieve, which has openings of 0.075 mm).
- *Plasticity Index* - The soil should have a plasticity index of at least 10 percent, although some soils with a slightly lower plasticity index may be suitable. Soils with plasticity indices less than about 10 percent have very little clay and usually will not produce the necessary low hydraulic conductivity. Soils with plasticity indices greater than 30 to 40 percent are difficult to work with, as they form hard chunks when dry and sticky clods when wet, which make them difficult to work with in the field. Such soils also tend to have high shrink/swell potential and may not be suitable for this

Table 2-4. Calculated Flow Rates for Soil Liners, Geomembrane Liners, and Composite Liners

Type of Liner	Overall Quality of Liner	Assumed Values of Key Parameters	Rate of Flow (gal/ acre/day)[a]
Compacted Soil	Poor	k_s=1 x 10^{-6} cm/s	1,200
Geomembrane	Poor	30 holes/acre; a=0.1 cm^2	10,000
Composite	Poor	k_s=1 x 10^{-6} cm/s 30 holes/acre; a=0.1 cm^2	100
Compacted Soil	Good	k_s=1 x 10^{-7} cm/s	120
Geomembrane	Good	1 hole/acre; a=1 cm^2	3,300
Composite	Good	k_s=1 x 10^{-7} cm/s 1 hole/acre; a=1 cm^2	0.8
Compacted Soil	Excellent	k_s=1 x 10^{-8} cm/s	12
Geomembrane	Excellent	1 hole/acre; a=0.1 cm^2	330
Composite	Excellent	k_s=1 x 10^{-8} cm/s 1 hole/acre; a=0.1 cm^2	0.1

[a]L = gal x 3.785

reason. Soils with plasticity indices between approximately 10 and 35 percent are generally ideal.

- *Percentage of Gravel* - The percentage of gravel (defined as material retained on the No. 4 sieve, which has openings of 4.76 mm) must not be excessive. A maximum amount of 10 percent gravel is suggested as a conservative figure. For many soils, however, larger amounts may not necessarily be deleterious if the gravel is uniformly distributed in the soil and does not interfere with compaction by footed rollers. For example, Shakoor and Cook found that the hydraulic conductivity of a compacted, clayey soil was insensitive to the amount of gravel present, as long as the gravel content did not exceed 50 percent (4). Gravel is only deleterious if the pores between gravel particles are not filled with clayey soil and the gravel forms a continuous pathway through the liner. The key problem to be avoided is segregation of gravel in pockets that contain little or no fine-grained soil.

- *Stones and Rocks* - No stones or rocks larger than 2.5 to 5 cm (1 to 2 in.) in diameter should be present in the liner material.

If the soil material does not contain enough clay or other fine-grained minerals to be capable of being compacted to the desired low hydraulic conductivity, commercially produced clay minerals, such as sodium bentonite, may be mixed with the soil. Figure 2-3 shows the relationship between the percentage of bentonite added to a soil and the hydraulic conductivity after compaction for a well-graded, silty soil that was carefully mixed in the laboratory. The percentage of bentonite is defined as the dry weight of bentonite divided by the dry weight of soil to which the bentonite is added (W_b/W_s). For well-graded soils containing a wide range of grain sizes, adding just a small amount of bentonite will usually lower the hydraulic conductivity of the soil to below 1 x 10^{-7}. For poorly graded soils, e.g., those with a uniform grain size, more bentonite is often needed.

Bentonite can be added to soil in two ways. One technique is to spread the soil to be amended over an area in a loose lift approximately 23 to 30 cm (9- to 12-in.) thick. Bentonite is then applied to the surface at a controlled rate and mixed into the soil using mechanical mixing equipment, such as a rototiller or road reclaimer (recycler). Multiple passes of the mixing equipment are usually recommended. The second procedure is to mix the ingredients in a pugmill, which is a large device used to mix bulk materials such as the ingredients that form Portland cement concrete. Bulk mixing in a pugmill usually provides more controlled mixing than combining ingredients in place in a loose lift of soil. However, mixing of bentonite into a loose lift of soil can be adequate if the mixing is done carefully with multiple passes of mechanical mixers and careful control over rates of application and depth of mixing. The reason why bulk mixing is usually recommended is that control over the mixing process is easier.

Water Content

The water content of the soil at the time it is compacted is an important variable controlling the engineering properties of soil liner materials. The lower half of Figure 2-4 shows a soil compaction curve. If soil samples are mixed at several water contents and then compacted with a consistent method and energy of compaction, the result is the relationship between dry unit weight and molding water content shown in the lower half of Figure 2-4. The molding water content at which the maximum dry unit weight is observed is termed the "optimum water content" and is indicated in Figure 2-4 with a dashed vertical line. Soils compacted at water contents less than optimum ("dry of optimum") tend to have a relatively high hydraulic conductivity whereas soils compacted at water contents greater than optimum ("wet of optimum") tend to have a low hydraulic conductivity. It is usually preferable to com-

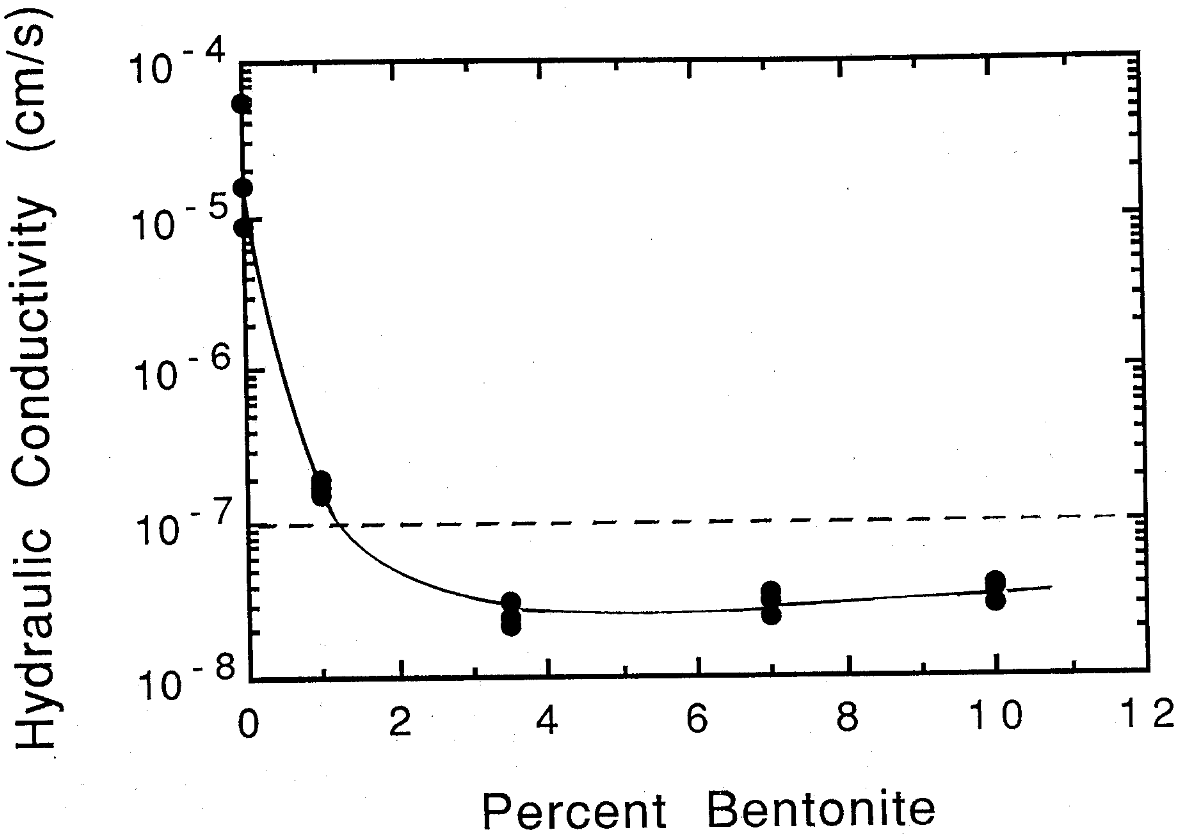

Figure 2-3. Effect of bentonite upon the hydraulic conductivity of a bentonite-amended soil. (Percent bentonite = $\frac{W_b}{W_s}$)

pact the soil wet of optimum to achieve minimal hydraulic conductivity.

Figures 2-5 to 2-7 illustrate for a highly plastic soil why wet-of-optimum compaction is so effective in achieving low hydraulic conductivity. These three photographs show a soil that was compacted with standard Proctor energy (ASTM D698). The soil had a plasticity index of 41 percent. The optimum water content for this soil and compaction procedure was 19 percent. The specimen shown in Figure 2-5 was compacted at a water content of 12 percent (7 percent dry of optimum). This compacted soil had a very high hydraulic conductivity (1×10^{-4} cm/s) because the dry, hard clods of soil were not broken down and remolded by the energy of compaction. The specimen shown in Figure 2-6 was compacted at a water content of 16 percent (3 percent dry of optimum) and had a hydraulic conductivity of 1×10^{-3} cm/s; the clods were still too dry and hard at this water content to permit the clods to be remolded into a homogeneous mass with low hydraulic conductivity. The specimen shown in Figure 2-7 was compacted at a water content of 20 percent (1 percent wet of optimum) and had a hydraulic conductivity of 1×10^{-9} cm/s. At this water content, the clods were wet, soft, and easily remolded into a homogeneous mass that was free of remnant clods and large inter-clod voids and pore spaces. The visual differences between specimens compacted dry versus wet of optimum are usually not as obvious as they are in Figures 2-5 to 2-7 for soils of lower plasticity. However, even for low-plasticity clays, experience has almost always shown that the soil must be compacted wet of optimum water content to achieve minimum hydraulic conductivity.

The water content of the soil must be adjusted to the proper value prior to compaction and the water should be uniformly distributed in the soil. If the soil requires additional water, it can be added with a water truck; care should be taken to apply the water to the soil in a controlled, uniform manner, e.g., with a spray bar mounted on the rear of the trucks. Rototillers (Figure 2-8) are very effective for mixing wetted soil; these devices distribute the water uniformly among clods of material. Figure 2-9 depicts the teeth on the blades of a rototiller, which provide the mixing action. Mechanical mixing to mix water evenly into the soil is especially important for highly plastic soils that form large clods of soil.

Compactive Energy

Another important variable controlling the engineering properties of soil liner materials is the energy of compaction. As shown in Figure 2-10, increasing the energy of compaction increases the dry unit weight of the soil, decreases the optimum water content, and reduces

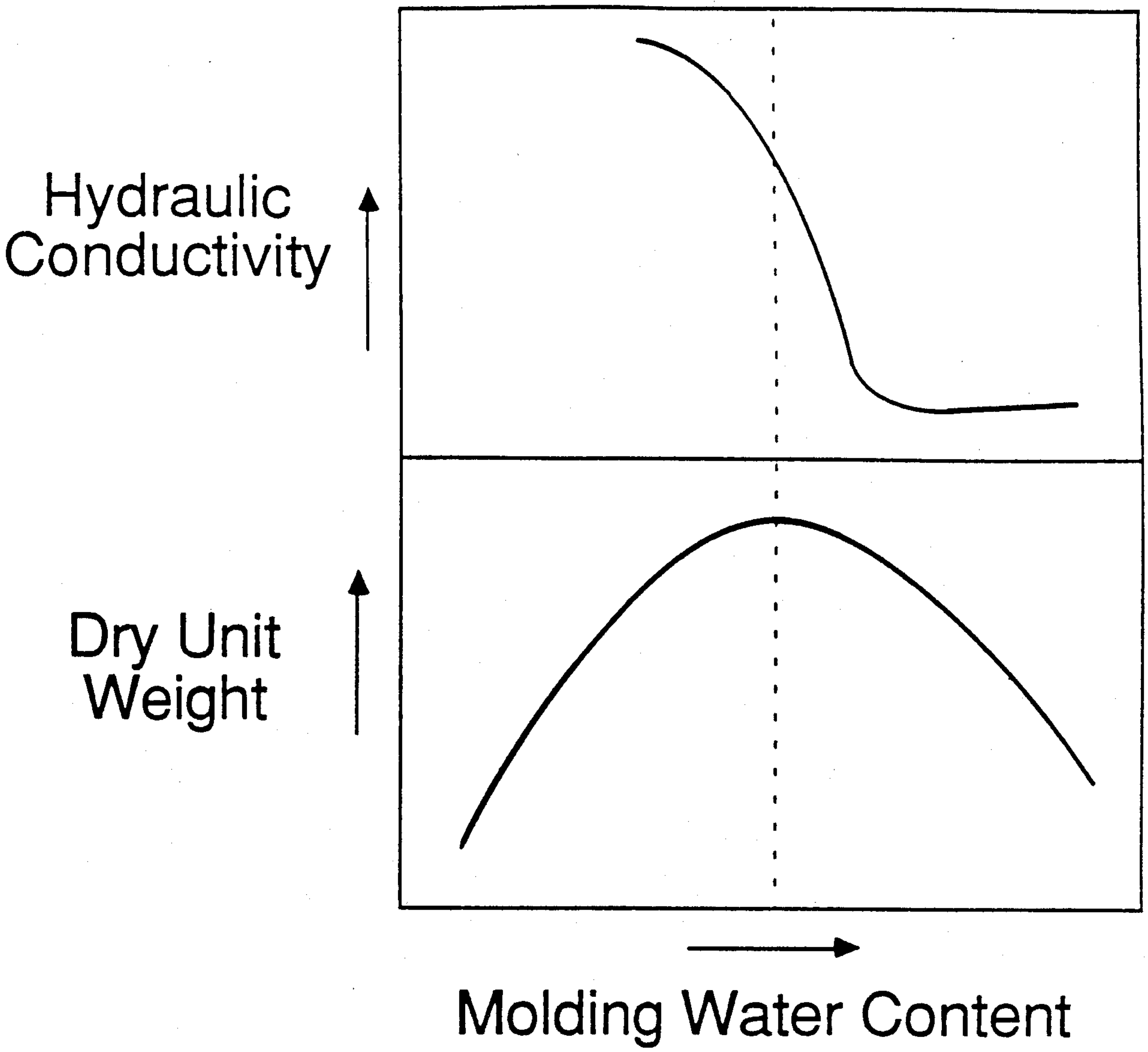

Figure 2-4. Hydraulic conductivity and dry unit weight versus molding water content.

hydraulic conductivity. The hydraulic conductivity of a soil that is compacted wet of optimum could be lowered by one to two orders of magnitude by increasing the energy of compaction, even though the dry unit weight of the soil is not increased measurably. More energy of compaction helps to remold clods of soil, realign soil particles, reduce the size or degree of connection of the largest pores in the soil, and lower hydraulic conductivity.

The compactive energy delivered to soil depends on the weight of the roller, the number of passes of the roller over a given area, and the thickness of the soil lift being compacted. Increasing the weight and number of passes, and decreasing the lift thickness, can increase the compactive effort. The best combination of these factors to use when compacting low hydraulic conductivity soil liners depends on the water content of the soil and the firmness of the subbase.

Heavy rollers cannot be used if the soil is very wet or if the foundation is weak and compressible (e.g., if municipal solid waste is located just 30- to 60-cm [1- to 2-ft] below the layer to be compacted). Rollers with static weights of at least 13,608 to 18,144 kg (30,000 to 40,000 pounds) are recommended for compacting low hydraulic conductivity layers in cover systems. Rollers that weigh up to 31,752 kg (70,000 pounds) are available and may be desirable for compacting bottom liners of landfills, but such rollers are too heavy for many cover systems because of the presence of compressible waste material a short distance below the cover.

The roller must make a sufficient number of passes over a given area to ensure adequate compaction. The minimum number of passes will vary, but at least 5 to 10 passes are usually required to deliver sufficient compactive energy and to provide adequate coverage.

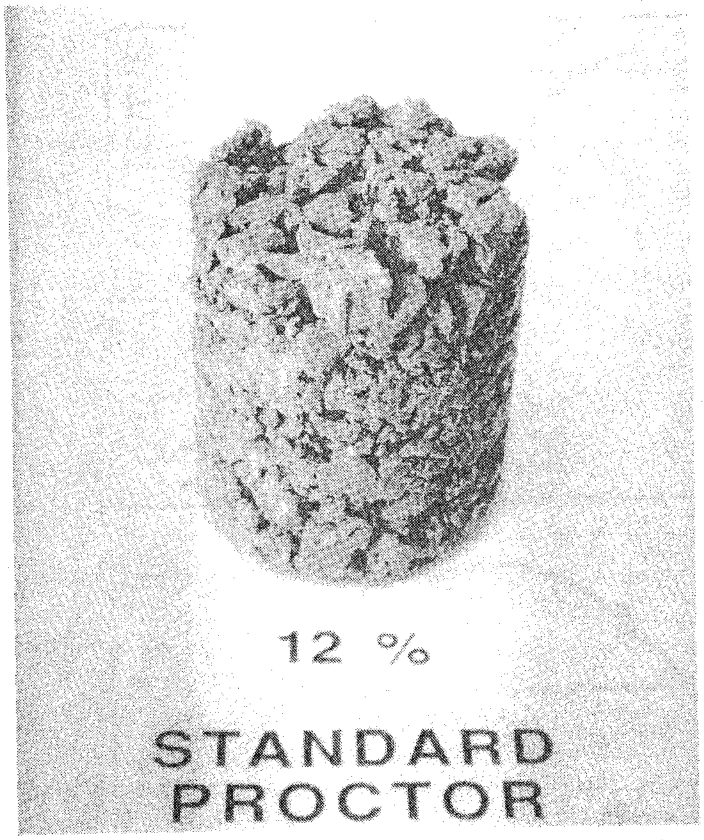

Figure 2-5. Highly plastic soil compacted with standard Proctor procedures at a water content of 12%.

Figure 2-€. Highly plastic soil compacted with standard Proctor procedures at a water content of 16%.

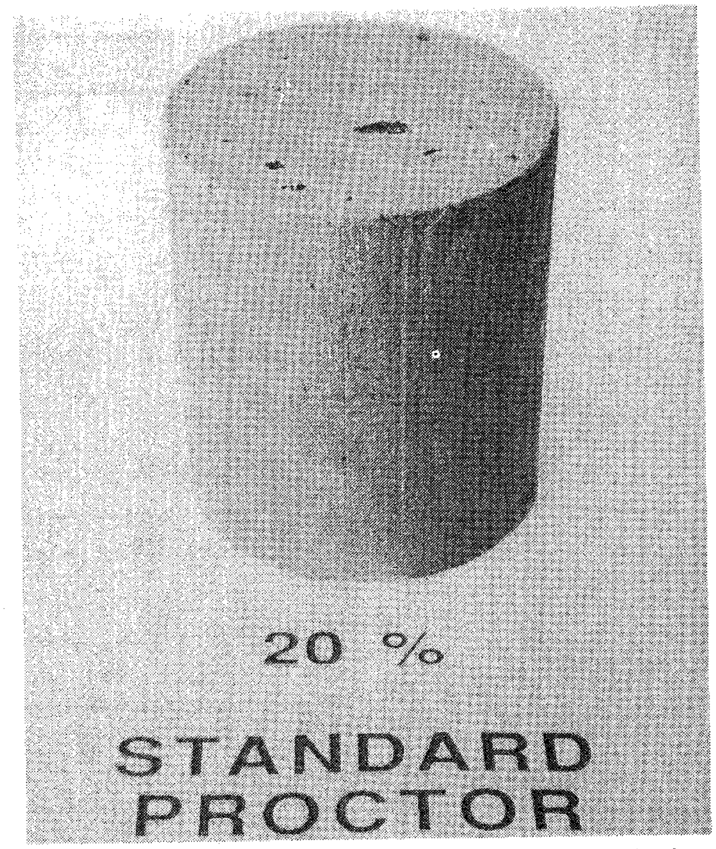

Figure 2-7. Highly plastic soil compacted with standard Proctor procedures at a water content of 20%.

Size of Clods

The clay-rich soils that are usually used to construct soil liners typically form dry, hard clods of soil or wet, sticky clods, depending on water content. Highly plastic soils almost always form large clods. Soils with low plasticity (plasticity index less than about 10%) do not form very large clods. For soils that form clods, the clods must be remolded into a homogeneous mass that is free of large inter-clod pores if low hydraulic conductivity is to be achieved.

Benson and Daniel described the influence of clod size of a highly plastic soil (plasticity index = 41%) upon hydraulic conductivity (5). These investigators processed a clayey soil by breaking clods down to pass either the No. 4 sieve (4.76 mm or 0.2 in. openings) or the 1.9-cm (3/4-in.) sieve. The soil was then wetted, allowed to hydrate at least 24 hours, compacted, and permeated.

Benson and Daniel's (1990) results are summarized in Table 2-5. The optimum water content was 17 percent for the clods processed through the sieve with a 0.5-cm (0.2-in.) opening and 19 percent for the soil processed through the sieve with a 1.9-cm (3/4-in.) opening. For soil compacted dry of optimum, the soil with smaller clods had a hydraulic conductivity that was several orders of magnitude lower than the soil with larger clods. When the soils were compacted wet of optimum, the size of clods had a negligible effect. Size is therefore important for dry,

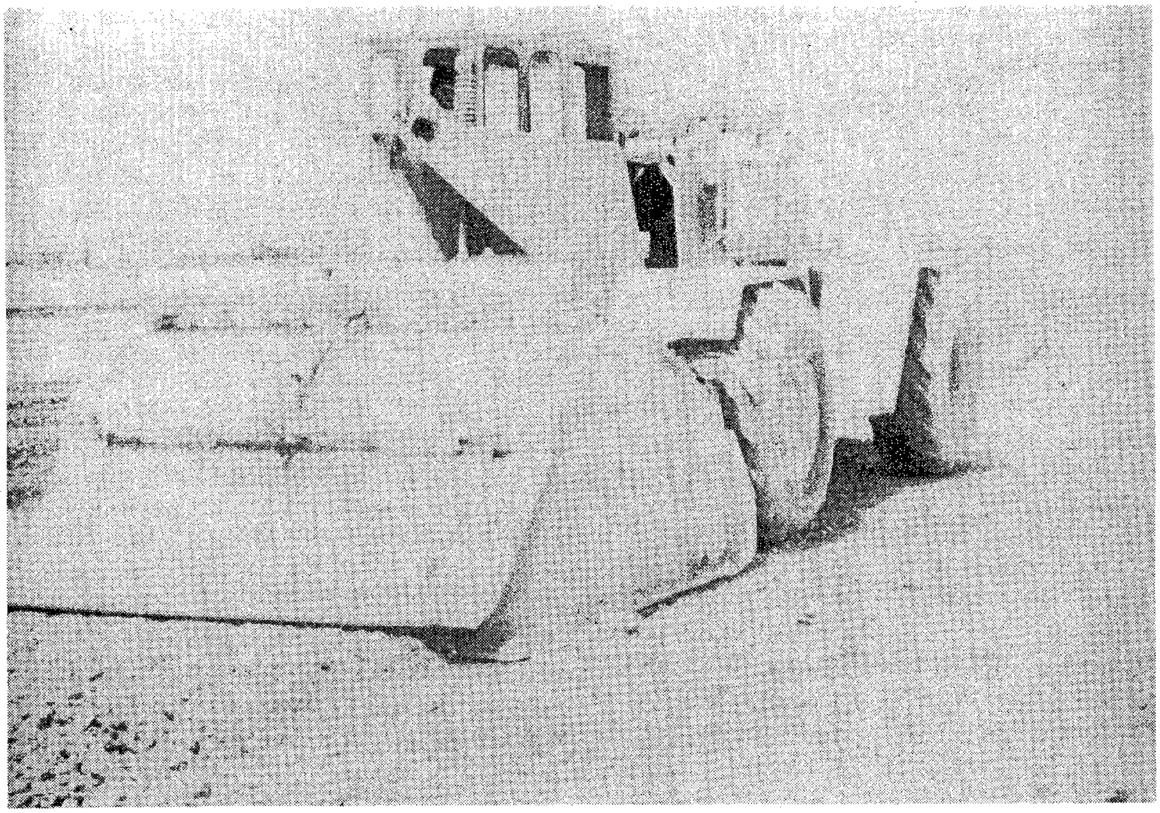

Figure 2-8. Rototiller used to mix soil.

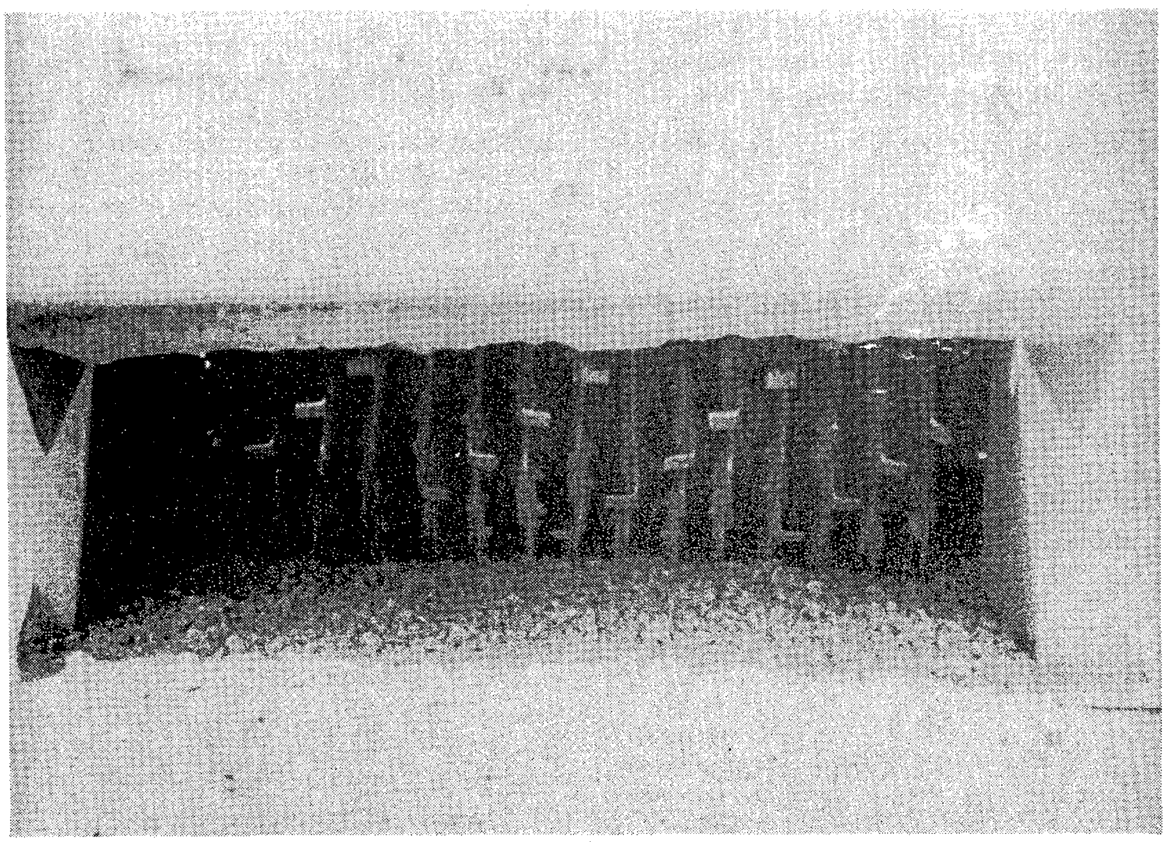

Figure 2-9. Blades and teeth on rototiller.

Table 2-5. Effect of Size of Clods during Processing of Soil upon Hydraulic Conductivity of Soil after Compaction

Molding Water Content (%)	Hydraulic Conductivity (cm/s) 0.2-in. Clods[a]	0.75-in. Clods[a]
12	2×10^{-8}	4×10^{-4}
16	2×10^{-9}	1×10^{-3}
18	1×10^{-8}	8×10^{-10}
20	2×10^{-9}	7×10^{-10}

[a]cm = in. x 2.540

hard clods (dry of optimum), but not for wet, soft clods (wet of optimum). When the soil is compacted wet of optimum, the clods are sufficiently soft that they are easily remolded regardless of their original size.

One way to reduce the size of clods in dry materials is to use a road reclaimer (also called a road recycler), such as the one shown in Figure 2-11. This device pulverizes materials with teeth that rotate on a drum at a high speed. The device was used with great effectiveness at a site in Pennsylvania in which a mudstone was used for a liner material (Figure 2-12). In the figure, the road reclaimer has made a pass through a loose lift of material. After just one pass of the road reclaimer, the size of mudstone clods has been greatly reduced.

Bonding of Lifts

Bonding of lifts is important in achieving a low hydraulic conductivity in soil liners. The upper half of Figure 2-13 illustrates a cross-section of a soil liner consisting of four lifts. A borehole has been drilled into the lowest lift, filled with a dye-stained fluid, left for a period of time, and then drained. The dye penetrates the soil further along lift interfaces than through the lifts themselves. Due to imperfect bonding of lifts, a zone of higher horizontal hydraulic conductivity exists at lift interfaces in this example.

Lift interfaces have important ramifications with respect to the overall hydraulic performance of a soil liner. The lower half of Figure 2-13 depicts a liner consisting of six lifts. Each lift has a few "hydraulic defects." If the lift interfaces have high hydraulic conductivity, water can flow downward through the more permeable zones in a lift and spread laterally along a lift interface until it encounters a permeable zone in the underlying lift. This process repeats for underlying lifts and lift interfaces. In this way lift interfaces provide hydraulic connection between defects in overlying and underlying lifts. Better overall performance (lower hydraulic conductivity) is achieved if lifts are bonded together to eliminate high conductivity at lift interfaces.

To bond lifts together, the surface of the previously compacted lift should be rough so that the newly placed lift can effectively blend into the surface. If necessary, the surface of the previously compacted lift can be roughened by discing the soil to a depth of approximately 2.5 cm (1 in). Discing the soil involves plowing up the soil surface to a shallow depth so that the surface is rough and so that there will be no abrupt interface between lifts.

Compactors with long "feet" on the drums are useful in blending one lift into another. Figure 2-14 shows a popular heavy compactor (20,000 kg [44,000 pounds]) with feet that are 18 to 23 cm (7 to 9 in.) long. During the first few passes of the compactor, the feet sink through a loose lift of soil and compact the newly placed lift into the surface of the previously compacted lift. Using a roller with feet that fully penetrate a loose lift of soil is recommended to bond lifts and to minimize high horizontal hydraulic conductivity at lift interfaces.

If a geomembrane liner will be placed on the compacted soil liner, the final surface of the soil liner should be compacted with a smooth, steel drum roller to achieve a good hydraulic seal.

EFFECTS OF DESICCATION

Desiccation of soil liners occurs whenever the soil liner dries, which can be during or after construction. Desiccation causes soil liner materials to shrink and, potentially, to crack. Cracking can be disastrous in terms of hydraulic conductivity because cracked liners are more permeable than uncracked liners.

Boynton and Daniel desiccated slabs of compacted clay, trimmed cylindrical test specimens for hydraulic conductivity testing from the desiccated slabs, and measured the hydraulic conductivity at different effective confining stresses (6). In laboratory tests, the confining stress simulates the weight of overburden soil; the greater the confining stress, the greater the depth of burial below the surface that is simulated. Control tests also were performed on soils that had not been desiccated. These results are summarized in Figure 2-15. At low confining stress, the desiccated soils were much more permeable than the control. At high confining stress, however, the desiccated soils were no more permeable than the control. It appeared that the application of a large compressive stress (>5 psi, or 35 kPa) closed the desiccation cracks that had formed and, in combination with hydration of the soil, essentially fully healed the damage done by desiccation.

In cover systems, the overburden stress on the liner components is controlled by the depth of soil overlying the liner. Because the thickness of soil overburden above the liner seldom exceeds a few feet, the overburden stress is normally low. Soil applies an overburden stress of approximately 1 psi per foot of depth. Thus, for example, if

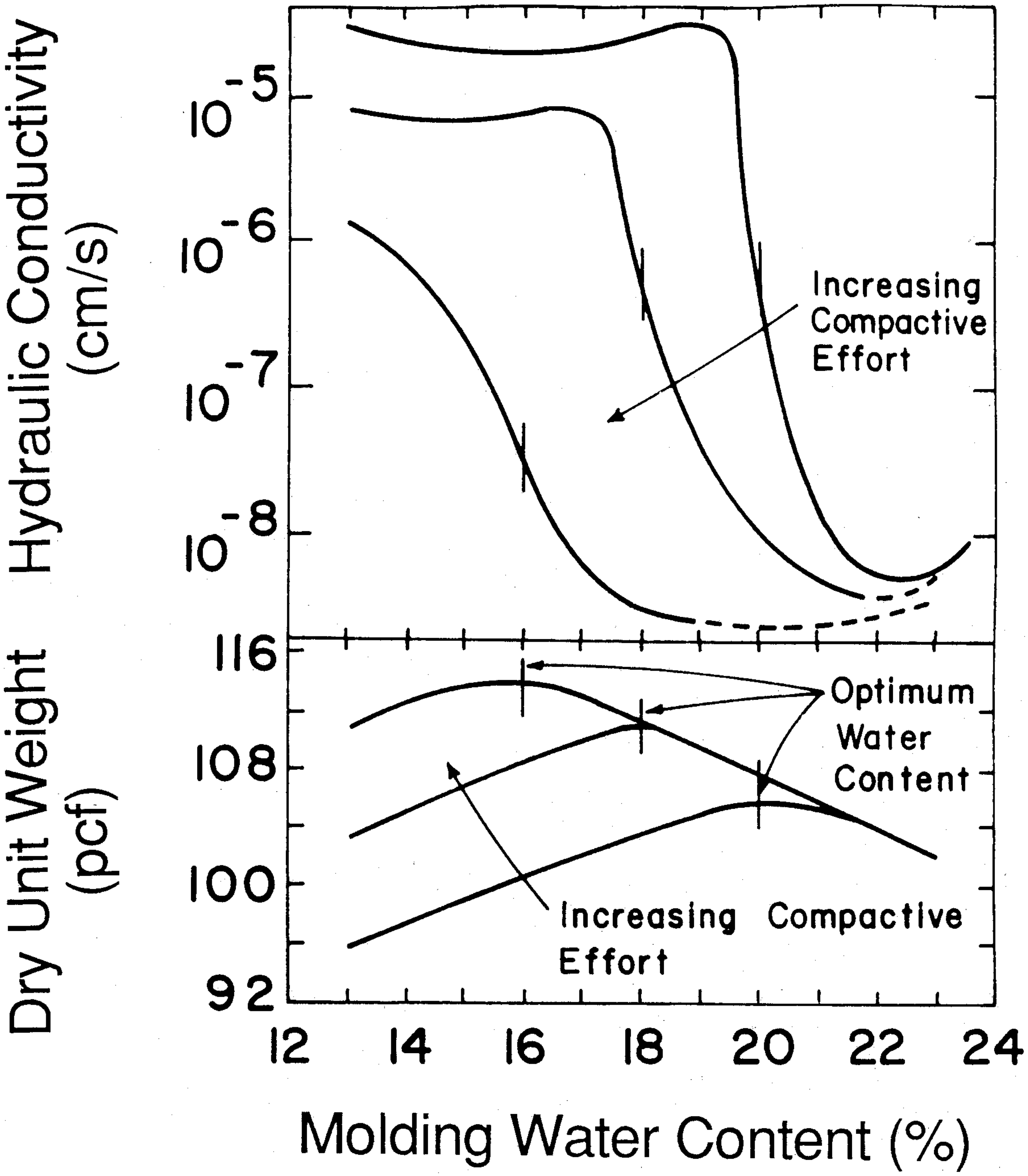

Figure 2-10. Influence of compactive effort upon hydraulic conductivity and dry unit weight.

60 cm (2 ft) of topsoil overlie a 60-cm (2-ft) thick layer of compacted clay, the maximum overburden stress at the bottom of the clay is approximately 4 psi. Based on Boynton and Daniel's results, if desiccation of the compacted soil liner occurs in a cover system, even though wetting of the soil may partly swell the soil and "heal" desiccation cracks, it is not expected that all the damage done by desiccation would be self-healing.

Montgomery and Parsons described an example of the damaging effects of desiccation (7). Test plots were built at the Omega Hills Landfill near Milwaukee, Wisconsin, in 1985. In both test plots, the cover systems consisted of 122 cm (4 ft) of compacted clay. The clay was overlain by 15 cm (6 in.) of topsoil in one plot and 46 cm (18 in.) of topsoil in the other. In both test plots, the upper 20 to 25 cm (8 to 10 in.) of compacted clay had weathered and become blocky after 3 years. Cracks up to 1.3-cm (1/2-in.) wide extended 89 to 102 cm (35 to 40 in.) into the compacted clay liner. The 46 cm (18 in.) of topsoil did not appear to be any more effective than 15 cm (6 in.) in protecting the underlying clay from desiccation.

The layer of low hydraulic conductivity, compacted soil placed in a cover system must be protected from the damaging effects of desiccation both during and after

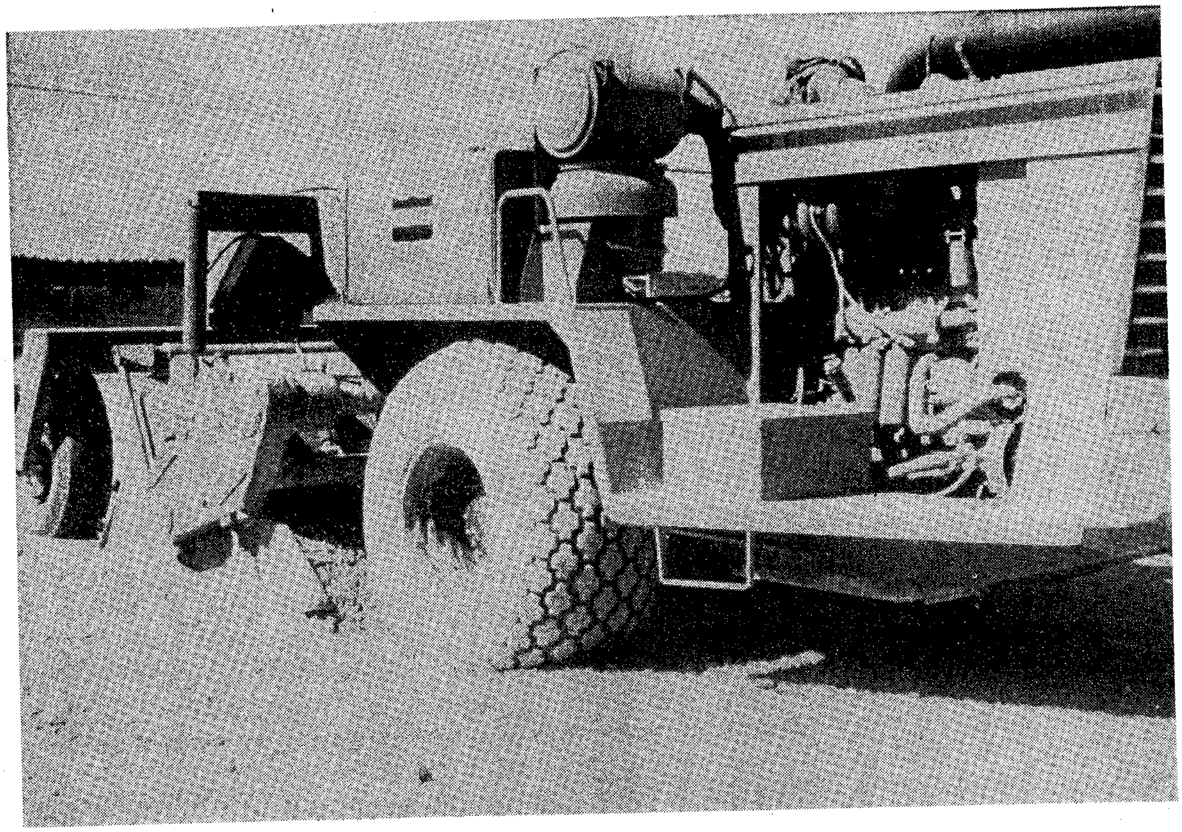

Figure 2-11. Road recycler used to pulverize clods of soil.

construction. During construction, the soil must not be allowed to dry significantly either during or after compaction of each lift. Frequent watering of the soil is usually the best way to prevent desiccation during construction. The higher the water content of the soil and the higher the plasticity of the soil, the greater is the shrinkage potential from desiccation. There are two ways to provide the required protection after construction. One way is to bury the liner beneath an adequate depth of soil overburden; another technique is to place a geomembrane over the soil. If a geomembrane liner is placed on a soil liner to form a composite, it is often convenient to overbuild the soil liner (i.e., make it thicker than necessary) and then to scrape away a few inches of potentially desiccated surficial soil just before the geomembrane is placed.

EFFECTS OF FROST ACTION

Zimmie and La Plante studied the effects of freezing and thawing upon the hydraulic conductivity of a compacted clay by testing soils compacted dry of optimum, at optimum, and wet of optimum (8). They found that freeze/thaw cycles caused an increase in hydraulic conductivity of one to two orders of magnitude in all soils examined. Most of the damage was done after only one to two cycles of freezing and thawing. From this and other work, it is recommended that the low hydraulic conductivity component of cover systems not be allowed to freeze. Freezing can be avoided by burying the low hydraulic conductivity soil layer under an adequately thick layer of soil.

EFFECTS OF SETTLEMENT

Two types of settlement are of concern with respect to covers: total settlement and differential settlement. Total settlement of the surface of a cover is the total downward movement of a fixed point on the surface. Differential settlement is always measured between two points and is defined as the difference between the total settlements at these two points. *Distortion* is defined as the differential settlement between two points divided by the distance along the ground surface between the two points. Excessive differential settlement of underlying waste can damage a cover system. If differential settlement occurs, tensile strains develop in cover materials as a result of bending stresses and/or elongation. Tensile strain is defined as the amount of stretching of an element divided by the original length of the element. Anytime the cover settles differentially, some part of the cover will be subjected to tension and will undergo tensile strain. Tensile strains are of concern because the larger the stretching (tensile strain), the greater the possibility that the soil will crack and that a geomembrane will rupture. *Bending stresses*, stresses that occur when an object is bent, result when covers settle differentially; part of the bent cover is in tension and part is in compression. Bending stresses are of concern because the tensile stresses associated with bending may be large enough to cause the

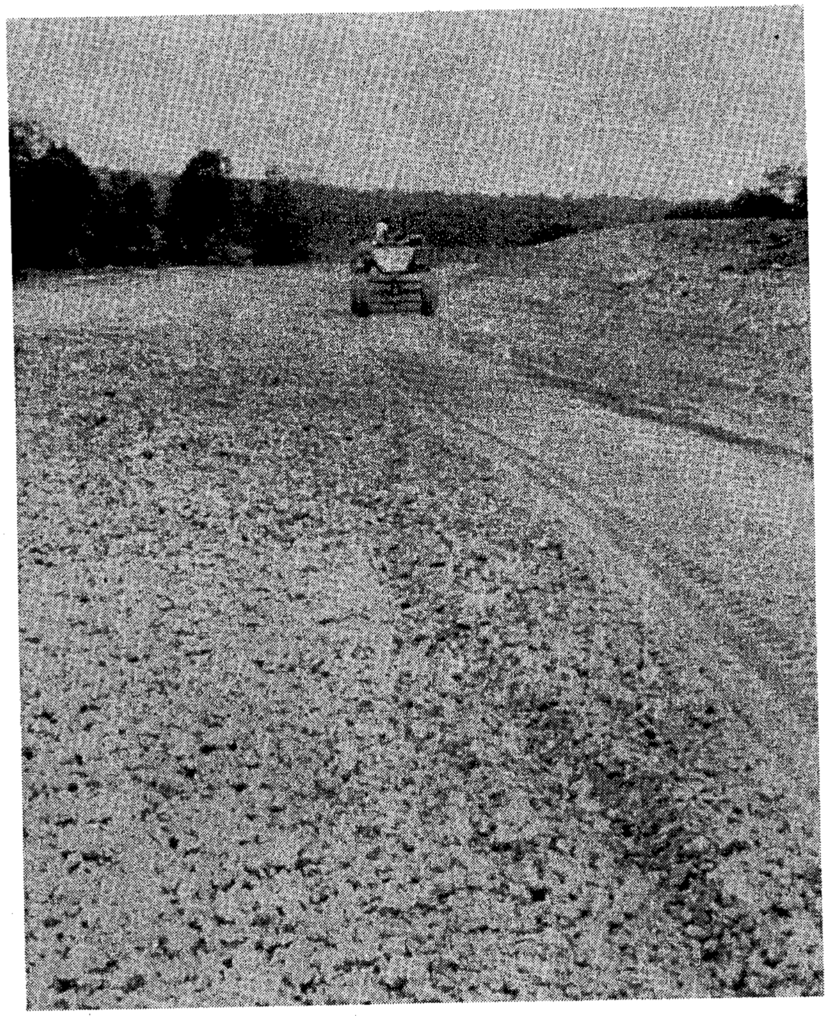

Figure 2-12. Passage of road recycler over loose lift of mudstone to reduce size of chunks of mudstone.

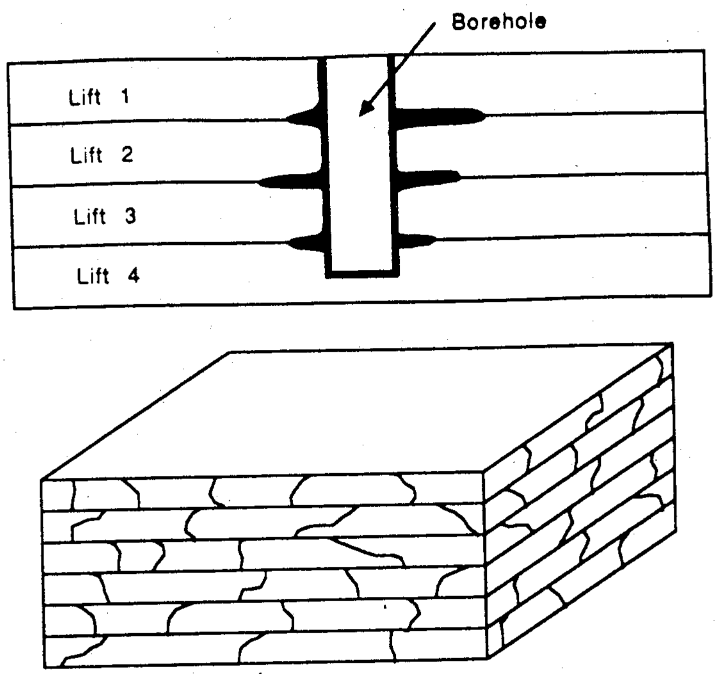

Figure 2-13. Effect of imperfect bonding of lifts on hydraulic performance of soil liner.

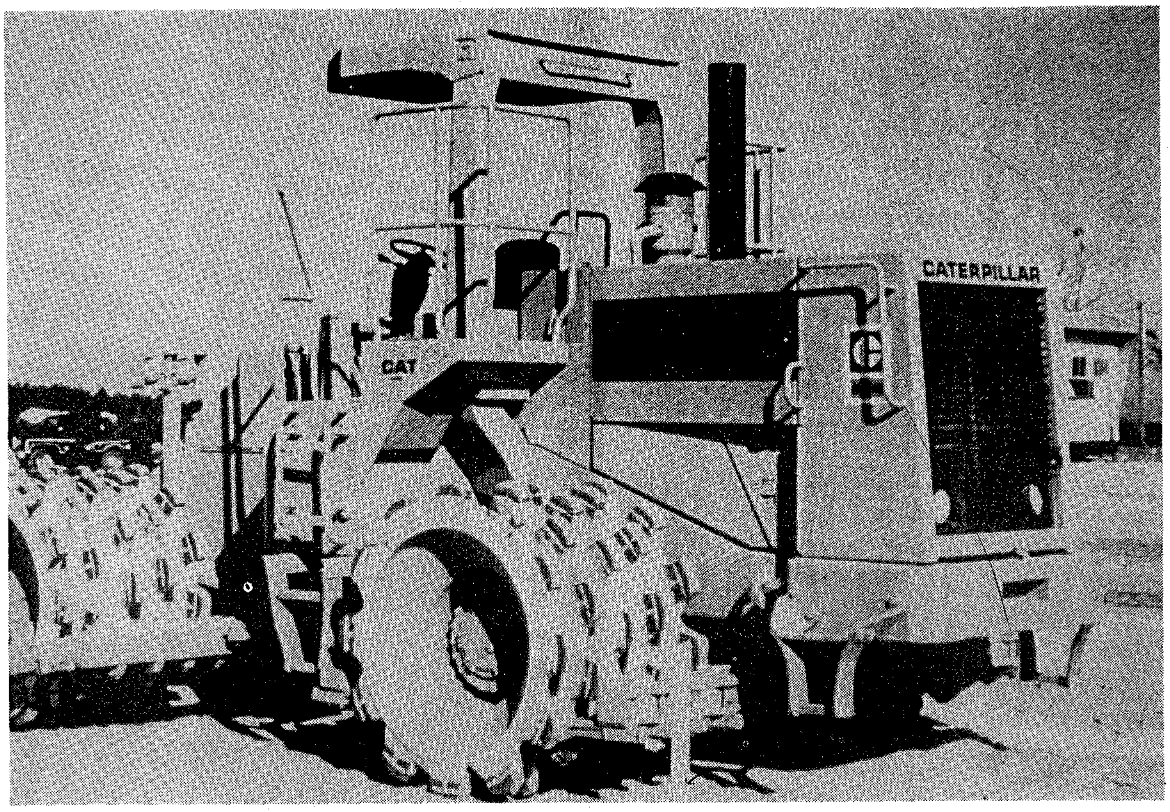

Figure 2-14. Example of heavy footed roller with long feet.

soil to crack. Geomembranes can generally withstand far larger tensile strains without failing than soils. The geomembrane also has the ability to elongate (stretch) a great deal without rupturing or breaking.

Gilbert and Murphy discuss the prediction and mitigation of subsidence damage to covers (9). Gilbert and Murphy developed a relationship between tensile strain in a cover and distortion, delta/L, where delta is the amount of differential settlement that occurs between two points that are a distance (L) apart. This relationship is shown in Figure 2-16. As the distortion increases, the tensile strain in the cover soils increases.

Minor cracking of topsoil or drainage layers as a result of tensile stresses is of little concern. However, cracking of a hydraulic barrier, such as a layer of low hydraulic conductivity soil, is of great concern because the hydraulic integrity of the barrier layer is compromised if it is cracked. The amount of strain that a low hydraulic conductivity, compacted soil can withstand prior to cracking depends significantly upon the water content of the soil. As shown in Figure 2-17, soils compacted wet of optimum are more ductile than soils compacted dry of optimum. For cover systems, ductile soils that can withstand significant strain without cracking are preferred. For this reason, as well as the hydraulic conductivity considerations discussed earlier, it is preferable to compact low hydraulic conductivity soil layers wet of optimum. The soil must then be kept from drying out and cracking, as discussed earlier.

Gilbert and Murphy summarize information concerning tensile strain at failure for compacted, clayey soils (9). The available data show that such soils can withstand maximum tensile strains of 0.1 to 1 percent. If the lower limit (0.1 percent) is used for design, the maximum allowable value of distortion (delta/L) is approximately 0.05 (Figure 2-17). 2-16

To put this in perspective, suppose that a circular depression develops in a cover system. The depression has a radius of 3 m (10 ft) (diameter=6 m [20 ft]). The maximum allowable delta/L is 0.05, and L is the radius of the depression, which is 3 m (10 ft). The maximum allowable settlement (delta) is 0.05 times 3 m (10 ft), or 15 cm (6 in.). If the settlement at the center of the depression exceeds 6 in., the clay layer may crack from the tensile strains caused by the settlement.

Some wastes (such as loose municipal solid waste or unconsolidated sludge of varying thickness) are so compressible that constructing a cover system above the waste will almost certainly produce distortions that are far larger than 0.05. The hydraulic integrity of a low hydraulic conductivity layer of compacted soil is likely to be seriously damaged by the distortion caused by large differential settlement. If the waste is continuing to settle, e.g., as a result of decomposition, it may be prudent to place a temporary cover on the waste and wait for settlement to take place prior to constructing the final cover system. Alternatives for stabilizing the waste include

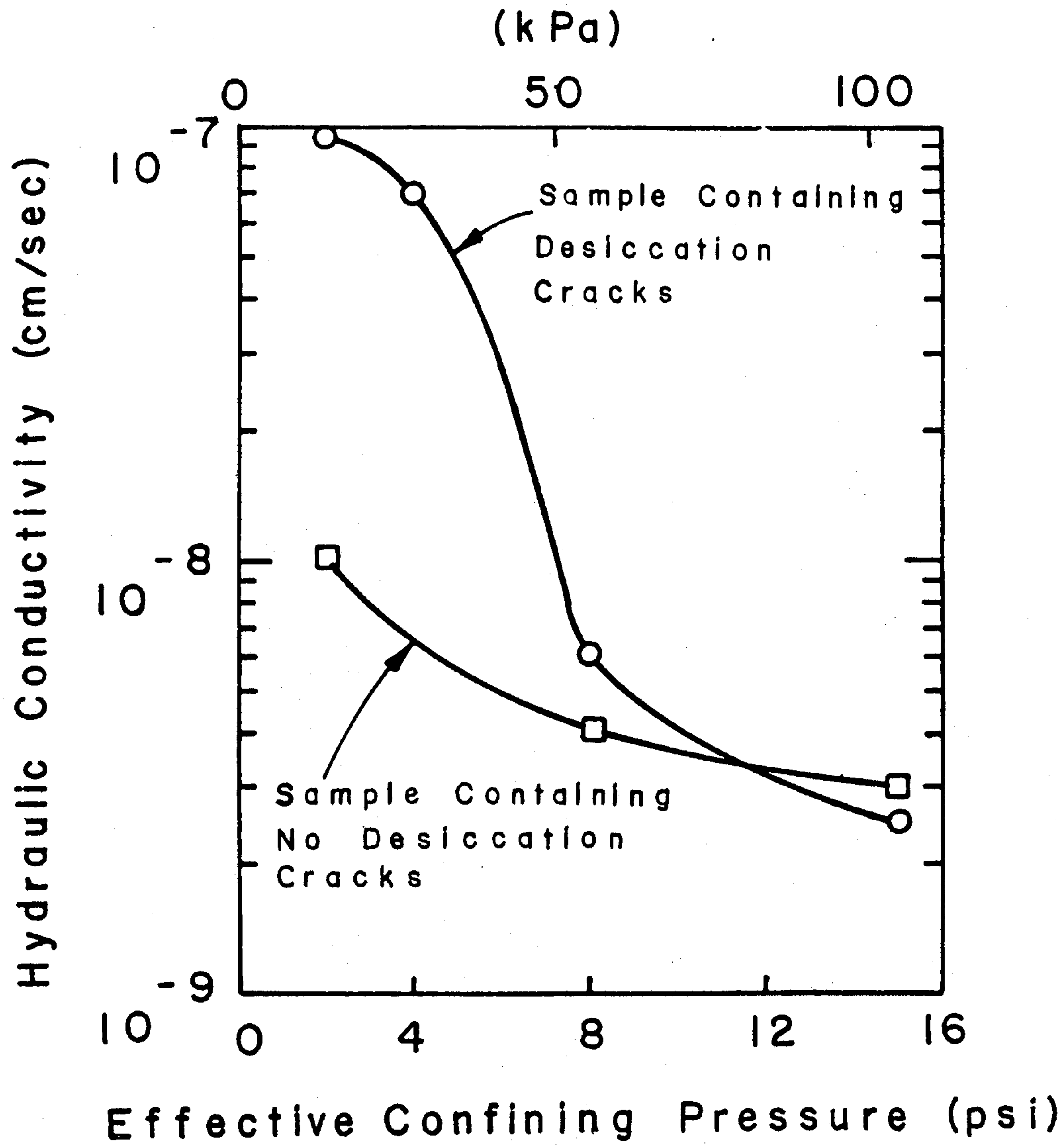

Figure 2-15. Effect of desiccation upon the hydraulic conductivity of compacted clay (6).

deep dynamic compaction, soil preloading, and the use of wick drains to consolidate sludges. These technologies for waste stabilization are presently emerging and appropriate descriptions are not available in the literature.

INTERFACIAL SHEAR

The stability of a cover system is controlled by the slope angle and the friction angles between the various interfaces of the cover system components. One potential problem with covers installed with a sloping surface is the risk that all or part of the cover system may slide downhill. The recent failure of a partly completed hazardous waste landfill provides an example of the problem (10). At this facility, slippage occurred between two components of the liner system in the landfill cell. The cell was filled such that a slope was created on the liner system that caused slippage.

The interfacial shearing characteristics of all components of a cover system, as well as internal shearing parameters of all soil layers, must be known in order to evaluate stability. If the soils are fully saturated and below the free water surface, e.g., during a heavy rainstorm, the stability is much less than if the soils are dry. Thus, one must consider both typical and worst-case

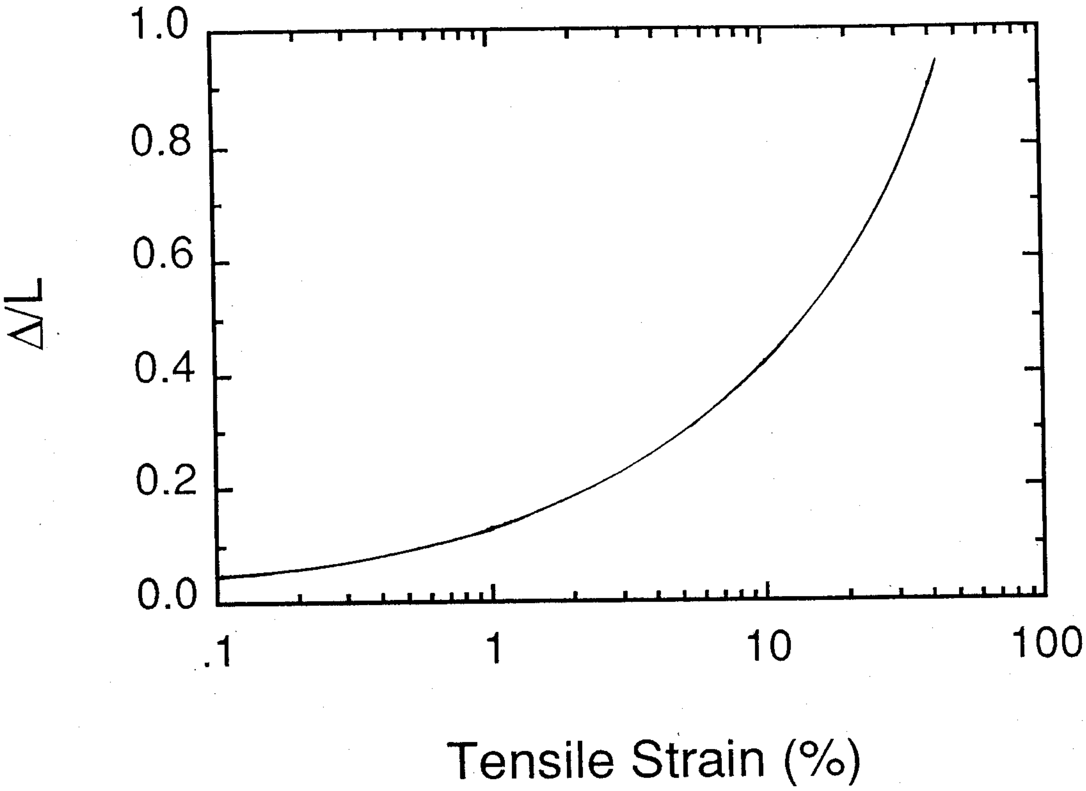

Figure 2-16. Relationship between distortion and tensile strain (9).

conditions when analyzing the stability of the cover system.

Methods of measuring interfacial friction between geosynthetic/geosynthetic or geosynthetic/soil interfaces are reviewed in detail by Takasumi et al. (11). No standard testing method exists, although one is under development by ASTM.

Seed and Boulanger (12) measured interfacial friction angles between a smooth high density polyethylene (HDPE) geomembrane and a compacted soil-bentonite mixture that contained 5 percent bentonite by dry weight. Interfacial friction angles were found to be very sensitive to compaction water content, dry unit weight, and the degree of wetting of the soil. For a given dry unit weight, increasing the molding water content or wetting the compacted soil reduced the interfacial friction angle. Increasing the density typically reduced the interfacial friction angle, as well. Unfortunately, the compaction conditions that would yield minimal hydraulic conductivity (i.e., compaction wet of optimum with a high energy of compaction) also yielded the lowest interfacial friction angles. Seed and Boulanger reported interfacial friction angles that were typically 5 to 10 degrees for the water content—unit weight combinations that would typically be employed to achieve minimal hydraulic conductivity.

The study of interfacial friction problems is an area of active research. At the present time, designers are cautioned to give careful consideration to the problem and to measure friction angles along all potential sliding surfaces using the proposed construction materials for testing. If adequate stability is not provided, the designer will need to consider alternative materials (e.g., rougher geomembranes with higher interfacial friction angles), flatter slopes, or reinforcement of the cover, e.g., with geogrids.

DRAINAGE LAYERS

Drainage layers are high-permeability materials used to drain fluids (such as infiltrating water) or gas produced from the waste. A drainage layer installed to drain infiltrating water is called a surface water collection and removal system. The hydraulic conductivity required for this layer depends upon the rate of infiltration, the slope of the layer, and the hydraulic conductivity of the underlying barrier layer. However, the efficiency of the drainage layer improves as the hydraulic conductivity of the drainage material increases. Thus, high hydraulic conductivity is a requirement for drainage layers.

The single most important factor controlling the hydraulic conductivity of sands and gravels is the amount of fine-grained material present. Geotechnical engineers define fine-grained materials as those materials that will pass through the openings of a No. 200 sieve (0.075 mm openings). A relatively small shift in the amount of fines

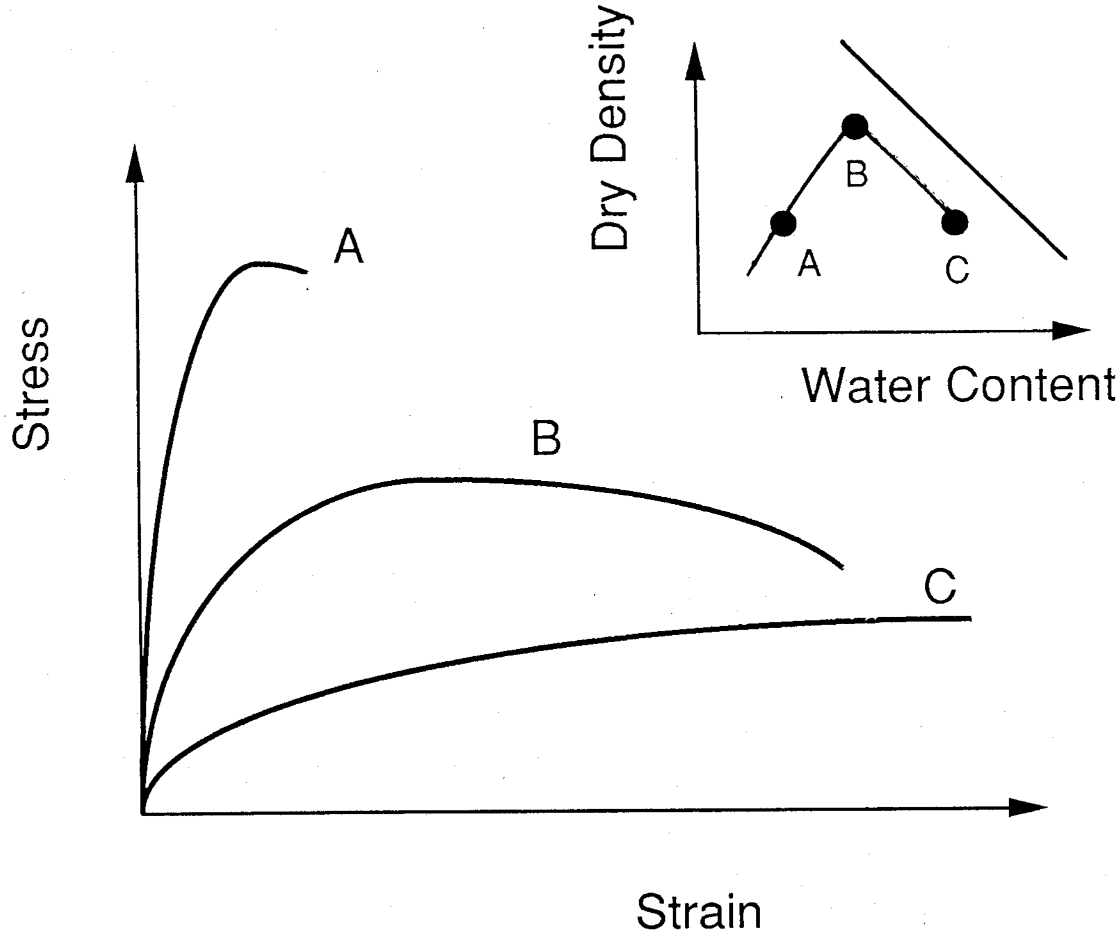

Figure 2-17. Relationship between shearing characteristics of compacted soils and conditions of compaction.

present in the soil can change the hydraulic conductivity by several orders of magnitude. The drainage material should be relatively free of fines if the material is to have a high hydraulic conductivity.

A minimal amount of compaction of the drainage materials in a cover is adequate to guard against settlement; excessive compaction is usually not necessary. In fact, excessive compaction may grind up soil particles, which would tend to lower the hydraulic conductivity of the drainage layer. However, sands may bulk if placed in a damp or wet condition, which can lead to an unacceptably loose material. If significant seismic ground shaking is possible at a site, compaction of drainage layers may be needed to minimize the risk of liquefaction-induced sliding.

Designers often place a highly permeable layer at the base of a cover system above gas-producing wastes, such as municipal solid waste. This layer aids in collecting gas and is called a gas collection layer. Adequate filters above and below the gas collection layer must be provided so that the collection layer does not become clogged with fine material. Vent pipes are normally placed in the gas collection layer at a frequency of approximately one per acre.

SUMMARY

Soils are used in cover systems to support growth of vegetation, to separate buried wastes or contaminated soils from the surface, to minimize the infiltration of water, and to aid in collecting and removing gases. The most challenging aspect of utilizing soils in cover systems is designing, constructing, and maintaining a barrier layer of low hydraulic conductivity. Soils can be compacted to achieve a low initial hydraulic conductivity, but the soils can be damaged by excessive differential settlement, desiccation, and other environmental stresses. Protecting a compacted soil liner from damage is therefore the greatest challenge to the designer.

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ferent geomembranes have been reported in EPA's technical resources document as follows:

PVC - 10 mil - 4.4 ml/m^2-day-atm.
- 20 mil - 3.3 ml/m^2-day-atm.

LLDPE (linear low density polyethylene)
- 18 mil - 2.3 ml/m^2-day-atm.

CSPE - 32 mil - 0.27 ml/m^2-day-atm.
- 34 mil - 1.6 ml/m^2-day-atm.

HDPE - 24 mil - 1.3 ml/m^2-day-atm.
- 34 mil - 1.4 ml/m^2-day-atm.

Biaxial Stresses via Subsidence

As the waste beneath the closure subsides, differential settlement is likely to occur. Thus a factor-of-safety formulation of FS = $\sigma_{allow}/\sigma_{reqd}$ is necessary. This situation has been modeled (see Appendix A, *Stability and Tension Considerations Regarding Cover Soils on Geomembrane Lined Slopes*), giving rise to the following formula for required strength (σ_{reqd}):

$$\sigma_{reqd} = \frac{2\,D\,L^2\,\gamma_{cs}\,H_{cs}}{3\,t\,(D^2 + L^{2)}}$$

where γ_{cs} = unit weight of cover soil
H_{cs} = height of cover soil
t = thickness of geomembrane
D, L = see Figure 3-1

The allowable strength σ_{allow} of the candidate geomembrane must be evaluated in a closely simulated test, e.g., GRI's GM-4 entitled "Three Dimensional Geomembrane Tension Test." Figure 3-2 presents the response to this test of a number of common geomembranes used in closure situations.

Planar Stresses via Friction

In addition to the above out-of-plane stresses, the cover soil over the geomembrane might develop greater frictional stresses than the soil material beneath it. This happens particularly if a wet-of-optimum clay is placed beneath. Again a factor-of-safety formulation is formed by comparing the allowable strength (T_{allow}) to the required strength (T_{reqd}) but now in force units rather than stress units, e.g., FS = T_{allow}/T_{reqd}. The required geomembrane tension can be obtained by the equation given in Figure 3-3 (see Appendix A for a more detailed discussion).

where c_{aU}, c_{aL} = adhesion of the material upper and lower of the geomembrane
δ_U, δ_L = friction angle of the material upper and lower of the geomembrane
ω = slope angle
L = slope length
W = unit width of slope
γ_{cs} = unit weight of cover soil
H_{cs} = height of cover soil

When calculated, the value of T_{reqd} in Figure 3-3 is compared to the T_{allow} of the candidate geomembrane. This value is currently taken from ASTM D-4885, the wide-width tensile test for geomembranes. Note that this value must be suitably adjusted for creep, long-term degradation, and any other site-specific situations that are considered relevant.

GEONET AND GEOCOMPOSITE SHEET DRAIN DESIGN CONCEPTS

Geonets and/or geocomposite drains are often used as surface water drains located immediately above the geomembrane in a landfill closure system. There are three aspects to the design that require attention: material compatibility, crush strength, and flow capability.

Compatibility

Since the liquid being conveyed by the geonet or drainage geocomposite is water, EPA 9090 testing is usually not warranted. The polymers from which these products are made are polyethylene (PE), polypropylene (PP), high-impact styrene (HIS) or other long-chain molecular structures that have good water resistance and long-term durability when covered by soil.

Crush Strength

The crush strength of the candidate product must be evaluated by comparing an allowable strength to a required stress, i.e., FS = $\sigma_{allow}/\sigma_{reqd}$. The allowable strength is taken as the rib lay-down for geonets and the telescoping crush strength for drainage geocomposites. Figure 3-4 illustrates common behavior for geonets and geocomposites. The test methods currently recommended are GRI GN-1 for compression behavior of geonets and GRI GC-4 for drainage geocomposites, i.e., for sheet drains.

The required stress is the dead load of the cover soil plus any live loads that may be imposed, such as construction and maintenance equipment.

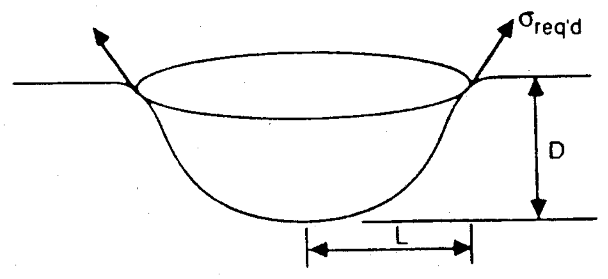

Figure 3-1. Required strength.

APPENDIX A

STABILITY AND TENSION CONSIDERATIONS REGARDING COVER SOILS ON GEOMEMBRANE-LINED SLOPES

Stability and Tension Considerations Regarding Cover Soils on Geomembrane Lined Slopes

by

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Abstract

The occurrence of cover soil instability in the form of sliding on geomembranes is far too frequent. Additionally, there have been cases of wide width tension failures of the underlying geomembranes when the friction created by the cover soil becomes excessive. While there are procedures available in the literature regarding rational design of those topics, it is felt that a unified step-by-step perspective might be worthwhile. It is in this light that this paper is written. Included are four separate, but closely interrelated, design models. They are the following;

- cover soil stability on side slopes when placed above a geomembrane,
- cover soil reinforcement provided by either geogrids or geotextiles,
- wide width tension mobilized in the geomembrane caused by the interface friction of the soils placed above and below the geomembrane, and
- circumferential tension mobilized in the geomembrane by subsidence of the subgrade material beneath the geomembrane.

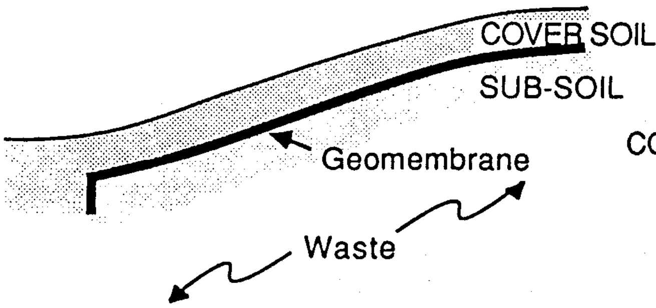
Each of these designs are developed in detail and a numeric problem is framed to illustrate the design procedure. Emphasized throughout the paper is the need for realistic laboratory test values of interface friction, in-plane tension and out-of-plane tension of the geomembranes. By having realistic experimental values of allowable strength they can be compared to the required, or design, strength for calculation of the resulting factor-of-safety against instability or failure.

Stability and Tension Considerations Regarding Cover Soils on Geomembrane Lined Slopes

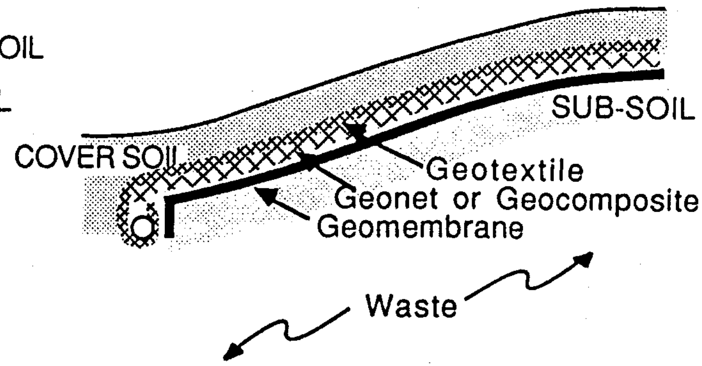
Introduction

Geomembrane lined soil slopes are common in many areas of civil engineering construction but nowhere are they more prevalent than in the environmental related field of the containment of solid waste. Cover soils on geomembranes placed above the waste as in landfill caps and closures as well as lined side slopes beneath the waste are commonplace as the sketches of Figure 1 indicate. The variations of soil types beneath the geomembrane as well as above the geomembrane are enormous. They range from moist clays in the form of composite liners to drainage sands and gravels of very high permeability. The likelihood of having other geosynthetic materials adjacent to the geomembranes (like geotextiles, geonets and drainage geocomposites) presents another set of variables to be considered. Lastly, the existence of many different geomembrane types, having different thicknesses, strengths, elongations and surface characteristics leads to the necessity of performing a rational design on such systems. Clearly, the development of design models to evaluate the stability of the overlying materials as well as the tensile stresses that may be induced in the underlying geomembranes should always be performed. Fortunately, both the stability of the overlying soil materials and the reduction of tensile stress in the geomembrane can be accommodated by reinforcing the cover soil with either geogrids or geotextiles. This is becoming known as veneer stability reinforcement and is necessitated due to a number of cover soil stability, or sloughing, failures, some of which are shown in the photographs of Figure 2.

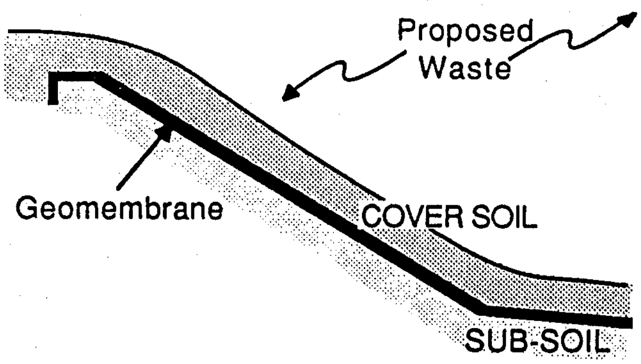
This paper presents several design models and their development into design equations for cover soil stability (both without and then with reinforcement) and for the induced tensile stresses that are mobilized in the underlying geomembrane. The approach taken in this paper will utilize a single geomembrane, but it should be recognized that double liners are frequently used beneath solid and liquid waste. Design considerations into the secondary liner, however, can be handled by reasonable extensions of the material to be presented.



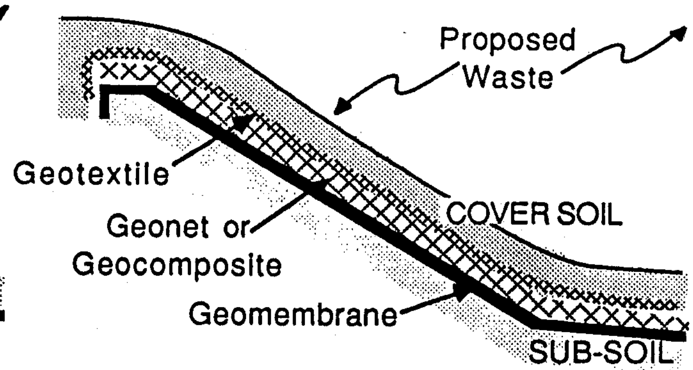
(a) Landfill Cover with Soil Above Geomembrane



(b) Landfill Cover with Drainage Geosynthetic Above Geomembrane



(c) Landfill Liner with Soil Above Geomembrane



(d) Landfill Liner with Drainage Geosynthetic Above Geomembrane

Figure 1 - Various Solid Waste Geomembrane Covers and Liners Involving Natural Soils and/or Drainage Geosynthetics

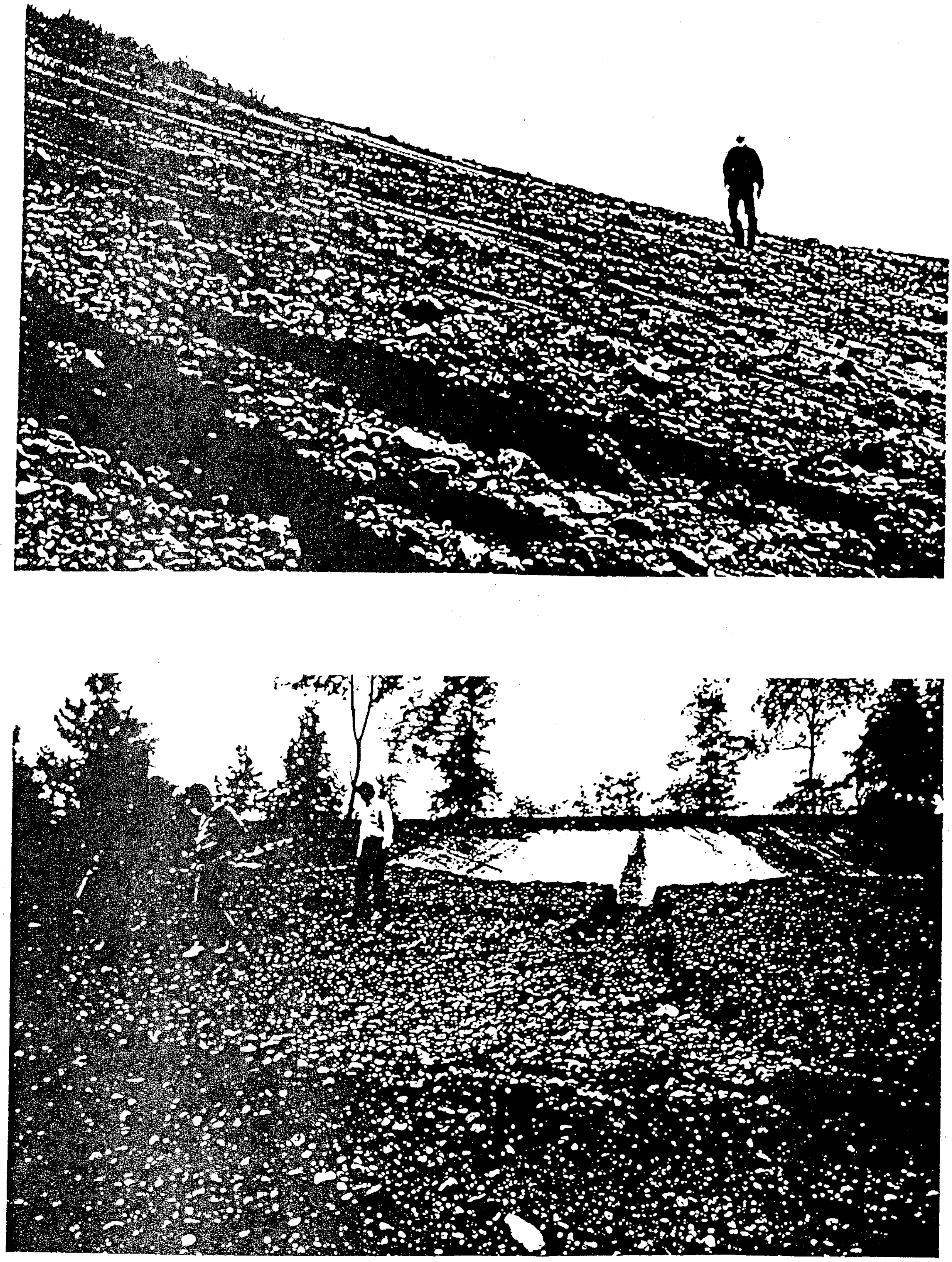


Figure 2 - Cases of Cover Soil Instability for Case 1(a) (upper photo) and for Case 1(c) (lower photo) as shown in Figure 1

Interface Friction Considerations

It will be seen that at the heart of the design equations to be developed in this paper are interface friction values between the geomembrane and the overlying soils or drainage geosynthetics and also against the underlying soils or drainage geosynthetics. These values are obtained by direct shear evaluation in simulated laboratory tests. Unfortunately, many aspects of the direct shear test have not yet been standardized (although ASTM has a Task Group working on a draft Standard), and many important details must be left to the design engineer and testing organization. For example, the following items need to be carefully considered.

- minimum or maximum size of shear box
- aspect dimensions of the test specimen
- type of fixity of the geomembrane to the shear box and to a substrate
- moisture conditions during normal stress application
- type of liquid to use during sample preparation and testing
- method and duration of normal stress application
- strain controlled or stress controlled shear application
- rate of shear application
- moisture conditions and drainage during shear application
- duration of test
- number of replicate tests at different normal stresses
- linearity of resulting failure envelope

Thus the use of reported values in the published literature can only be used with considerable caution and, at best, for preliminary design.[1-3] For final design and/or permitting, the site specific conditions and the proposed materials must be used in the tests so as to obtain realistic values of the shear strength parameters adhesion (c_a) and interface friction (δ). Additionally, tests should also be performed on the soil by itself so as to obtain a reference value for comparison to the inclusion of the geomembrane. Calculation of the adhesion efficiency on soil cohesion and a frictional efficiency to that of the soil by itself are meaningful in assessing the numeric results of the designs to follow, i.e.

$$E_c = c_a/c\,(100) \tag{1}$$

$$E_\phi = \tan\delta/\tan\phi\,(100) \tag{2}$$

where

E_c = efficiency on cohesion
c_a = adhesion of soil-to-geomembrane
c = cohesion of soil-to-soil
E_ϕ = efficiency on friction
δ = friction angle of soil-to-geomembrane
ϕ = friction angle of soil-to-soil

Past Investigations and Analyses

The isolation of free body diagrams depicting the site specific situation to be analyzed is certainly not new. It is a direct extension of geotechnical engineering of soil stability and is reasonably straightforward since the failure plane against the geomembrane is clearly defined. Thus a computer search is generally not necessary to locate the minimum factor-of-safety stability value. Also it should be recognized that the failure surface is usually linear, rather than circular, log spiral, or other complicated geometric shape in that it follows the surface of the geomembrane itself.

A procedure which nicely accommodates a clearly defined straight line slip surface has been developed by the U.S. Army Corps of Engineers.[4] Their wedge analysis procedures form the essence of the developments to follow. The graphic procedures are outlined in Reference #4 but are developed in this paper into design equations in a more rigorous manner. Also to be mentioned is the work of Giroud and Beech[5] and Giroud, et al.[6] in providing excellent insight into several aspects of the design.

In the first referenced paper, by Giroud and Beech[5], a two-part wedge method is utilized to arrive at a similar equation as ours except without an adhesion term. Also, the treatment at the top of the slope is slightly different. Their work will be referenced in the second problem of this set of four examples and a comparison of results will be made. In the second referenced paper, by Giroud, et al.[9], a large overburden stress necessitated the use of arching theory to recognize that a limiting value will occur when the geomembrane is located beneath deep fills. This is not the case with the shallow overburden stresses imposed by cover soils placed on geomembranes that are the focus of this paper. In the fourth example to be presented we will use the full thickness of the overburden times its unit weight. Additionally, we will not use a deformation/strain reduction value in the interest of being conservative. The reference cited by Giroud, et al.[9] should be used in this regard.

Model #1: Stability of Cover Soil Above a Geomembrane

Consider a cover soil (usually a permeable soil like gravel, sand or silt) placed directly on a geomembrane at a slope angle of "ω". Two discrete zones can be visualized as seen in Figure 3. Here one sees a small passive wedge resisting a long, thin active wedge extending the length of the slope. It is assumed that the cover soil is a uniform thickness and constant unit weight. At the top of the slope, or at an intermediate berm, a tension crack in the cover soil is considered to occur thereby breaking communication with additional cover soil at higher elevations.

Resisting the tendency for the cover soil to slide is the adhesion and/or interface friction of the cover soil to the specific type of underlying geomembrane. The values of "c_a" and "δ" must be obtained from a simulated laboratory direct shear test as described earlier. Note that the passive wedge is assumed to move on the underlying cover soil so that the shear parameters "c" and "ϕ", which come from soil-to-soil friction tests, will also be required.

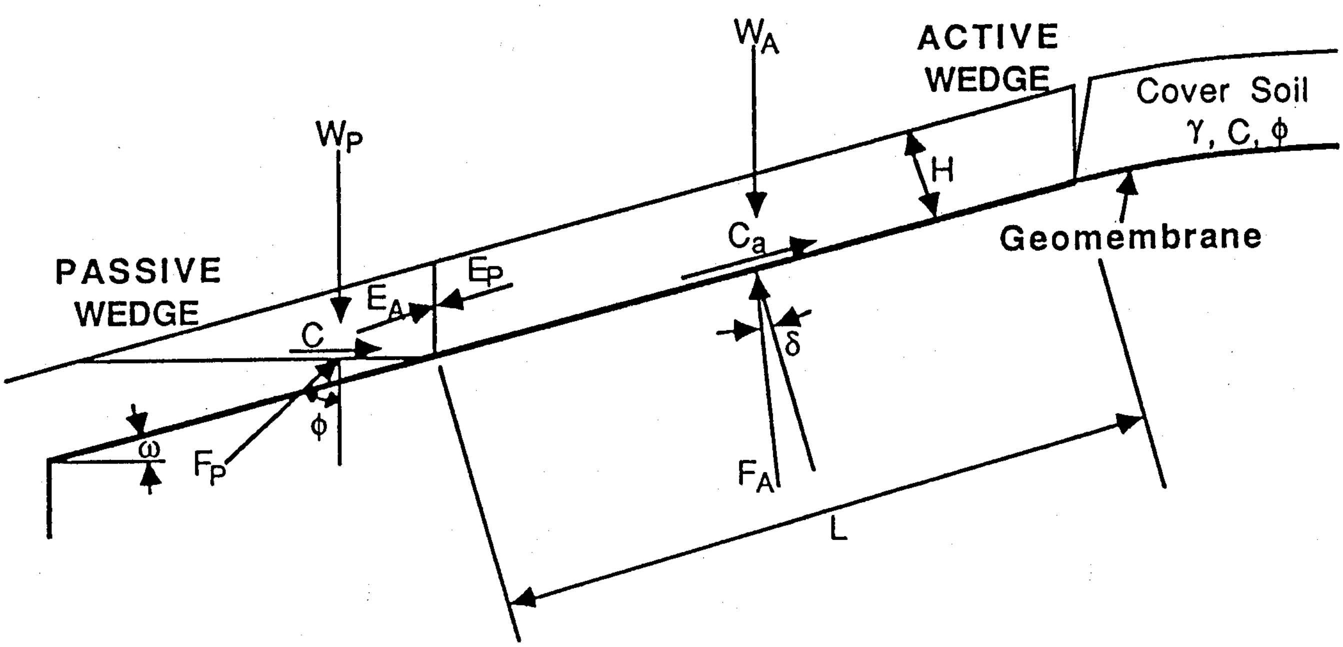


Figure 3 - Cross Section of Cover Soil on a Geomembrane Illustrating the Various Forces Involved on the Active and Passive Wedges

By taking free bodies of the passive and active wedges with the appropriate forces being applied, the following formulation for the stability factor-of-safety results, see Equation 3. Note that the equation is not an explicit solution for the factor-of-safety (FS), and must be solved indirectly. The complete development of the equation is given in Appendix "A".

$$(FS)^2 [0.5 \gamma LH \sin^2 (2 \omega)] - (FS) [\gamma LH \cos^2 \omega \tan \delta \sin (2 \omega) + c_a L \cos \omega \sin (2 \omega)$$
$$+ \gamma LH \sin^2 \omega \tan \phi \sin (2 \omega) + 2 c H \cos \omega + \gamma H^2 \tan \phi]$$
$$+ [(\gamma LH \cos \omega \tan \delta + c_a L) (\tan \phi \sin \omega \sin (2 \omega)] = 0 \qquad (3)$$

Using $ax^2 + bx + c = 0$, where

$$a = 0.5 \gamma LH \sin^2 2\omega$$
$$b = -[\gamma LH \cos^2\omega \tan \delta \sin (2\omega) + c_a L \cos \omega \sin (2 \omega)$$
$$+ \gamma LH \sin^2\omega \tan \phi \sin (2\omega) + 2cH \cos \omega + \gamma H^2 \tan \phi]$$
$$c = (\gamma LH \cos \omega \tan \delta + c_a L) (\tan \phi \sin \omega \sin (2\omega))$$

the resulting factor-of-safety is as follows:

$$FS = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \qquad (4)$$

When the calculated factor-of-safety value falls below 1.0, a stability failure of the cover soil sliding on the geomembrane is to be anticipated. However, it should be recognized that seepage forces, seismic forces and construction placement forces have not been considered in this analysis and all of these phenomena tend to lower the factor-of-safety. Thus a value of greater than 1.0 should be targeted as being the minimum acceptable factor-of-safety. An example problem illustrating the use of the above equations follows:

Example Problem: Given a soil cover soil slope of $\omega = 18.4°$ (i.e., 3 to 1), $L = 300$ ft., $H = 3.0$ ft, $\gamma = 120$ lb/ft^3, $c = 300$ lb/ft^2, $c_a = 0$, $\phi = 32°$, $\delta = 14°$, determine the resulting factor-of-safety

Solution:

$$a = 0.5 (120) (300) (3) \sin^2 (36.8°)$$
$$= 19{,}400 \text{ lb/ft}$$
$$b = -[(120) (300) (3) \cos^2 (18.4°) \tan (14°) \sin (36.8°)$$
$$+ 0 + (120) (300) (3) \sin^2 (18.4°) \tan (32°) \sin (36.8°)$$
$$+ 2 (300) (3) \cos (18.4°) + 120 (9) \tan (32°)]$$

$$= -[14523 + 0 + 4028 + 1708 + 675]$$

$$= -20{,}934 \text{ lb/ft}$$

$$c = [(120)(300)(3)\cos(18.4°)\tan(14°) + 0]$$

$$[\tan(32°)\sin(18.4°)\sin(36.8°)]$$

$$= [25500][0.118]$$

$$= 3019 \text{ lb/ft}$$

$$FS = \frac{20{,}934 + \sqrt{(-20934)^2 - 4(19400)(3019)}}{38{,}800}$$

FS = 0.91, which signifies that a stability failure will occur

Model #2: Reinforcement of Cover Soil on a Geomembrane

Once the cover soil factor-of-safety becomes unacceptably low for the site specific conditions (as illustrated in the previous problem), a possible solution to the situation is to add a layer of geogrid or geotextile reinforcement as shown in Figure 4. In the case of landfill covers, the tensile stresses that are mobilized in the reinforcement are carried over the crown to (generally) an equal and opposite reaction on the opposing slope. Alternatively, these stresses can be carried in friction via an anchorage mode of resistance as would occur in an intermediate berm situation. For a landfill liner, the stresses in the reinforcement are generally carried to an individual anchor trench extending behind the geomembrane anchor trench. If the reinforcement is a geogrid it is placed within the cover soil so that soil can strike-through the apertures and the maximum amount of anchorage against the transverse ribs can be mobilized. When using geotextiles, they can be placed directly on the geomembrane, or embedded within the cover soil so as to mobilize friction in both surfaces.

The tensile stress of the reinforcement layer per unit width is calculated by setting "E_A" equal to "E_p" in Figure 3 and solving for the unbalanced force "T" in Figure 4 which is required for a factor-of-safety equal to one. This value of T becomes T_{reqd} which is given in Equation 5. The complete development is available in Appendix "B".

$$T_{reqd} = \frac{\gamma LH \sin(\omega-\delta)}{\cos\delta} - c_a L - \frac{\cos\phi\left[\frac{cH}{\sin\omega} + \frac{\gamma H^2}{\sin 2\omega}\tan\phi\right]}{\cos(\phi+\omega)} \qquad (5)$$

This value is now compared to the allowable wide width tensile strength of the particular geogrid or geotextile under consideration, i.e.,

$$FS = T_{allow}/T_{reqd} \qquad (6)$$

Note that the value of "T_{allow}" must include such considerations as installation damage, creep and long-term degradation from chemical or biological interactions. If the value is obtained from a test such as ASTM D-4595, the wide width strip tensile test, the use of partial factors-of-safety is recommended to accommodate the above items.[7] An example problem using Equations 5 and 6 follows:

Example Problem: Continue the previous problem of cover soil instability where a geogrid with allowable wide width tensile strength of 4000 lb/ft is being considered (i.e., the value includes the above mentioned partial factors-of-safety). What is the resulting overall factor-of-safety? The parameters are $\omega = 18.4°$, L = 300 ft., H = 3.0 ft.,

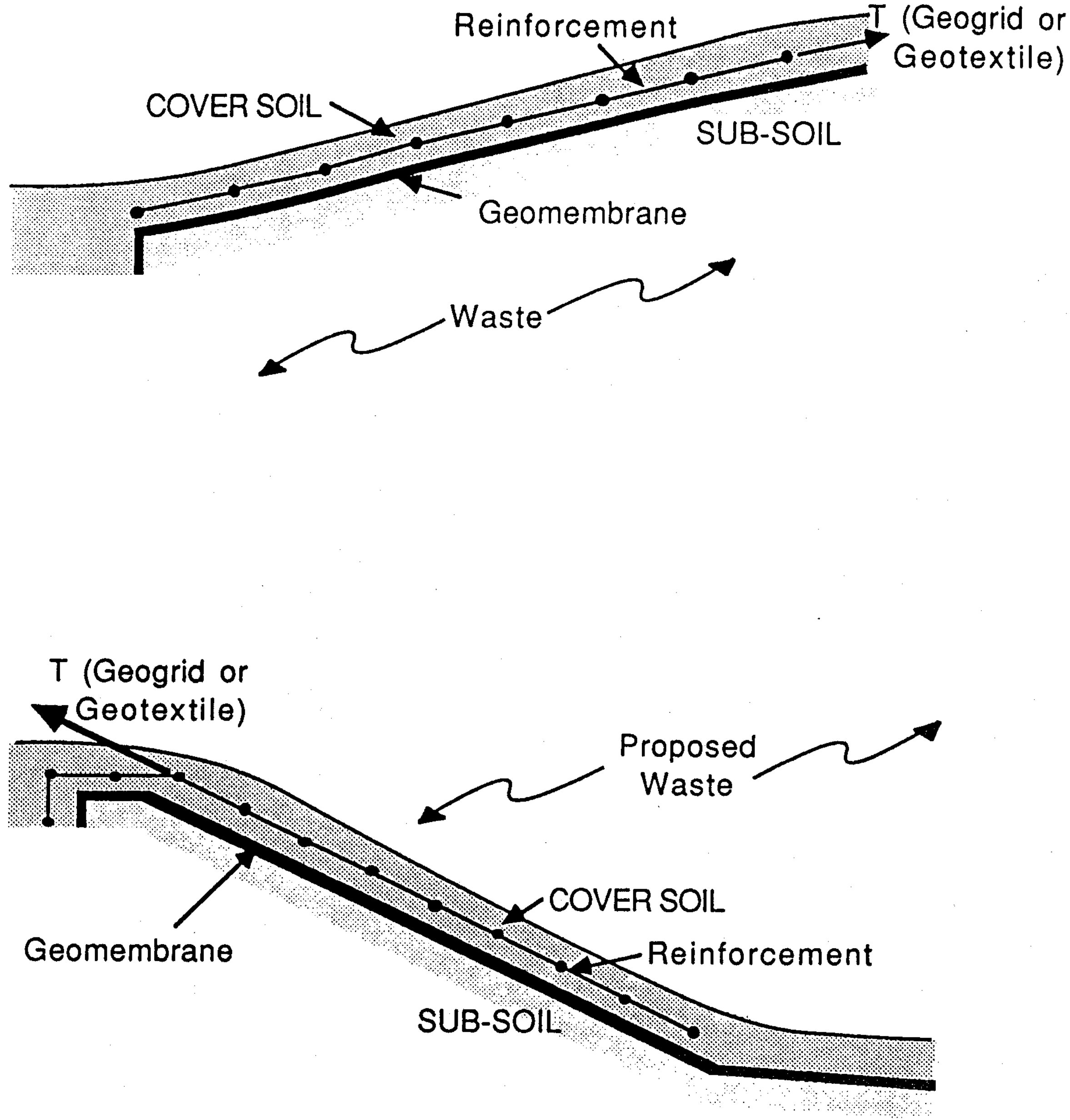


Figure .4 - Geogrid or Geotextile Reinforcement of a Cover Soil Above Waste and of a Cover Soil on a Geomembrane Beneath Waste

$\gamma = 120$ lb/ft^3, $c = 300$ lb/ft^2, $c_a = 0$, $\phi = 32°$, $\delta = 14°$.

Solution:

$$T_{reqd} = \frac{(120)\,(300)\,(3)\,\sin(4.4°)}{\cos(14°)} - 0 - \frac{\cos(32°)\left[\dfrac{(300)\,(3)}{\sin(18.4°)} + \dfrac{(120)\,(9)}{\sin(36.8°)}\tan(32°)\right]}{\cos(50.4°)}$$

$$= 8539 - 0 - 5292$$

$$T_{reqd} = 3247 \text{ lb/ft}$$

$$FS = T_{allow}/T_{reqd}$$

$$= \frac{4000}{3247}$$

FS = 1.23, which is marginally acceptable and a stronger reinforcement or a double layer should be considered.

Note: Using the formulation developed by Giroud and Beech[5] with the soil cohesion equal to zero results in a T_{reqd} = 6890 lb/ft, while the above formulation adjusted for a zero cohesion results in T_{reqd} = 7040 lb/ft. Thus the methods appear to be comparable to one another.

Model #3: Geomembrane Tension Stresses Due to Unbalanced Friction Forces

The shear stresses from the cover soil above the liner act downward on the underlying geomembrane and in so doing mobilize upward shear stresses beneath the geomembrane from the underlying soil. The situation is shown in the sketch of Figure 5.

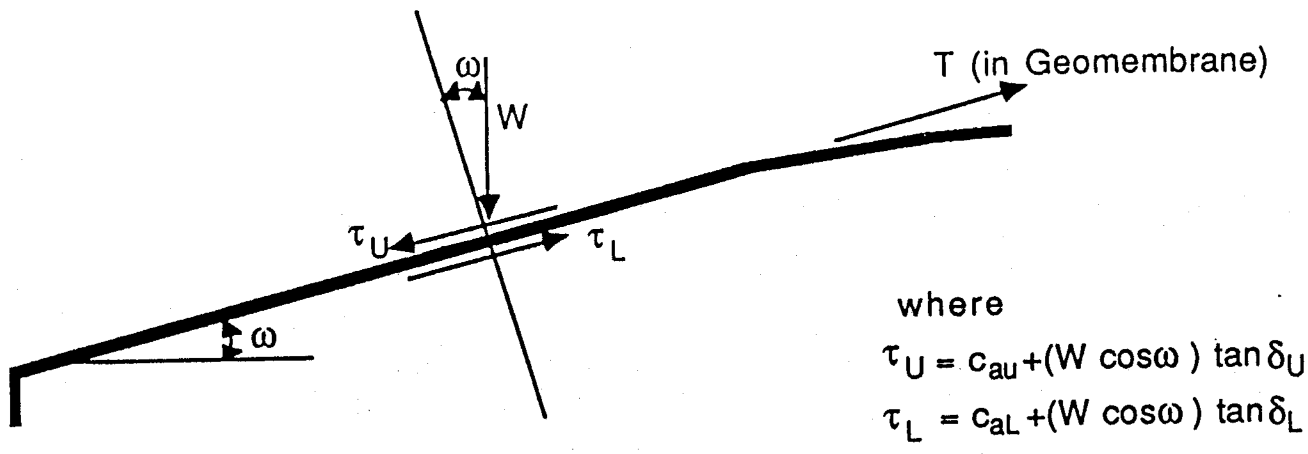


Figure 5 - Shear and Tensile Stresses Acting on a Covered Geomembrane

Here three different scenarios can be envisioned:

- If $\tau_U = \tau_L$, the geomembrane goes into a state of pure shear which should not be of great concern for most types of geomembranes
- If $\tau_U < \tau_L$, the geomembrane goes into a state of pure shear up to a magnitude of τ_U and the balance of $\tau_L - \tau_U$ is simply not mobilized
- If $\tau_U > \tau_L$, the geomembrane goes into a state of pure shear equal to τ_L and the balance of $\tau_U - \tau_L$ must be carried by the geomembrane in tension.

This latter case is the focus of this part of the design process. The situation generally occurs when a material with high interface friction (like sand or gravel) is placed above the geomembrane and a material with low interface friction (like high moisture content clay) is placed beneath the geomembrane. The essential equation for the design is as follows where "T" is in units of force per unit width, i.e., T/W. The complete derivation follows in Appendix "C".

$$T/W = \left[\left(c_{aU} - c_{aL}\right) + \gamma H \cos\omega \left(\tan\delta_U - \tan\delta_L\right)\right] L \tag{7}$$

The resulting value of force per unit width "T/W" is then compared to the allowable strength of the geomembrane which is shown schematically for different geomembranes in Figure 6. The target values are T_{break} for scrim reinforced geomembranes, T_{yield} for semi-crystalline geomembranes and T_{allow} (at a certain value of strain) for nonreinforced flexible geomembranes. Note that these curves should be obtained from a wide width tensile test which is currently under development in Committee D-35 on Geosynthetics in ASTM.[8]

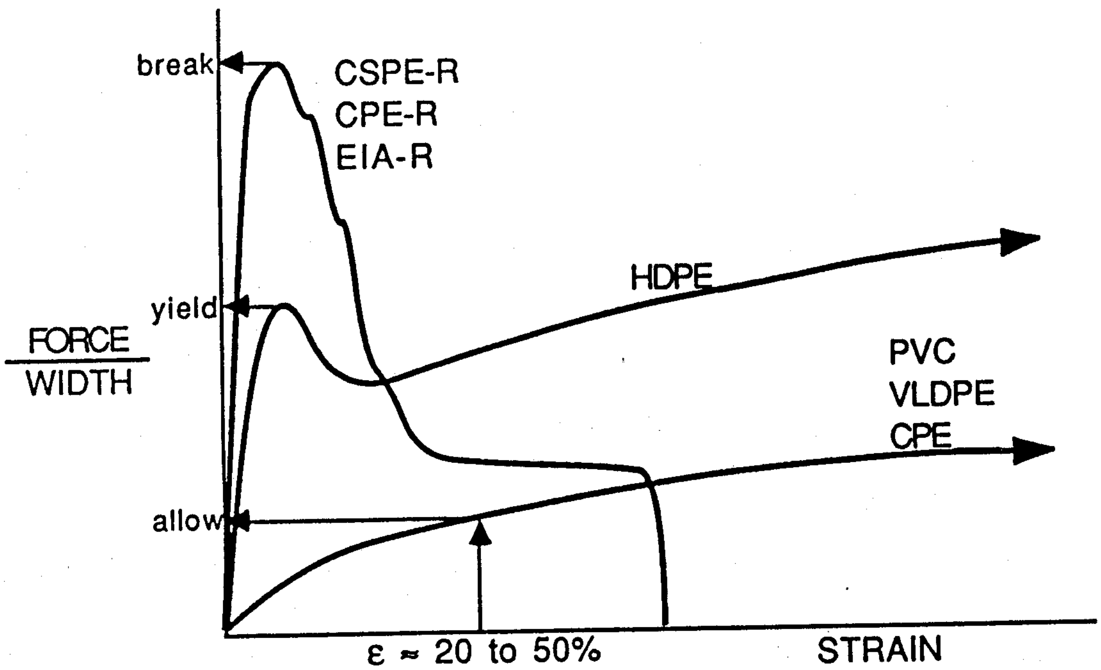


Figure 6 - Tensile Behavior of Various Geomembrane Types

Since there is generally no reduction for partial factors-of-safety in these values of laboratory obtained strength, the final factor-of-safety in the design should be quite conservative. An example problem follows:

Example Problem: Given the same landfill cover as described in the previous problems with a geomembrane having an allowable strength of 2000 lb/ft. The shear strength parameters of the geomembrane to the upper soil are $c_{aU} = 0$ lb/ft^2 and $\delta_U = 14°$ and to the lower soil are $c_{aL} = 50$ lb/ft^2 and $\delta_L = 5°$. Calculate the tension in the geomembrane and the resulting factor-of-safety against geomembrane failure.

Solution:

$$T/W = \left[\left(c_{a_U} - c_{a_L}\right) + \gamma H \cos\omega \left(\tan\delta_U - \tan\delta_L\right)\right] L$$

$$= \left[(0 - 50) + (120)(3)\cos(18.4°)\,[\tan(14°) - \tan(5°)\,]\right] 300$$

$$= [-50 + 55.3]\ 300$$

$$= 1590\ \text{lb/ft}$$

$$FS = T_{allow}/T_{reqd}$$

$$= \frac{2000}{1590}$$

$FS = 1.25$, which is barely acceptable.

Note: An alternative design to the above is to bench the cover soil (thereby decreasing the slope length) or use a liner whose lower surface has a higher adhesion or a higher friction surface, than the one used in the example thereby increasing "c_{aL}" and/or "δ_L".

Model #4: Geomembrane Tension Stresses Due to Subsidence

Whenever subsidence occurs beneath a geomembrane and it is supporting a cover soil some induced tensile stresses will occur due to out-of-plane forces from the overburden. Such subsidence is actually to be expected in closure situations above completed or abandoned landfills where the underlying waste is generally poorly compacted. The magnitude of the induced tensile stresses in the geomembrane depends upon the dimensions of the subsidence zone and on the cover soil properties.

The general scheme is shown in Figure 7 where the critical assumption is the shape of the deformed geomembrane. In the analysis which is provided in Appendix "D", the deformed shape is that of a spheroid of gradually decreasing center point along the symmetric axis of the deformed geomembrane.[9] As a worst case assumption, the geomembrane is assumed to be fixed at the circumference of the subsidence zone. The required tensile force in the geomembrane can be solved in terms of a force per unit width "T_{reqd}", or as a stress, i.e. "σ_{reqd}". The latter will be used in this analysis since it will be compared to a laboratory test method resulting in the compatible term. The necessary design equation is as follows where the specific terms are given in Figure 7.

$$\sigma_{reqd} = \frac{2\,DL^2\,\gamma_{cs}\,H_{cs}}{3\,t\,(D^2 + L^2)} \qquad (8)$$

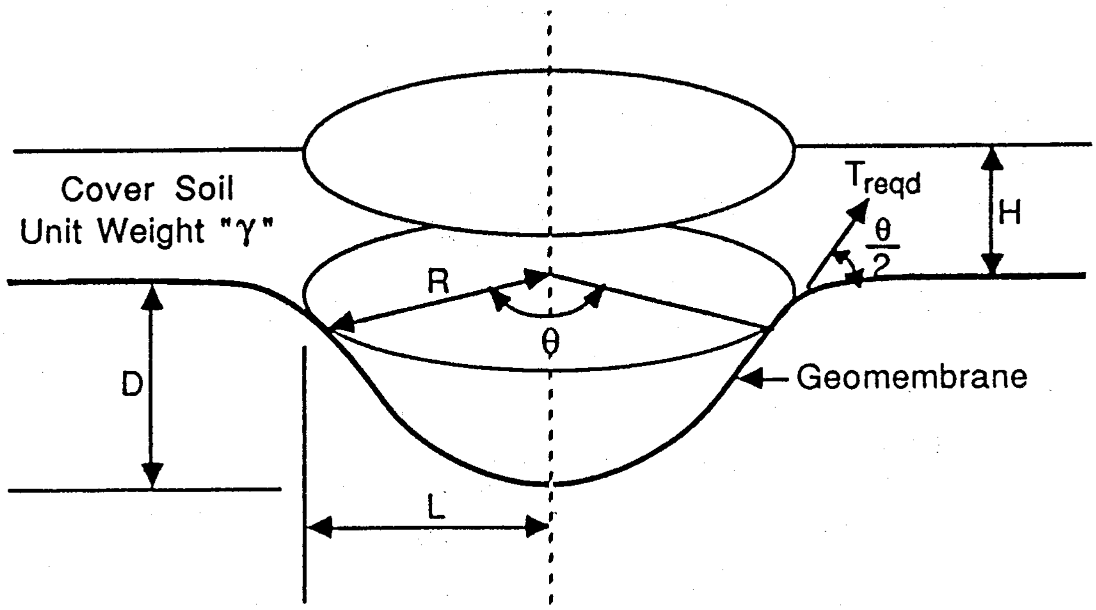


Figure 7 - Tensile Stresses in a Geomembrane Mobilized by Cover Soil and Caused by Subsidence

Upon calculating the value of σ_{reqd} for the site specific situation under consideration, it is compared to an appropriate laboratory simulation test. Recommended at this time is a three-dimensional, out-of-plane, tension test of the same configuration as Figure 6. It is available as GRI Test Method GM-4.[10] Thus the formulation for the final factor-of-safety becomes the following:

$$FS = \sigma_{allow}/\sigma_{reqd} \tag{7}$$

Since the value of σ_{allow} is used directly from the test method without any reduction in the form of partial factors-of-safety, relatively conservative values should be required. An example problem follows:

Example Problem: Given the same cover soil situation as in the previous example, except now a local subsidence occurs which is estimated to be 1.0 ft deep by 3.0 ft radius. The geomembrane is 40 mils thick has a σ_{allow} of 1000 lb/in^2. Determine the factor-of-safety of the geomembrane against the mobilized tensile stresses.

Solution:

$$\sigma_{reqd} = \frac{2\, DL^2\, \gamma_{cs}\, H_{cs}}{3\, t\, (D^2 + L^2)}$$

$$= \frac{2\,(1.0)\,(3.0)^2\,(120)\,(3.0)}{3\,(0.040/12)\,[(1.0)^2 + (3.0)^2]}$$

$$= 64{,}800 \text{ lb/ft}^2$$

$$\sigma_{reqd} = 450 \text{ lb/in}^2$$

$$FS = \sigma_{allow}/\sigma_{reqd}$$

$$= 1000/450$$

$$= 2.2, \text{ which is acceptable}$$

Summary and Conclusions

The occurrence of cover soils sliding off geomembrane lined slopes is not an infrequent incident. While less obvious, but of even greater concern, there are often tensile stresses imposed on the underlying geomembrane. The occurrence of extensive tensile failures of geomembranes on side slopes is also known to have occurred.. This paper is focused toward a series of four design models to be used to analyze various aspects of the situation.

The first model considered the cover soil's stability by itself. The design procedure is straightforward but it does require a set of carefully generated direct shear tests to realistically obtain the interface friction parameters.

The growing tendency toward steeper and longer slope angles gives rise to the second design model which is veneer reinforcement of the cover soil. Geogrids and geotextiles have shown that they can nicely reinforce the cover soil and the first design example was modified accordingly. The design leads to the calculation of the required tensile strength of the reinforcement. This value must then be compared to a laboratory generated wide width tensile strength of the candidate reinforcement material. It is important in this regard to consider long term implications which can be addressed by partial factors-of-safety.

Both of the above analyses serve to set up the third design scenario, that being a calculation procedure for determination of the induced tensile stresses in the underlying geomembrane brought on by unbalanced friction values. Whenever the frictional characteristics beneath the liner are low (e.g., when the liner is placed on a high moisture clay soil as it is in a composite liner), this type of analysis should be performed. The tensile stress in the geomembrane is then compared to the wide width tensile strength of the geomembrane for its resulting factor-of-safety.

Lastly, a design procedure for calculation of out-of-plane generated tensile stresses in the geomembrane was developed. This situation could readily arise by subsidence of solid waste beneath the geomembrane. The resulting tensile stresses in the geomembrane must then be compared to a properly simulated laboratory test for the factor-of-safety. Such three dimensional axi-symmetric test procedures are currently available.

Each of the four above described models along with their design/analysis procedures were illustrated by means of an example problem dealing with a cover soil in a solid waste closure situation. This type of application is the primary focus of the paper. However, similar situations can arise elsewhere. For example, the same situation occurs in the case of gravel covered primary geomembrane liners on the side slopes of unfilled, or partially filled, landfills. These slopes may have to be exposed to the elements for many years until the waste provides sufficient passive resistance and final stability. In the meantime, cover soil instability will cause sloughing and can expose the geomembrane to ultraviolet light, high temperatures via direct exposure, and a significantly shortened lifetime.

Hopefully, the use of design models such as presently here (and elsewhere), coupled with the appropriate test method simulating actual field behavior, will lead to recognition of the problems encountered and to a widespread rational design of cover soils on geomembrane lined side slopes.

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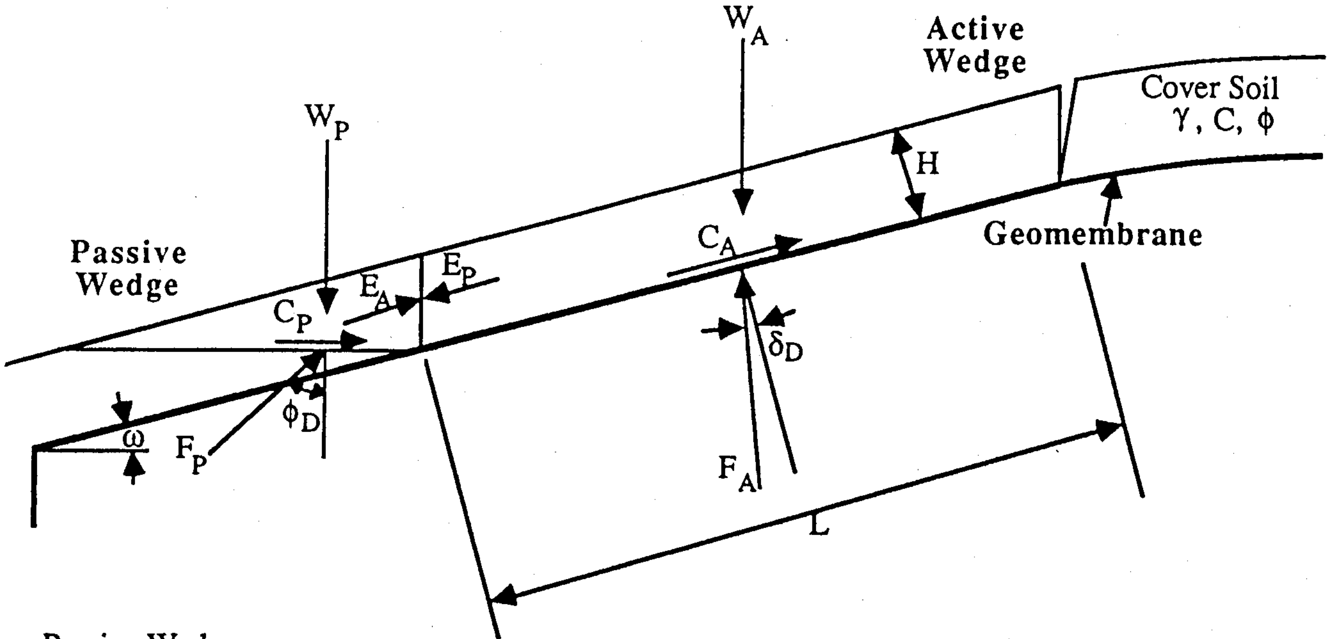
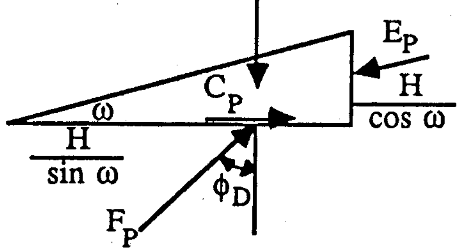
Appendix "A"
Derivation of FS for Cover Soil Stability on a Geomembrane

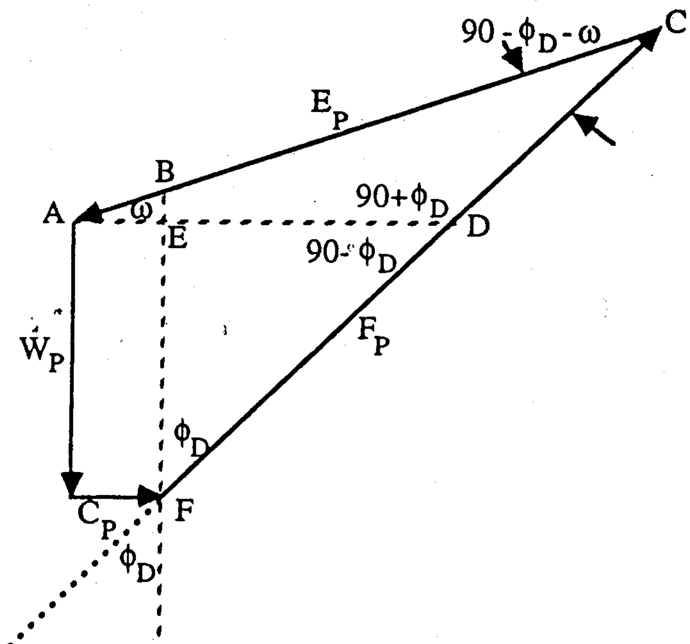


Passive Wedge

$$W_P = \frac{1}{2}\gamma \frac{H^2}{\sin\omega \cos\omega} = \frac{\gamma H^2}{\sin 2\omega}$$



$$C_P = \frac{C}{FS}\,\frac{H}{\sin\omega}$$



Passive Wedge

$$\overline{EF} = W_P$$

$$\frac{\overline{DE}}{\sin \phi_D} = \frac{\overline{EF}}{\sin (90^\circ - \phi_D)} = \frac{W_P}{\cos \phi_D}$$

$$\overline{DE} = W_P \cdot \tan \phi_D$$

$$\frac{E_P}{\sin (90^\circ + \phi_D)} = \frac{\overline{AD}}{\sin (90^\circ - \phi_D - \varpi)}$$

$$\frac{E_P}{\cos \phi_D} = \frac{C_P + W_P \cdot \tan \phi_D}{\cos (\phi_D + \varpi)}$$

$$E_P = \frac{\cos \phi_D \cdot [C_P + W_P \cdot \tan \phi_D]}{\cos (\phi_D + \varpi)}$$

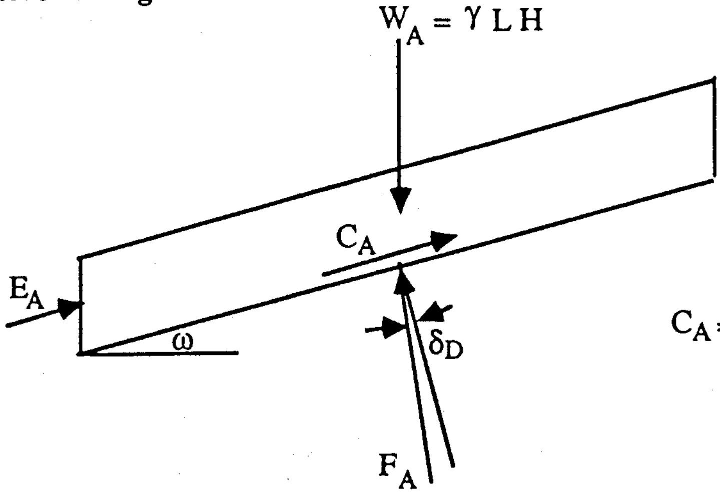
$$= \frac{\cos \phi_D \cdot \left[\frac{C}{FS} \cdot \frac{H}{\sin \varpi} + \frac{\gamma \cdot H^2}{\sin 2\varpi} \cdot \tan \phi_D \right]}{\cos (\phi_D + \varpi)}$$

$$= \frac{\cos \phi_D}{(\cos \phi_D \cos \varpi - \sin \phi_D \sin \varpi)} \left[\frac{C \cdot H}{FS \cdot \sin \varpi} + \frac{\gamma \cdot H^2}{2 \sin \varpi \cos \varpi} \cdot \frac{\tan \phi}{FS} \right]$$

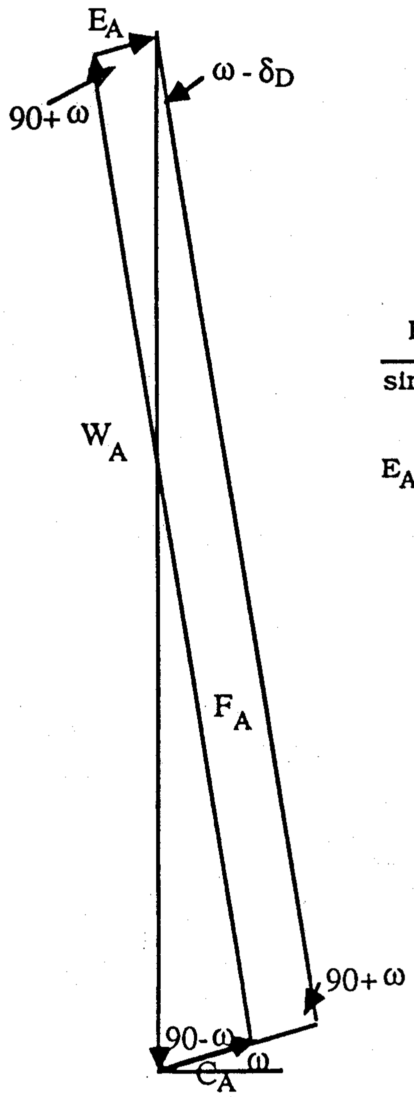
$$= \frac{1}{(\cos \varpi - \tan \phi_D \sin \varpi)} \left[\frac{2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi}{2 \cdot \sin \varpi \cdot \cos \varpi \cdot FS} \right]$$

$$= \frac{FS}{(FS \cdot \cos \varpi - \tan \phi \cdot \sin \varpi)} \left[\frac{2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi}{2 \cdot \sin \varpi \cdot \cos \varpi \cdot FS} \right]$$

Active Wedge



$$C_A = \frac{C_a L}{FS}$$



$$\frac{E_A + C_A}{\sin(\omega - \delta_D)} = \frac{W_A}{\sin(90^\circ + \delta_D)} = \frac{W_A}{\cos \delta_D}$$

$$E_A = \frac{W_A \sin(\omega - \delta_D)}{\cos \delta_D} - C_A$$

$$= \frac{W_A\left[\sin \omega \cos \delta_D - \cos \omega \sin \delta_D\right]}{\cos \delta_D} - C_A$$

$$= \gamma \cdot L \cdot H\,(\sin \omega - \cos \omega \tan \delta_D) - \frac{C_a \cdot L}{FS}$$

$E_A = E_P$

$$\gamma \cdot L \cdot H \cdot (\sin \varpi - \cos \varpi \cdot \tan \delta_D) - \frac{C_a \cdot L}{FS} = \frac{(2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)}{(FS \cdot \cos \varpi - \tan \phi \cdot \sin \varpi) \cdot (\sin 2\varpi)}$$

$$\gamma \cdot L \cdot H \cdot \left(\sin \varpi - \frac{\cos \varpi \cdot \tan \delta}{FS}\right) - \frac{C_a \cdot L}{FS} = \frac{(2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)}{(FS \cdot \cos \varpi - \tan \phi \cdot \sin \varpi) \cdot (\sin 2\varpi)}$$

$$\frac{\gamma \cdot L \cdot H \cdot (\sin \varpi \cdot FS - \cos \varpi \cdot \tan \delta) - C_a \cdot L}{FS} = \frac{(2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)}{(FS \cdot \cos \varpi \cdot \sin 2\varpi - \tan \phi \cdot \sin \varpi \cdot \sin 2\varpi)}$$

$$\frac{\gamma \cdot L \cdot H \cdot \sin \varpi \cdot FS - \gamma \cdot L \cdot H \cdot \cos \varpi \cdot \tan \delta - C_a \cdot L}{FS} = \frac{(2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)}{(FS \cdot \cos \varpi \cdot \sin 2\varpi - \tan \phi \cdot \sin \varpi \cdot \sin 2\varpi)}$$

$$FS^2 \cdot (\gamma \cdot L \cdot H \cdot \sin \varpi \cdot \cos \varpi \cdot \sin 2\varpi) - FS \cdot (\gamma \cdot L \cdot H \cdot \cos^2 \varpi \cdot \tan \delta \cdot \sin 2\varpi)$$

$$- FS \cdot (C_a \cdot L \cdot \cos \varpi \cdot \sin 2\varpi) - FS \cdot (\gamma \cdot L \cdot H \cdot \sin^2 \varpi \cdot \tan \phi \cdot \sin 2\varpi)$$

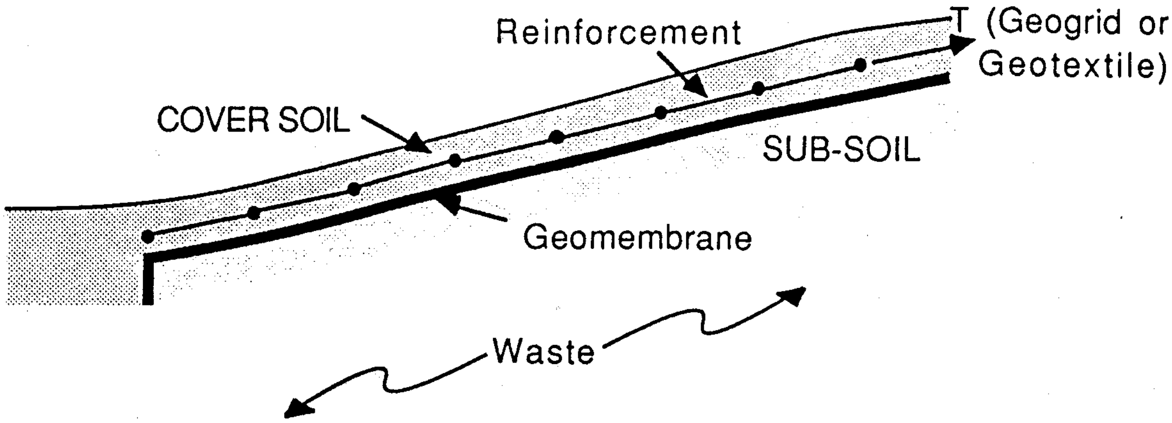
$$+ (\gamma \cdot L \cdot H \cdot \cos \varpi \cdot \tan \delta + C_a \cdot L) \cdot (\tan \phi \cdot \sin \varpi \cdot \sin 2\varpi) = FS \cdot (2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)$$

$$FS^2 \cdot \left(\frac{1}{2} \cdot \gamma \cdot L \cdot H \cdot \sin^2 2\varpi\right) - FS \cdot (\gamma \cdot L \cdot H \cdot \cos^2 \varpi \cdot \tan \delta \cdot \sin 2\varpi + C_a \cdot L \cdot \cos \varpi \cdot \sin 2\varpi$$

$$+ \gamma \cdot L \cdot H \cdot \sin^2 \varpi \cdot \tan \phi \cdot \sin 2\varpi + 2 \cdot C \cdot H \cdot \cos \varpi + \gamma \cdot H^2 \cdot \tan \phi)$$

$$+ (\gamma \cdot L \cdot H \cdot \cos \varpi \cdot \tan \delta + C_a \cdot L) \cdot (\tan \phi \cdot \sin \varpi \cdot \sin 2\varpi) = 0$$

Appendix "B"
Derivation of Required Tensile Strength of Geogrid or Geotextile Reinforcement of Cover Soil on a Geomembrane



$$E_P + T = E_A \Rightarrow T = E_A - E_P$$

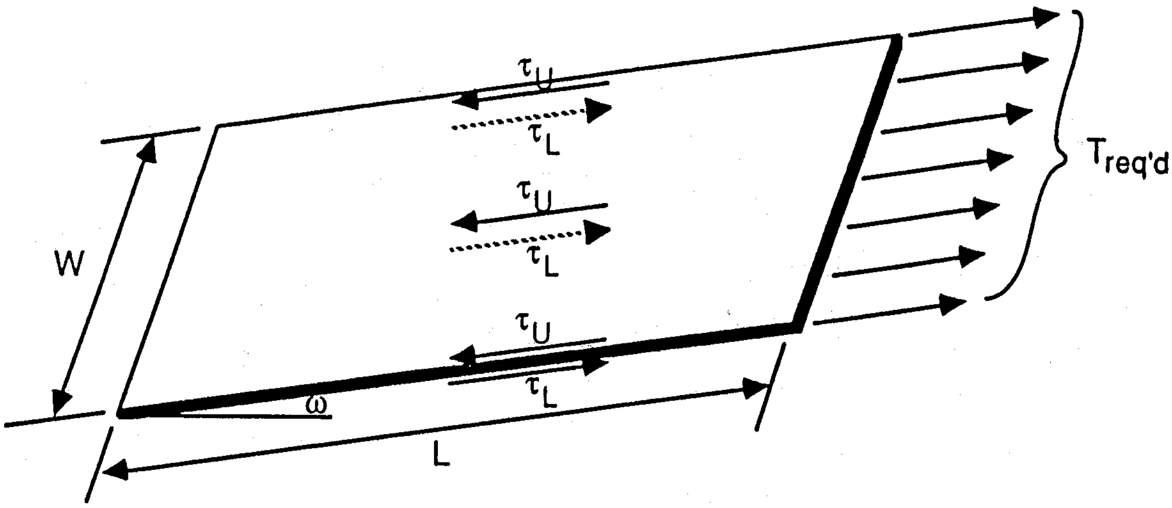
$$E_A = \frac{\gamma \cdot L \cdot H \cdot \sin(\varpi - \delta_D)}{\cos \delta_D} - C_A$$

$$E_P = \frac{\cos \phi_D \cdot \left[\frac{C}{FS} \cdot \frac{H}{\sin \varpi} + \frac{\gamma \cdot H^2}{\sin 2\varpi} \cdot \tan \phi_D \right]}{\cos(\phi_D + \varpi)}$$

$$FS = 1, \ \delta_D = \delta, \ \phi_D = \phi$$

$$T_{reqd} = \frac{\gamma \cdot L \cdot H \cdot \sin(\varpi - \delta)}{\cos \delta} - C_A - \frac{\cos \phi \cdot \left[\frac{C \cdot H}{\sin \varpi} + \frac{\gamma \cdot H^2}{\sin 2\varpi} \cdot \tan \phi \right]}{\cos(\phi + \varpi)}$$

Appendix "C"
Derivation of Geomembrane Tensile Stress Due to Unbalanced Friction Forces



$$FS = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{T + \tau_L \cdot W \cdot L}{\tau_U \cdot W \cdot L}$$

(a) If $\tau_U = \tau_L$, FS > 1, pure shear @ $\tau_U = \tau_L$

(b) If $\tau_U < \tau_L$, FS > 1, pure shear = τ_U, rest of $(\tau_L - \tau_U)$ is not mobilized.

(c) If $\tau_U > \tau_L$, FS may be >, = or < 1, which depend on the T value

when FS = 1 $\Rightarrow \tau_U \cdot W \cdot L = T + \tau_L \cdot W \cdot L$

$$T = \tau_U \cdot W \cdot L - \tau_L \cdot W \cdot L$$

$$\frac{T}{W} = (\tau_U - \tau_L) \cdot L$$

$$\tau = C + \sigma_n \cdot \tan\phi, \ \sigma_n = \gamma \cdot H \cdot \cos\varpi$$

$$\tau_U = C_{aU} + \gamma \cdot H \cdot \cos\varpi \cdot \tan\delta_U$$

$$\tau_L = C_{aL} + \gamma \cdot H \cdot \cos\varpi \cdot \tan\delta_L$$

$$\frac{T}{W} = [(C_{aU} - C_{aL}) + \gamma \cdot H \cdot \cos\varpi \cdot (\tan\delta_U - \tan\delta_L)] \cdot L$$

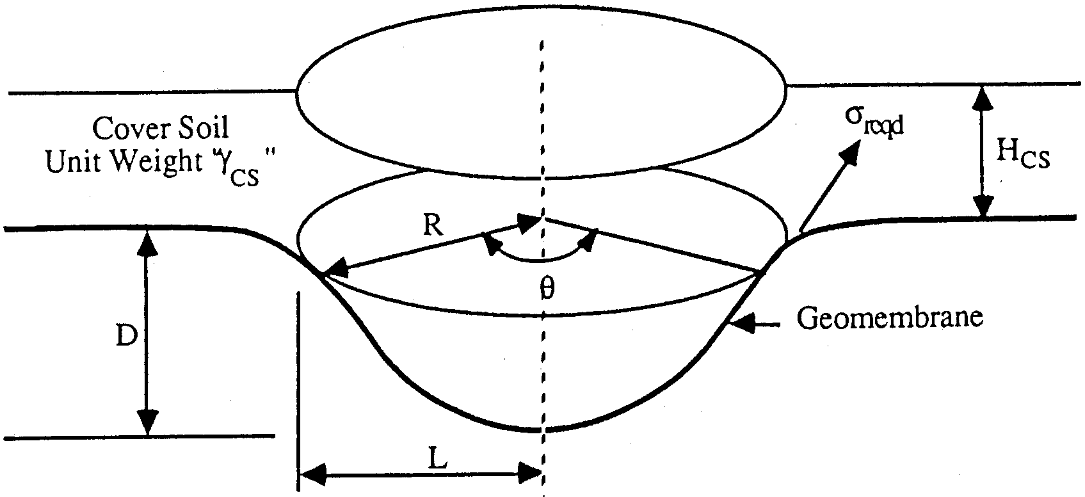
$$FS = \frac{\sigma_{allow}}{\sigma_{reqd}}$$

where $\sigma_{allow} = \dfrac{\sigma_{ult}}{(FS)_T}$

σ_{ult} = Geomembrane Wide Width Tensile Test

$$\sigma_{reqd} = \left(\frac{T}{W}\right) \cdot \left(\frac{1}{t}\right)$$

Appendix "D"
Derivation of Geomembrane Tensile Stress Due to Subsidence of Material Beneath the Geomembrane

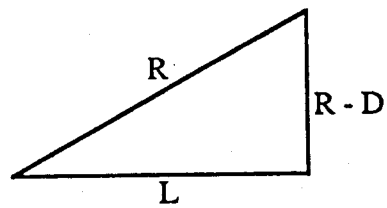


C (circumference) $= 2 \cdot \pi \cdot L$

t = thickness of the geomembrane

$$R^2 = (R - D)^2 + L^2 \Rightarrow R^2 = R^2 - 2 \cdot R \cdot D + D^2 + L^2$$

$$R = \frac{D^2 + L^2}{2 \cdot D}$$



$$\int_0^L (2 \cdot \pi \cdot r) \cdot dr \cdot \gamma_{CS} \cdot H_{CS} \cdot r = \sigma_{reqd} \cdot t \cdot C \cdot R$$

$$\frac{2}{3} \cdot \pi \cdot L^3 \cdot \gamma_{CS} \cdot H_{CS} = \sigma_{reqd} \cdot t \cdot (2 \cdot \pi \cdot L) \cdot R$$

$$\sigma_{reqd} = \frac{\frac{2}{3} \cdot \pi \cdot L^3 \cdot \gamma_{CS} \cdot H_{CS}}{t \cdot (2 \cdot \pi \cdot L) \cdot R}$$

$$= \frac{L^2 \cdot \gamma_{CS} \cdot H_{CS}}{3 \cdot t \cdot R}$$

$$\sigma_{reqd} = \frac{2 \cdot D \cdot L^2 \cdot \gamma_{CS} \cdot H_{CS}}{3 \cdot t \cdot (D^2 + L^2)}$$