GEOSYNTHETIC LANDFILL COVER DESIGN METHODOLOGY AND CONSTRUCTION EXPERIENCE IN THE PACIFIC NORTHWEST

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ABSTRACT

The use of geosynthetics in landfill covers has increased in recent years and are likely to be required for most municipal solid waste (MSW) landfills in the U.S. by the new Environmental Protection Agency (EPA) subtitle D regulations. Critical cover-design concerns are stability and drainage. An infinite slope analysis that uses the properties of site-specific materials is considered appropriate for evaluating stability. Estimating the amount of seepage into the drainage layer above a geomembrane cover on steep slopes is critical for designing proper spacing of the layer's drainage outlets. The design should be based on interface shear strength and permeability testing with site-specific materials.

Well-prepared specifications and a construction quality assurance program are crucial to successful geosynthetic cover installations. This paper reflects experience gained from designing and constructing landfill covers with geomembranes on 11 projects totaling over 200 acres in Oregon and Washington. Lessons learned from these projects will lead to improvements in future designs.

INTRODUCTION

Geomembranes are increasingly used in landfill cover designs because they

- Are often preferred in regulations
- Reduce long-term leachate generation better than soil-only covers, especially in wet climates
- May reduce long-term liability by limiting future leachate generation and controlling gas
- Often cost less than low-permeability soil

Recent EPA regulations (Federal Register, 1991) will likely require composite covers (geomembrane over soil) on most new MSW landfills that have bottom geomembrane liners. Koerner and Daniel (1992) discuss the trends, benefits, and issues related to using geomembranes in landfill covers. Special design issues for landfill covers with geomembranes include

- Cover slope stability
- Settlement (total and differential)
- Landfill gas control
- Side slope seeps
- Construction methods and materials

Other more standard civil design items such as access roads, erosion, surface drainage structures, and vegetation must account for the special issues listed above.

All elements of the completed design must be considered when budgeting for long-term maintenance and planning for future site end use.

This paper describes a typical landfill cover design using a geomembrane barrier layer. The design discussion focuses on slope stability, taking into account partial saturation and seepage forces in the cover. Construction issues and production rates relating to the layered elements of the cover design are described at the end of the paper.

TYPICAL COVER DESIGN ELEMENTS AND MATERIALS

The following are the elements, from bottom to top, of a typical landfill cover with a geomembrane barrier layer (see Figure 1).

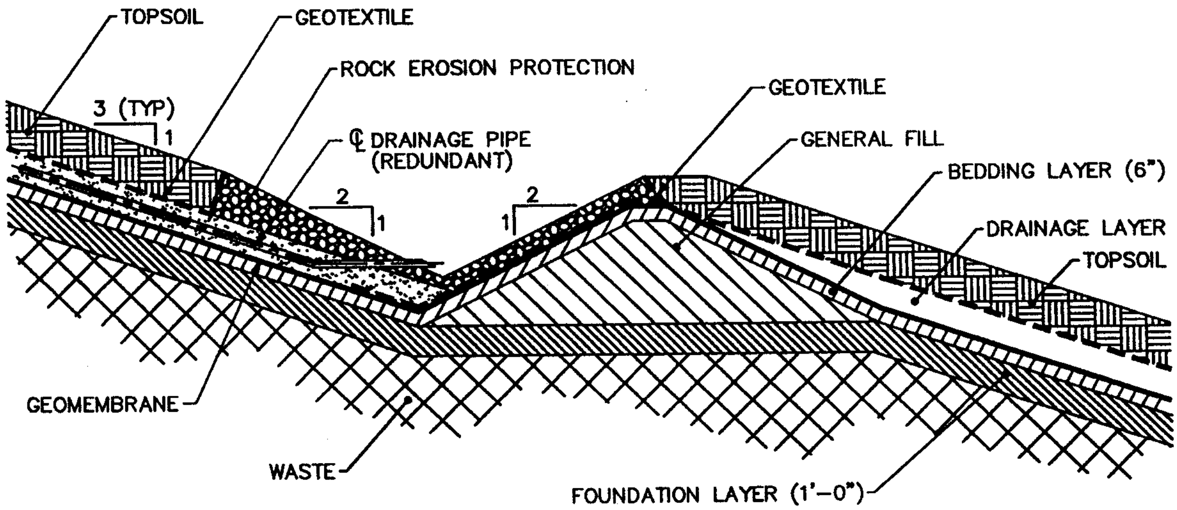


FIGURE 1. TYPICAL LANDFILL COVER SECTION WITH GEOMEMBRANE BARRIER LAYER AND INTERMEDIATE BENCH DITCH

Foundation Soil Overlying the Waste. This layer provides a firm foundation for subsequent cover construction. It is usually installed during landfill operations, or during cover construction when regrading is required. Typically at least 1 foot (30 cm) thick, it consists of general soil material that can be compacted. Some types of waste, such as ash, can even be used. Compaction requirements are not stringent (approximately 90 percent of standard Proctor maximum dry density) because the layer is not expected to support footings for structures. Structures can be constructed on the final cover but require considerations beyond the scope of this paper.

Geomembrane Bedding Layer. This layer typically consists of 6 inches (15 cm) of sand because sand particles have good frictional characteristics, are easy to grade, are of appropriate size for intimate contact with the geomembrane, can convey side slope seeps, and can transmit gas from shallow horizontal collection trenches. Cover design under Subtitle D (EPA, 1992a) will generally require this layer to be part of a composite cover to act as a hydraulic barrier and consist of 18 inches (46 cm) of soil with a hydraulic conductivity no greater than $2x10^{-5}$ feet/minute ($1x10^{-5}$ cm/sec) for most MSW landfills that have bottom geomembrane liners.

Geomembrane. This is the barrier layer that replaces the conventional soil barrier. Key design concerns relating to the geomembrane include interface shear strength with adjacent materials and ability to handle differential settlement. EMCON has found only textured polyethylene products provide the required interface shear strength for steep (3:1) slopes at a reasonable cost. These products, especially the lower density variety, also have three-dimensional strain capabilities that would generally not be exceeded by differential settlements expected on landfill covers.

Drainage Layer. This is typically 1 foot (30 cm) of sand or gravel. Geosynthetic drainage elements, such as a geonet-geotextile composite, are generally not cost competitive with natural drainage materials in the Northwest. Most natural drainage material can be obtained and installed for about \$10 per cubic yard (0.76 m^3) or about \$0.37 per square foot (0.09 m^3) for a 1-foot (30 cm) thick layer. The cost of geocomposite drainage layers is usually significantly more than this, depending on the materials selected. No compaction requirements are specified for this layer.

Geotextile Filter. A filter is generally required between the topsoil and the drainage layer. Sometimes the relative gradations of the drainage and topsoil layers act as a natural filter, precluding the need for an additional filter. In either case the potential for clogging and piping should be checked either by strict filter design criteria, such as those developed by the Corps of Engineers (Cedergren, 1989), or by geotextile filter design methods with site-specific soils, as published in literature (GRI, 1991). If it is not known at the design stage what soil materials will be used, the specifications should require a method for selecting the proper geotextile.

Topsoil. The thickness of this layer depends on the type of vegetation to be established. The layer can be divided into two sublayers: a lower, rooting layer with less organic content and an upper, organic-rich layer. Most designs specify a rooting layer at least 1-foot (30 cm) thick overlain by 6 inches (15 cm) of organic soil. The topsoil layer should be at least as thick as the rooting depth of the proposed vegetation. Although compaction is generally not specified, to avoid overly loose placement the material should be subjected to at least two passes of construction equipment. If permeability is a concern, a minimum compaction can be specified for the rooting layer.

The elements described above are typical of the cover designs used in the Northwest. The following discussions on cover stability and construction reference these elements, but would be applicable to any layered cover section.

LABORATORY TESTING FOR THE COVER DESIGN

Topsoil. Tests required for topsoil include saturated unit weight, gradation, and permeability. Gradation is used to evaluate the filter design between the topsoil and the drainage layer. Looser (more permeable) soil produces more conservative permeability test results. Laboratory sample preparation should replicate as closely as possible the expected field placement condition.

Drainage Layer. This material also requires saturated unit weight, gradation, and permeability testing. Gradation test results should be checked for maximum allowable fines content, gap gradation, and filter compatibility with the topsoil. A dense sample would produce a more conservative permeability test result than a loose sample.

Geotextile. The main criteria for selecting a geotextile, if one is needed, are construction survivability and filtration between the topsoil and drainage layer. The GRI (1991) or Christopher and Holtz (1985) references are recommended as good sources that discuss the appropriate testing and material selection criteria for geotextile filters. The requirement for interface shear strength with adjacent soils is discussed below.

Geomembrane. Geomembrane cover material selection criteria (other than those required by state regulations) include construction survivability, ability to maintain integrity under total and differential landfill settlement conditions, cover slope stability, and cost. Secondary considerations, which could be primary considerations for certain projects, include ease of installation; construction quality assurance requirements; susceptibility and resistance to animal and plant penetration; ability to install in climatic extremes of hot and cold; aging durability; and, if the liner needs solvents for welding, health, safety, and environmental considerations. The testing and specifications required for the project will depend on the project's design considerations. Once a product type is selected, product specifications will generally require a cover geomembrane to meet minimum physical requirements for strength and elongation and provide minimum required interface friction with the adjacent soils (discussed below).

Geosynthetic-Soil Interfaces. Shear strength should be evaluated for the following interfaces:

- Topsoil-geotextile
- Geotextile-drainage material
- Drainage material-geomembrane
- Geomembrane-bedding soil

Typically the critical interface is where the drainage material meets the geomembrane, which should be the only interface where pore pressures may build up. With proper materials selection, the other interfaces should be able to provide adequate stability for slopes up to 33 percent.

The American Society for Testing and Materials is currently preparing a standard for interface direct-shear testing. It would be prudent to perform the testing with a minimum 1-foot (0.09 m^2) square shear box; under saturated conditions; to sandwich the geosynthetic between the actual materials it will experience in the design; and to use the range of normal loads anticipated in the final cover (e.g., 150 to 400 pounds per square foot [7.2 to 19.1 kPa]). An experienced laboratory with proper testing equipment sensitive at these low, normal loads should be used.

STABILITY ANALYSIS

The following procedure is for a potential cover failure sliding parallel to the slope, with the drainage layer-geomembrane interface the primary plane of weakness.

Description of the Problem. The stability analysis theory for a cover on a slope is well documented (Koerner, 1990; Lambe and Whitman, 1969; Giroud and Beech, 1989). It can consist of calculating the force vectors for the entire slope, including toe resistance, or it can be simplified somewhat for a uniform cover section by performing an infinite slope analysis. The method used will depend on the degree of conservatism wanted, its impact on design decisions, and the relative magnitude of the toe resistance compared to overall slope resistance. In many cases with long cover slopes toe resistance accounts for less than 5 percent of slope resistance. If an infinite slope analysis yields a factor of safety (FS) less than desired then the results can be checked by including toe resistance and re-evaluating the FS.

The method discussed in this paper is the infinite slope analysis. The most important parameters to consider in such an analysis are the slope angle, the shear strength of the potential failure plane parallel to the slope, and the amount of water above the potential failure plane (i.e., pore pressures acting on that plane). The unit weights of the materials above the potential failure plane have a small influence on the analysis. The most difficult of the parameters to estimate are the pore pressures acting on the potential failure plane. The potential failure plane of a geomembrane cover is assumed to be the drainage layer-geomembrane interface.

The problem is relevant to slopes steeper than about 15 percent (6.7H:1V), depending on the interface shear strength. Figure 2 depicts the variation in FS with slope angle (β) for a fully saturated infinite slope (worst case condition) assuming an interface friction parameter (ϕ) of 28 degrees (a typical low-end value between textured polyethylene and granular drainage materials). In this case imminent failure (FS=1.0) would occur at a slope of about 22 percent (4.5:1) and FS equal to 1.5 is obtained at a slope of about 15 percent (6.7:1). Landfills in Oregon and Washington are commonly constructed with slopes as steep as 33 percent.

Pore Pressures Acting on the Drainage Layer-Geomembrane Interface. Estimating the amount of head that builds up over the geomembrane requires consideration of the spacing and orientation of underdrain collectors, which collect water from within the drainage layer and discharge it outside the cover section. At a minimum the drainage layer will discharge at the slope toe. The need for additional intermediate discharge locations from the drainage layer is determined by the amount of water it collects and the minimum FS required for stability.

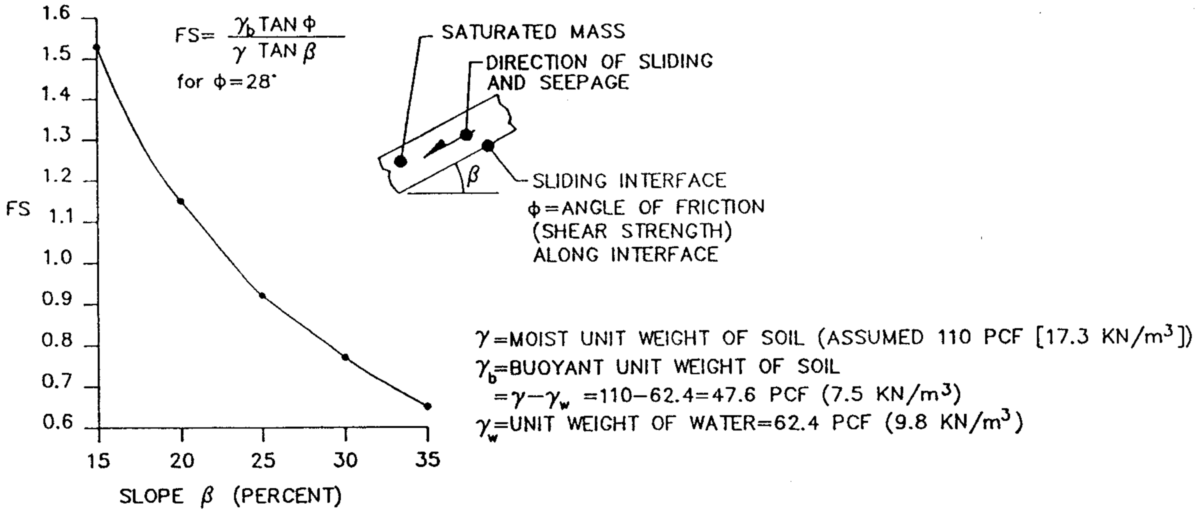


FIGURE 2. SLOPE ANGLE VS. FACTOR OF SAFETY FOR SATURATED INFINITE SLOPE

At least two methods can be used to provide intermediate discharge from the drainage layer: pipes (or collection ditches) running up and down the slope, and pipes (or collection ditches) running subparallel to the slope contours. The latter method seems to be more efficient and has been preferred in Oregon and Washington. The methods of analysis discussed here use this method. Figure 1 shows an example of an intermediate bench ditch that allows discharge from the drainage layer and that controls surface water runoff. The required spacing between drainage outlets is obtained by estimating the head buildup over the geomembrane and performing an infinite slope stability analysis. Head buildup can be estimated as follows (see Figure 3):

1. Assume that the topsoil is saturated during heavy rain, sheet flow is occurring over the surface, and water is percolating through the topsoil at a rate governed by its hydraulic conductivity (k_1). Using a hydraulic gradient (i_1) of 1 over an area (A_1) of unit width and length (L), the percolation into the drainage layer (Q_{in}) from the topsoil is based on Darcy's law as

$$Q_{(in)} = (k_1)\,(i_1)\,(A_1) = (k_1)\,(1)\,(1)\,(L) = (k_1)\,(L) \qquad (1)$$

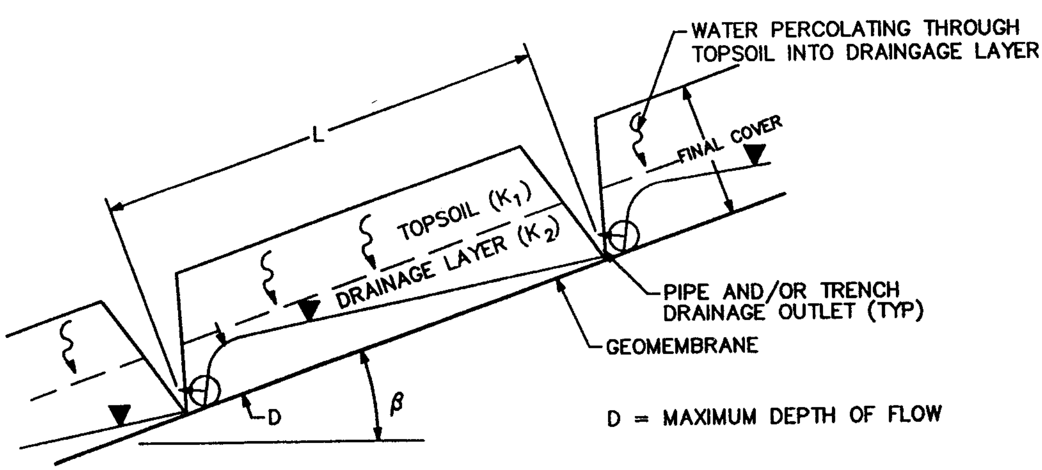


FIGURE 3. SCHEMATIC OF HEAD BUILDUP IN DRAINAGE LAYER

2. Apply the equation of continuity: $Q_{(in)} = Q_{(out)}$. $Q_{(out)}$ is the discharge flow rate in the drainage layer (assumed to flow parallel to the slope) at the downstream (discharge) end of a slope length (L). $Q_{(out)}$ is governed by the hydraulic conductivity of the drainage material (k_2); the gradient (i_2), which is the sine of the slope angle (β); and the area (A_2) normal to the flow equal to a unit width times the flow depth (D).

$$Q_{(out)} = (k_2)\,(i_2)\,(A_2) = (k_2)\sin(\beta)(1)(D) \tag{2}$$

3. Keep the maximum flow depth (D) less than the drainage layer thickness in this model because it is assumed that the topsoil is fully saturated. If the flow in the drainage layer rises to touch the bottom of the topsoil layer, according to this model there will be an incremental jump in pore pressures exerted on the geomembrane from the depth of the drainage layer to the full depth of the cover section. Therefore, by setting (D) equal to the drainage layer thickness, an equation can be written to solve for (L), which immediately shows the importance of estimating the relative permeabilities of the topsoil and drainage layers.

$$L = (k_2)\sin(\beta)(D)/(k_1) \tag{3}$$

This method of estimating head buildup is conservative, though may not be unrealistic, because it does not account for evapotranspiration reducing the amount of water percolating through the topsoil. Some engineers promote the use of HELP (Schroeder, 1989) or other water balance models to estimate percolation into the drainage layer. The conservative model discussed here, however, is preferable for Northwest conditions. During the critical winter months soils are saturated by substantial antecedent rainfall, and plants are not only subject to long periods of total saturation during this time but are either dormant or relatively inactive. It is not difficult to imagine that such percolation conditions modeled above occurred in the Northwest several times during the winters of 1989-90 and 1990-91, both of which had up to 100-year storm events. Only a few minutes of these conditions can initiate a slope failure. It may not be prudent for a water balance model to include evapotranspiration for a condition during several minutes of an intense rainstorm after weeks of antecedent rainfall.

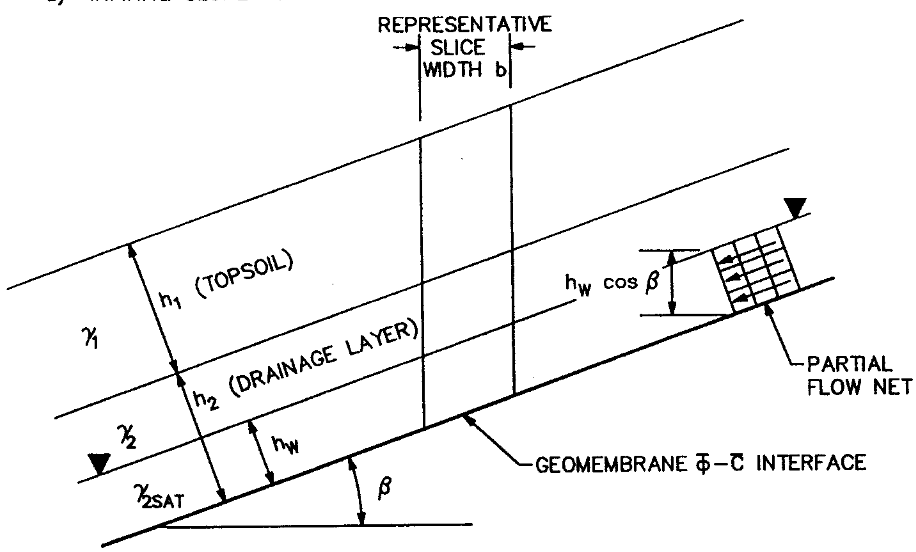
The following results were calculated for a site near Portland, Oregon, to illustrate the discrepancy between the results of the model described above and those of the HELP model to predict the maximum anticipated flow into the drainage layer. A hydraulic conductivity of 2×10^{-5} ft/min (1×10^{-5} cm/sec) was used for the topsoil in both methods.

Water Percolating into Drainage Layer through Topsoil:
- HELP Model: 0.007 inch per hour
- Method described in this paper: 0.14 inch per hour
- Difference: Factor of 20
- Influence on selecting spacing of discharge points: Factor of 20

Although on average the HELP model may produce a good, and even conservative, estimate of the amount of liquid that would percolate annually through the topsoil, it may not estimate the peak amount that would control the stability analysis in an extreme, yet reasonable, situation. Estimating the maximum reasonable amount of water percolation through a topsoil layer is an area for potential future study and refinement.

Infinite Slope Analysis. The general geometry of a partially-saturated slope with seepage parallel to the surface is shown in Figure 4(a). The free-body diagram and force polygon for a vertical slice of the cover soils above the geomembrane are shown in Figure 4(b). Note that the pore pressure uplift force is based on the partial flow net shown in Figure 4(a).

a) INFINITE SLOPE GEOMETRY AND MATERIAL PARAMETERS



b) FORCES

$W = [h_1\ \gamma_1 + (h_2 - h_W)\gamma_2 + h_W\ \gamma_{2SAT}]\ b/\cos\beta$

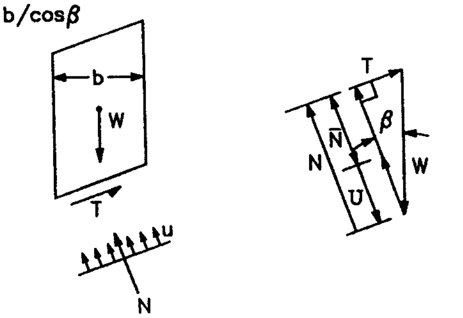
$u = \gamma_W h_W \cos\beta$

$U = ub/\cos\beta = \gamma_W h_W b$

$N = W\cos\beta$

$T = W\sin\beta$

$\bar{N} = N - U$



FREE BODY DIAGRAM FORCE POLYGON

Geometric Parameters: β = slope angle; h_1 = thickness of topsoil; h_2 = thickness of drainage layer; h_w = average height of water in drainage layer normal to slope; b = width of representative slice.

Material Parameters: γ_1 = saturated unit weight of topsoil; γ_2 = moist unit weight of drainage layer; γ_{2SAT} = saturated unit weight of drainage layer; γ_w = unit weight of water; $\bar{\phi}$ = effective friction parameter for shear strength at base of drainage layer; $\bar{c}$ = effective cohesion parameter for shear strength at base of drainage layer.

Forces: u = pore pressure on base of drainage layer; U = uplifting water force; W = total weight of slice; N = total force normal to slope exerted by weight; T = tangential force to slope exerted by weight; $\bar{N}$ = effective normal force.

FIGURE 4. INFINITE SLOPE STABILITY WITH SEEPAGE PARALLEL TO SLOPE
(MODIFIED AFTER DUNN ET AL, 1980, p.241)

The effective stress normal to base of the slice, $\bar{\sigma}$, is

$$\bar{\sigma} = \frac{\bar{N}}{b/\cos\beta} = [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2\,SAT} - h_w\gamma_w]\cos\beta \tag{4}$$

The shear stress exerted tangential to the slice base, τ, is

$$\tau = \frac{T}{b/\cos\beta} = [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT}]\sin\beta \tag{5}$$

The shear strength at base of the slice, S, is

$$S = \bar{c} + \bar{\sigma}\tan\bar{\phi} \tag{6}$$

The factor of safety, defined as the ratio of the resisting shear strength divided by the driving shear stress, is

$$FS = \frac{S}{\tau} = \frac{\bar{c} + [h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT} - h_w\gamma_w]\tan\bar{\phi}\cdot\cos\beta}{[h_1\gamma_1 + (h_2 - h_w)\gamma_2 + h_w\gamma_{2SAT}]\sin\beta} \tag{7}$$

Because the depth of saturation in the drainage layer varies, the FS would vary also. A common procedure is to compute the average FS by using the average water depth in the drainage layer, assumed to be half the maximum water depth (D) used in Equation (2). The method therefore computes an average factor of safety for the slope length between drainage discharge points. Locations upgradient of the average flow depth will have a slightly higher FS, and downgradient locations, a slightly lower FS.

The design methodology would be to compute the FS for a given cover geometry and materials properties using Equation (7), and using one half the maximum water depth (D) used in Equations (2) and (3). If the FS is acceptable, use the maximum drainage discharge spacing (L) computed in Equation (3). If the FS is unacceptably low, reduce the distance (L), recompute the average flow depth (D/2) in the drainage layer, and recompute the FS. Iterate until the FS is acceptable.

Design Example. Given: thickness (h_1) of 1.5 feet (45 cm) of topsoil with a saturated unit weight (γ_1) of 115 pcf (18 KN/m^3) and hydraulic conductivity (k_1) of 2×10^{-4} ft/min (1×10^{-4} cm/sec); thickness (h_2) of 1 foot (30 cm) of drainage layer with moist unit weight (γ_2) of 100 pcf (15.7KN/m^3), saturated unit weight (γ_{2SAT}) of 105 pcf (16.5 KN/m^3) and hydraulic conductivity (k_2) of 0.2 ft/min (0.1 cm/sec); slope angle (β) of 3:1 (18.4 degrees); and interface friction parameter (ϕ) of 30 degrees. Unit weight of water (γ_w) = 62.4 pcf (9.8 KN/m^3).

Find: Maximum allowable spacing (L_{max}) between drainage outlets designed subparallel to slope contours such that the maximum depth (D) of accumulated water in the drainage layer is one foot (30 cm), and a minimum average FS of 1.5 is maintained.

Solution:

$$L_{(max)} = (k_2)\sin(\beta)(D)/(k_1) = 0.2\sin(18.4)(1)/0.0002 = 316\,ft\,(96\,m)$$

$$FS = \frac{[(h_1)(\gamma_1) + (h_2 - D/2)(\gamma_2) + (D/2)(\gamma_{2SAT}) - (D/2)(\gamma_w)]\tan(\phi)}{[(h_1)(\gamma_1) + (h_2 - D/2)(\gamma_2) + (D/2)(\gamma_{2SAT})]\tan(\beta)} \tag{8}$$

$$= \frac{[(1.5)(115) + (1-.5)(100) + (.5)(105) - (.5)(62.4)]\tan(30)}{[(1.5)(115) + (1-.5)(100) + (.5)(105)]\tan(18.43)} = 1.5\ (ok)$$

Factor of Safety. Geotechnical engineers often feel comfortable with a minimum FS of 1.5 for long-term static slope stability conditions. This value originated from dam designs and may or may not be applicable to landfill slope stability in general, or cover stability in particular. The EPA (1992b) suggests that a FS between 1.25 and 1.5 might be acceptable depending on the level of certainty in the shear strength parameters. The following factors should be considered when selecting a minimum FS for a landfill cover: experience with geosynthetics has a relatively short history; level of confidence in assumed material properties; factors of safety may already be included in the estimated material properties; and the situation being analyzed is considered a worst case that will occur a small percentage of the time. A minimum FS of 1.5 has been used without failures attributable to underdesign of the drainage layer in the Northwest.

Seismic Considerations. New Subtitle D regulations require checking all elements of landfill design for stability due to seismic loading. A Newmark-type analysis (Newmark, 1965) is recommended for the cover. Consideration should be given to how much saturation should be allowed for in the drainage layer for the analysis (using the maximum amount computed for static stability would probably be overly conservative), and what amount of deformation would be acceptable. Because cover deformation is not life-threatening, and because it could be inspected and repaired after the event, the acceptable amount should be somewhat higher than the amount for a bottom liner. Acceptable deformations for bottom liners have been cited as 6 to 12 inches (15 to 30 cm) for a conservatively selected design earthquake (Seed and Bonaparte, 1992). One approach is to use these same criteria for covers modeled in a dry condition because the chance of an earthquake occurring at the same time as the maximum conservative saturation condition discussed in this paper is much more remote than the earthquake alone. Another approach is to recognize that the cover system does not directly affect the ability of a lined landfill to contain waste, and that it can be inspected and repaired. Therefore, the cover could be allowed to sustain a higher level of deformation, say on the order of one to three feet (30 to 90 cm). Often a design that is satisfactory for the worst case static condition with a reasonable FS is acceptable for a seismic condition.

CONSTRUCTION ISSUES AND PRODUCTION

The following paragraphs describe construction issues related to major elements of a final cover section, surface water ditches, and geomembrane penetrations. Anticipated production rates for constructing major elements of a cover are also provided for planning and scheduling purposes.

Settlement Considerations. Design changes on landfill closures are nearly unavoidable, mainly because landfills settle. Landfill owners and operators, however, expect the designer to achieve the lowest possible construction costs by preparing such complete bid packages that contractor claims and change orders during construction are minimized. Nevertheless, design changes to account for settlement must be expected, planned for, and executed during the construction phase of the project.

Design elements particularly susceptible to settlement are the horizontal and vertical control for roads, ditches and pipelines. For example, the design of both the

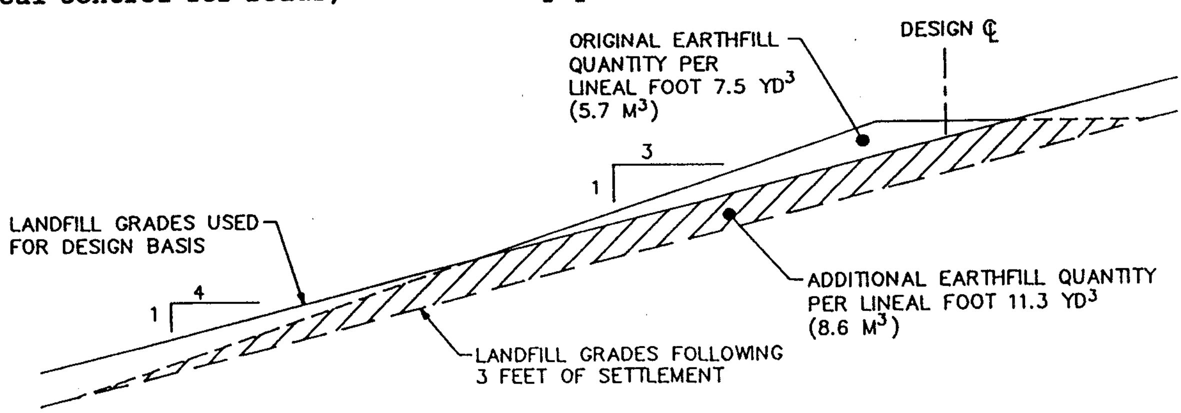


FIGURE 5. EFFECT OF SETTLEMENT ON ROAD DESIGN WITH SET HORIZONTAL AND VERTICAL CONTROL

horizontal and vertical alignment of a road for the side hill of a landfill was based on topographic mapping prepared 12 months before construction. Figure 5 shows the effect of 3 feet of settlement on an actual road section designed on a 4:1 side slope. Accommodating this settlement required either increasing the material to construct the road to design grades and alignments or rapidly redesigning the horizontal and vertical alignment during construction. The increased quantity of soil required to build the road to design grades was 11.3 cubic yards ($8.6m^3$) per lineal foot (0.3m) at a potential additional cost to the client of $158 per lineal foot. This would have added approximately $450,000 to the total project cost. Redesigning the road was the less expensive option, even if the contractor had put in a claim for delay.

Such design problems can be avoided. Instead of specifying set horizontal and vertical control in the bid documents, it would be better to design a typical road cross section relative to the final cover, set a general horizontal alignment, and require minimum cross slopes and longitudinal grades. Although more field engineering is involved, settlement makes this extra work inevitable. Final alignment of items that can be affected by settlement should not be fixed until the construction phase of the project.

<u>Foundation Soil.</u> A critical element in planning foundation layer construction is estimating the quantity of soil required to meet the design intent. Often landfill surfaces are poorly graded, have refuse exposed in areas, and have areas requiring significant earthfills to meet minimum final slope requirements. Assuming an average depth of foundation soil based on a typical cover cross section usually results in underestimating the quantity required to complete the work.

Experience has shown that for poorly graded landfills, where the refuse was recently placed and where daily cover is minimal, the quantity of soil required to provide a uniform grade and a 1-foot (30 cm) minimum depth of soil cover is 2 to 3 cubic feet (0.06 to $0.08m^3$) of soil per square foot ($0.09m^2$) of landfill area.

Production rates for placing foundation layer soil depend on the source of the material, available equipment, and site access. If soil is available on-site near the construction area daily quantities of 5,000 to 10,000 cubic yards (3,800 to $7,650m^3$) per day can be achieved. If the soil source is from off-site, a large trucking fleet will be required and production of more than 6,000 cubic yards ($4,600m^3$) per day is difficult to achieve.

<u>Bedding Soil.</u> The critical element in placing bedding layer soil is preparing a well graded subgrade (top of foundation soil). A well graded subgrade will minimize the quantity of bedding soil required to meet minimum thickness requirements. A rough graded subgrade with holes or ridges will require higher quantities of bedding soil, raise costs, and slow the schedule. Closely spaced grade control is also a key to constructing a quality uniform-depth bedding layer. Spacing grade control on a 50-foot (15m) grid is common.

Bedding soil, particularly where it is used to transmit gas or leachate, is typically a processed and imported soil. Production rates of 1,500 to 2,000 cubic yards (1,150 to $1,500m^3$) per day can be achieved which result in 2 to 3 acres (0.8 to 1.2 hectares) of completed bedding layer per day.

<u>Geomembrane.</u> Critical elements for geomembrane installation are maintenance of the bedding layer grades during deployment, quality seaming, and protection of the geomembrane during placement of the drainage layer.

Equipment and foot traffic can rut and put holes in a sandy bedding layer. Repair of this disrupted grade is required for a quality geomembrane installation. A smooth foundation for geomembrane installation is also critical to seaming operations and for avoiding undue stress on the geomembrane materials.

A large amount of information has been written concerning obtaining quality geomembrane seams. It is not the intent of this paper to describe geomembrane seam construction in detail (see, for example, EPA, 1989). However, in summary, the following should be considered:

- Qualified personnel should perform the work.
- Welding equipment should be well maintained and checked frequently to assure proper operation. This is particularly important for the heating elements of welding machines.
- The installer should have a well planned quality control plan and the Owner should provide on-site representation to assure compliance with the plan and provide quality assurance testing.
- Test equipment should be well maintained, properly calibrated, and meet the requirements of the test procedures.

Production rates for geomembrane installation vary significantly based on complexity of projects, project size and schedule. Installations of 100,000 square feet (9,300m^2) per day for polyethylene are not uncommon. For planning purposes, however, production rates of 30,000 to 40,000 square feet (2,800 to 3,700m^2) per day are reasonable.

<u>Drainage Layer.</u> The drainage layer component of a final cover, particularly for steep slopes, is the most difficult to construct and the most important for the design. Many things can go wrong and planning to prevent problems is essential to success.

The drainage material must meet gradation and permeability requirements. Discovering that a product does not meet these requirements following its placement on the geomembrane can be costly. Removing material from the top of geomembrane is much more difficult than placing it.

A good quality control plan can avoid this problem by verifying material compliance with specifications before the material is placed. This may include source sampling and testing before approval of the material is granted. Monitoring delivery of material to the site is also important. Providing on-site testing capabilities can speed up the quality assurance monitoring of the material.

Drainage material placement should be monitored closely to assure no damage is done to the geomembrane, wrinkles are controlled, and proper thickness is achieved. Low ground pressure dozers work well to place this material but often require long pushes from the point where the material is delivered.

Drainage material placement is often the critical path in scheduling a cover project. The time required to produce the material, available trucking for delivery, and equipment necessary to handle the material on-site all need to be considered. Production rates of 3,000 to 4,500 cubic yards (2,300 to 3,400m^3) per day can be achieved, but logistical plans for this production rate must take into account that this means about 250 truck loads of material will be delivered per day.

<u>Geotextile.</u> Installation of geotextile is straight forward. Items to consider are protection from ultraviolet degradation and seaming of the product in the field. Production rates to meet schedules are usually not a problem.

<u>Topsoil.</u> Topsoil is placed at the end of a project, and the key to its success is constructing it in time to seed it, get vegetation to germinate, and protecting top soil from erosion. Simple methods of erosion protection that have proven effective are track walking a slope so that dozer tracks are deep and perpendicular to the line of slope, and covering the completed soil layer with 2 to 4 inches (5 to 10 cm) of straw. In colder climates this helps insulate seed and provides additional erosion protection. More sophisticated erosion control methods are available if needed. Production rates are usually between those for foundation soil and drainage layer.

Ditches. The most common construction error for surface water ditches is not paying attention to the cross sectional details of the ditch. The ditch in Figure 1 is a good example. In this ditch the design water depth is the highest elevation of geomembrane. If during construction the geomembrane is not brought up on the downstream side of the ditch and the contractor decides to maintain the overall depth of ditch with soil, then the soil above the geomembrane on the downstream side of the ditch must contain the water. If this soil is permeable or erodible the ditch can fail at high water levels, and damage the downslope cover. Protection against this problem is a good design, quality ditch installation, good quality control, and detailed construction staking and construction techniques.

Geomembrane Penetration. Two common problems occur with penetrations through geomembrane covers: (1) stresses are put on the geomembrane boot during drainage layer construction that result in boot failures or failures of the pipe penetrating the geomembrane, and (2) landfill gas escapes.

Figure 6 shows a detail that has been constructed successfully on cover projects that prevents these problems. The key to this detail is not the detail itself, but the timing of installation. The geomembrane boot should not be welded to the geomembrane cover until the major portion of drainage layer is placed. Geomembrane covers move during drainage layer placement, particularly on slopes. If the boot is installed prior to drainage layer placement then movement of the geomembrane transfers stresses to the area of the rigid pipe penetration. Landfill gas losses through the boot are prevented by the hydrated bentonite shown on the detail.

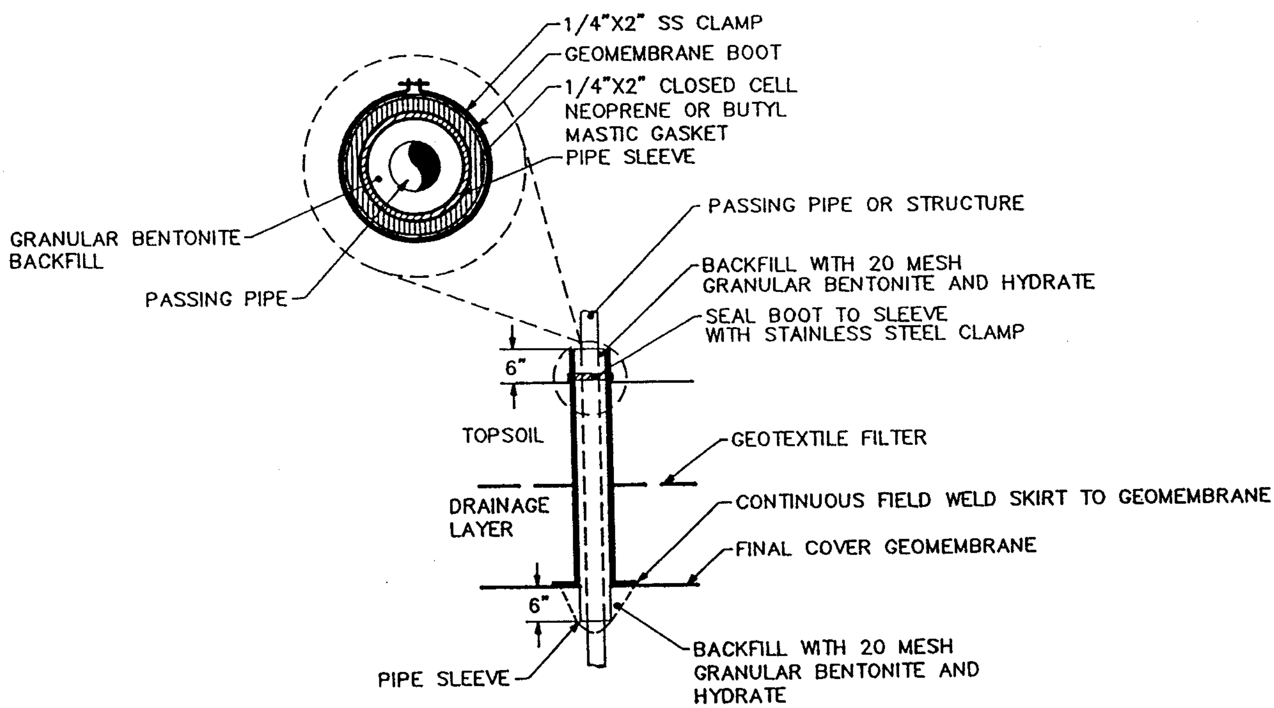


FIGURE 6. GEOMEMBRANE BOOT PENETRATION

Overall Production Rate. Overall construction time for a typical 25-acre (10-hectare) project is on the order of 3 to 4 months, although projects up to twice this size have been constructed in the same amount of time. Key factors affecting the schedule are the amount of equipment and manpower used by the contractor, efficient scheduling and construction management, well prepared drawings and specifications, and cooperation between the owner, engineer, and contractor.

SUMMARY AND DISCUSSION

1. Using geomembranes for landfill covers has increased in recent years because they perform better and are often more cost-effective than soil-only covers. The new Subtitle D regulations will likely require geomembrane final covers on most new landfills that have bottom geomembrane liners. Engineers need to develop an understanding and methodology for designing such landfill covers.

2. Two important considerations in designing geomembrane covers are the interrelated issues of slope stability and drainage. The infinite slope method is appropriate for modeling cover slope stability. Applying Darcy's law to an assumed saturated topsoil condition is a conservative yet rational method for estimating maximum head buildup in the drainage layer above the geomembrane for use in the infinite slope analysis.

3. The actual materials that will be used for each project should be tested to obtain design parameters. In cases where material selection is left to the contractor, provisions should be made in the specifications to address minimum or maximum allowable material properties and conformance testing so that the design intent is met.

4. Construction project planning should include provisions for field engineering modifications to allow for landfill settlement.

5. Construction quality control and quality assurance monitoring is important for

- Soil material conformance testing
- Geosynthetic material conformance testing and installation observation
- Grade tolerances and layer thickness control
- Construction details such as ditch cross sections and pipe boot installation

6. Construction planning and sequencing can improve efficiency and allow realistic scheduling.

REFERENCES

Cedergren, H.R., (1989) "Seepage, Drainage, and Flow Nets", 3rd Ed., John Wiley and Sons, New York.

Christopher, B.R. and Holtz, R.D. (1985) "Geotextile Engineering Manual", Federal Highway Administration, FHWA Contract No. DTFH61-83-C-00150.

Dunn, I.S., Anderson, L.R., and Kiefer, F.W., (1980) "Fundamentals of Geotechnical Analysis", John Wiley and Sons, New York.

EPA, (1989), "Technical Guidance Document: The Fabrication of Polyethylene FML Field Seams", EPA/530/SW-89/069, September 1989.

EPA, (1992a) "Final Cover Requirements for Municipal Solid Waste Landfills", Environmental Fact Sheet, EPA/530/SW-91084, March 1992.

EPA, (1992b) "Draft Technical Manual for Solid Waste Disposal Facility Criteria - 40 CFR Part 258", April 1992.

Federal Register, (1991) "Solid Waste Disposal Facility Criteria; Final Rule", 40 CFR257 and 258, Part II Environmental Protection Agency, Vol. 56, No. 196, Wednesday October 9, 1991 Rules and Regulations, U.S. Department of Commerce, Washington, D.C.

Giroud, J.P. and Beech, J.F., (1989) "Stability of Soil Layers on Geosynthetic Lining Systems", Geosynthetics '89 Conference Proceedings, Vol. 1, pp. 35-46.

GRI, (1991) "Proceedings of the 5th GRI Seminar on Geosynthetics in Filtration, Drainage, and Erosion Control", Geosynthetics Research Institute, Drexel University, Philadelphia, December 12 and 13, 1991.

Koerner, R.M., (1990) "Designing With Geosynthetics", Prentice Hall, New Jersey.

Koerner, R.M., and Daniel, D.E., (1992) "Better Cover-Ups", Civil Engineering, May, 1992, pp. 55-57.

Lambe, W.T. and Whitman, R.V., (1969) "Soil Mechanics", John Wiley and Sons, New York.

Newmark, N.M, (1965) "Effects of Earthquakes on Dams and Embankments", Geotechnique, Vol. 5, No. 2, London, England, June, 1965.

Seed, R.B. and Bonaparte, R., (1992), "Seismic Analysis and Design of Lined Waste Fills: Current Practice", Stability and Performance of Slopes and Embankments - II, ASCE Geotechnical Special Publication No. 31, New York, NY.

Schroeder, P.R. (1989), "The Hydrologic Evaluation of Landfill Performance (HELP) Model" U.S. EPA, Cincinnati, Ohio.