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GEOSYNTHETIC FUNDAMENTALS IN LANDFILL DESIGN

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ABSTRACT: Contemporary design of lined landfills in the United States began in the early 1980's with Federal regulations focused on hazardous waste disposal. This regulatory framework was expanded to include municipal solid waste in the early 1990's. With nearly 30-years experience in the design of lined landfills, simple design practices have evolved to ensure the successful performance of lined landfills. Geosynthetic components play a critical role in the two main components of these landfills: (1) liners to limit the vertical migration of liquids, and (2) drainage field that collect the liquids and limit the potential for leaks to develop. Geomembrane liners provide a very stable barrier to the spread of liquids but must be built to very high standards, be protected from damage during construction and their service life, and come with the potential for stability problems generated by their slick surfaces. Geosynthetic drainage composites provide a means of draining the collected liquid from large areas of the liner surface. Properly designed, such geosynthetic drainage layers will add to the stability of the landfill by limiting hydrostatic forces due to the collected liquids. Past failures of lined landfills provide an effective means of demonstrating the inevitable consequences of not meeting fundamental design requirements for these components.

KEYWORDS: geosynthetics, landfill design, liners, drainage composites

INTRODUCTION

Over the past 25 years, design procedures for the liner and liquid collection systems have evolved with the development of both improved design methodology and geosynthetic components, and in response to observed failures in the field. A review of the more than 40 failure investigations performed by the authors indicates that failures can be minimized if design and construction fundamentals are observed. These fundamentals do not address the economics of the landfill or additional appurtenances that are also required. However, the fundamentals do protect the designer and owner from expensive failures.

Contemporary lined landfills rely on two key systems to control the migration of leachate draining from the waste: a low permeability liner system that limits the vertical and lateral movement of the leachate and a liquid collection/removal system (LCRS) to collect and remove the leachate that accumulates on the liner system. The design and construction considerations for these two systems are inherently very different and must be understood if landfill is to be a success.

An additional consideration not addressed in this paper is the need to operate the landfill in a manner consistent with its design. The operations manual prepared by the designers must provide the owner with clear guidance on field operational practices that must be followed to ensure failure free operation. All contemporary landfills can be failed by field operations that are inconsistent with the design assumptions.

COMPOSITE LINER SYSTEM PERFORMANCE

Modern landfills rely on a composite liner system that depends on a synergistic relationship between the geomembrane liner (GM) and an underlying soil liner. A better understanding of the important design and construction considerations for these two components is obtained by examining the theoretical basis for estimating leakage through the composite liner. Empirical modeling and field observations (Giroud and Badu-Tweneboah 1992) have resulted in the "*Giroud*" equation for estimating leakage through a hole in the geomembrane portion of a composite liner. The empirical equation takes the form of :

good contact

$$Q = 0.21 h^{0.9} a^{0.1} k^{0.74} \tag{1}$$

for poor contact

$$Q = 1.15 h^{0.9} a^{0.1} k^{0.74} \tag{2}$$

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where "contact" refers to the contact between the soil liner and the geomembrane, e.g. no wrinkles in the liner and a properly smoothed clay surface contribute to a good contact, Q = rate of leakage through a defect (m^3/s); h = head of liquid on top of the geomembrane (m); t = thickness of the soil component of the composite

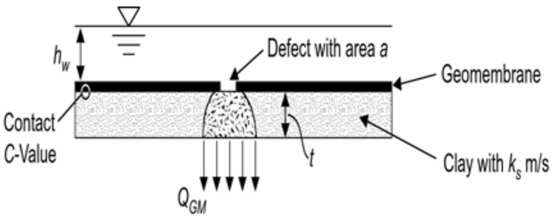

Figure 1 Composite Liner Variables

liner (m); a = area of defect in geomembrane (m^2); and k = hydraulic conductivity of the underlying clay liner (m/s). Equations (1) and (2) are incorporated into the latest versions of the HELP computer model (U.S. EPA 1994) used for predicting landfill leachate generation and leakage.

Leakage through a composite liner system **increases** with the following:

1. Increasing head, h, of leachate,
2. Decreasing soil liner thickness, t,
3. Increasing soil liner permeability, k,
4. Increasing area of defect in GM, a, and
5. Decreasing lack of good contact between the two liner components.

Item 1 is the focus of the design of the LCRS system, Items 2 and 3 are typically specified by the regulatory agency, and Items 4 and 5 are minimized using a Construction Quality Assurance program (CQA). A discussion of CQA programs is beyond the scope of this paper but readers are directed to Koerner and Daniels (1993), and Daniels and Koerner (2007).

Equations (1) and (2) are generally not used in the actual design of a lined landfill but does provide a means of recognizing key factors that influence liner leakage. The limitations of equation 1 relate to the uncertainty as to the number of actual defects in the liner. With a comprehensive CQA program as outlined above, it is estimated that 1 to 3 $1cm^2$ defects remain per 4000 m^2 area. This allows a rough estimate of potential leakage. Additionally, equations (1) and (2) are frequently used to evaluate problems with excessive leakage in existing lined landfills. This means that Equations (1) and (2) should be considered an excellent forensic tool and not a design tool.

GEOMEMBRANE LINER DESIGN

While the hydraulic performance of the GM requires a field CQA program to minimize defects, the stability of the GM must be verified in the design. The typical GM is formed of a thermo-plastic that has a very low inherent interface shear strength, e.g. is very slick. Before the stability of the liner system can be calculated, the interface shear strength between each of the layers forming the liner and collection system must be evaluated.

Liner Interface Strength Testing

Within the United States, the interface shear strength is measured using the direct shear test (ASTM-D5321) on the interfaces of concern under project specific conditions. This test must be performed on each interface of the liner and LCRS system that will be placed beneath the waste. Typically, the controlling interface, i.e. having the lowest strength, is one of the two interfaces related to the geomembrane, i.e. GM to soil or GM to LCRS.

Figure 2 show the results of a D5321 direct shear test to determine the interface shear strength between a textured GM and geosynthetic drainage composite, e.g. textured GM against a nonwoven geotextile. The test

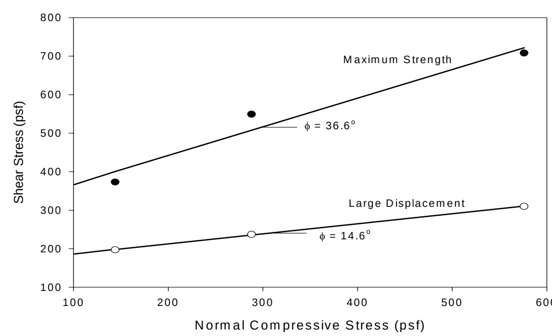

Figure 2 ASTM D5321 Direct Shear Test Results (1kPa = 20.5psf)

was performed at three normal loads typical of what would be anticipated for a GM in a landfill final cover system. The maximum strength data represents the peak shear strength at displacements typically less that 12.5mm. The large displacement data reflects shear strengths that occur at displacements greater that 25mm. The large displacement strengths are typically lower due to abrasion damage to the texturing. The large displacement strengths are commonly used in seismic stability evaluations where the static factor of safety can drop below 1.0 during the event.

As shown on Figure 2, the measured shear strength, τ, is typically represented by the Mohr-Coulomb equation

$$t = c + \sigma \tan \delta \quad (3)$$

where c is apparent adhesion, σ is the normal pressure on the interface, and δ is the interface friction angle. For liners placed on slopes, c should be a minimum of 2.5 kPa based on stability considerations during construction. The normal loads used in the ASTM D5321 direct shear test should be representative of the normal loads anticipated during the service life of the liner system.

Liner Stability Analysis

One of the greatest challenges facing the design engineer is to accurately estimate the density and shear strength of the waste and geometry of future waste placement. All of these parameters are beyond the direct control of the design engineer but must be conservatively estimated or failure can result.

Waste Properties

The density of waste can be estimated from historical data but can be impacted by local waste characteristics. Lacking local data on waste density, the authors recommend the use of densities presented on Table 1. Waste densities are obtained using annual surveys to establish waste volumes and truck scales to record the actual weight of waste received each year. The weight of waste must be increased to account for the daily cover soils applied to the waste to control vectors.

Table 1 Default Waste Total Unit Weights

Waste Type	Total Unit Weight, kN/m^3
Municipal Solid Waste	8-10
Construction/Demolition	6.5-9
Industrial	6-13
Coal/Fly Ash	9-12

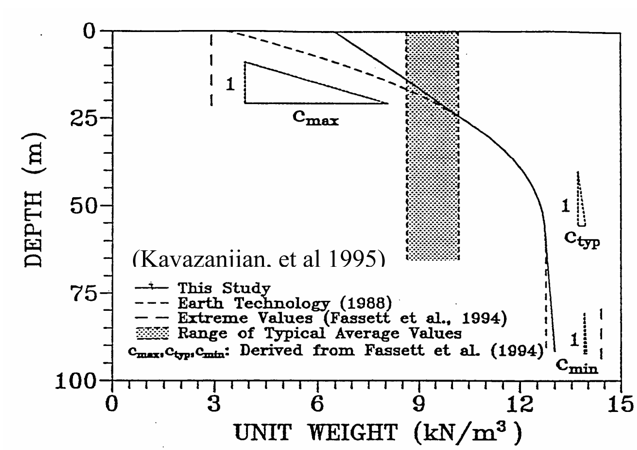

Figure 3 Waste Unit Weight Increase With Depth

Note that waste densities generally increase with increasing normal loads so that the upper limits of density should be assumed for wastes having a height greater than approximately 45m. Figure 3 shows this increase in density with depth for typical MSW (Kavazanjian,et.al, 1995).

Strength properties of common wastes are impossible to evaluate using typical geotechnical laboratory soil testing procedures due to the very large size gradation of the waste. What the design engineer must typically rely on is empirical strength values based on historic observation of both successful and failed waste placement conditions. Figure 4 shows rotational stability failure of waste that allowed back calculation of the average shear strength of the waste at the time failure occurred. The nearly 30-m vertical faces of the waste require the waste to have an apparent cohesion even though the material is essentially non-plastic. For

Figure 4 Landfill Waste Stability Failure

municipal solid waste, Figure 5 shows a compilation of back-calculated shear strengths with a recommended design shear strength envelop for MSW (Kavazanjian, et.al, 1995).

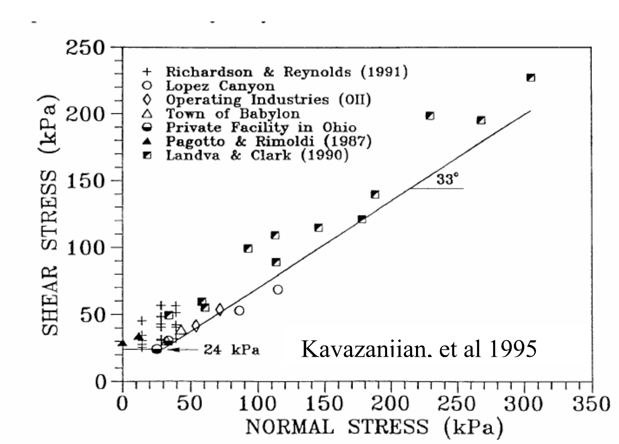

Figure 5 Shear Strength of MSW Waste

Waste Geometry

The future geometry of the waste is beyond the direct control of the design engineer but can be influenced if the designer prepares an operational manual for the landfill that provides waste placement guidance to the operator. The design engineer must understand that all lined landfills can be failed from the improper placement of waste within the landfill. Thus, a properly prepared operations manual protects both the design engineer and the owner.

Correct placement of waste is typically controlled by specifying a maximum lift thickness and exterior slope for the lift. Stability can be improved if the operations manual requires full placement of each lift before subsequent lifts are begun. Additionally, the operations manual should provide the operator guidance on interim features that can influence localized stability such as temporary roads for truck access during placement of each lift.

Numerical Analysis

Once the projected waste properties and geometry are defined, the stability analysis is performed using one of many slope stability programs commonly used by geotechnical engineers. The only real limitation on software selection is the need for the program to allow block type failures since the failure surface will follow much of the liner surface. Geotechnical programs that rely on circular failure surfaces cannot properly model the actual failure surface that will commonly along and rarely cross a portion of the liner surface. Figure 6 shows the results of a typical waste stability evaluation. This evaluation was performed with the software program STABL.

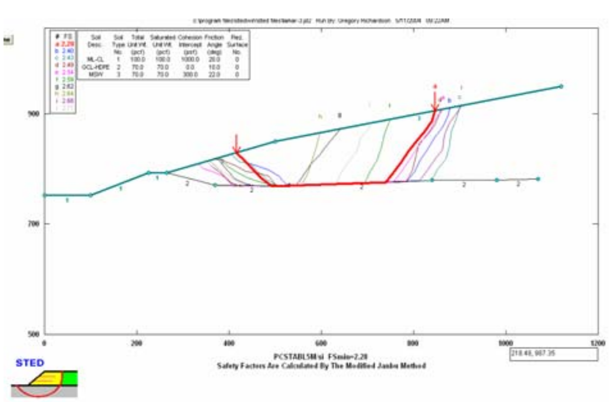

Figure 6 Slope Stability Evaluation of Landfill

LEACHATE COLLECTION SYSTEM DESIGN

Recalling Equations (1) and (2), the impact of a defect in the liner system is minimized by maintaining a low head, h, of leachate acting on the liner system. In addition to minimizing leakage, the leachate head must be minimized to ensure stability of the waste. In the USA, the maximum head of leachate acting on a liner system is limited to 30-cm.

For the above reasons, the liners system must be covered by a collector system that can withstand the normal loads generated by the weight of the waste and potential clogging due to both the suspended solids carried by the leachate and the biological clogging potential from the micro-organisms also carried by the leachate. The manner in which the landfill is operated will determine the potential for clogging of the leachate collection systems with landfills. Recirculation of leachate back into the waste has the greatest potential for clogging, thus demand an enhanced leachate collection system.

The LCS consists of two major systems: (1) an area collection system that covers the surface of the liner, and (2) a piping system that in turn drains the area collection system. The area collector is the primary mechanism for satisfying the regulatory limit on head acting on the liner while the piping system is responsible for draining the area system and directing the leachate to a sump for removal from the landfill.

Area Leachate Collection System

Area collection leachate collection systems were originally envisioned as sand layers placed immediately above the liner systems. In recent years, the sand layer is commonly replaced with a geocomposite drainage layer that consists of a geonet with a nonwoven geotextile bonded to one or both faces of the geonet. The variables in the analysis are shown on Figure 7 below and include the impingement rate, q_h, at which leachate is draining from the waste, the liner slope β, and the slope base length L.

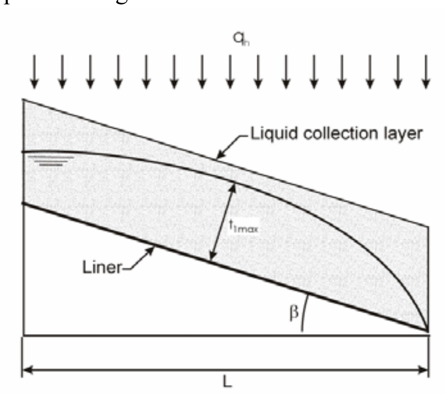

Figure 7 Leachate Mounding Analysis

A simplified but accurate analysis for the maximum head in the above analysis was developed by Giroud et al. (1992) and is discussed in Giroud et al. (2000). The maximum head on the liner, termed t_{max} by Giroud, is given by the following equation:

$$t_{max} = j \frac{\sqrt{\tan^2 \beta + 4\, q_h / k} - \tan \beta}{2 \cos \beta} L \qquad (4)$$

where k is the permeability of the lateral drain. The j term is a numerical modifying factor whose value is always slightly less than 1 such that it can be neglected in the typical landfill analysis. Generally, the uncertainty regarding the impingement rate q_h does not justify the use of the j factor.

The total flow rate through the collecter, Q, is equal to $q_h L$ per unit width of the drainage layer. For lateral drains constructed using geocomposite drainage layers, Giroud (2000) has shown that Eq. (4) can be simplified due to the small flow thickness in drainage composites to the following:

$$t_{\max} = \frac{q_h L}{k \sin \beta} \qquad (5)$$

This greatly simplified equation is appropriate when the thickness of the geocomposite drainage layer is less than 20 mm over the range of slopes common to most landfills. This equation should not be used with thicker natural drainage layers. This allows the required transmissivity for a geocomposite drainage layer to be directly solved for as follows

$$\theta_{required} = \frac{q_h L}{\sin \beta} \qquad (6)$$

Solutions based on Giroud's numerical solution will be conservative and less than 5% in error. Again, the simplified solution in Equations (6) is applicable to geocomposite drainage systems only.

Published values of typical leachate generation values for non-arid regions show typical leachate generation rates of 11,000-20,000 liters/hector/day or a design rate of liquid supply of 1.3-2.3 x 10^{-6} cm/sec during active placement of waste. This reduces to 2000-5600 liters/hector/day once a 30cm interim cover is placed, and 500-1500 liters/hector/day after placement of the final cover. The accuracy of these values was confirmed through discussions with solid waste regulators in the USA. Leachate generation rates will be dramatically impacted by the degree of leachate/storm water separation that is inherent in the facilities design and operational practices. The leachate generation rates referenced above reflect typical design and operational efforts. The 10^{-6} cm/sec design rate of liquid supply approximately represents the infiltration into a leachate collection system through a silty/clayey protective soil layer during initial waste placement. Facilities that employ supplemental 'rain sheets' may have less leachate, while those using porous operational covers will have significantly more. The design is obviously conservative for long-term flows if it is designed properly for the short-term operational flows.

Long term performance

The long term performance of a lateral drain requires a larger allowed transmissivity, $\theta_{allowed}$, than that obtained from the design equations, $\theta_{req'd}$. This process was initially quantified by Koerner (1998) as follows:

$$FS_{dc} = \frac{\theta_{allowed}}{\theta_{req'd}} \qquad (7)$$

where FS_{dc} is the overall safety factor for drainage, $\theta_{req'd}$ is the required transmissivity based on Equation (6), and $\theta_{allowed}$, is the allowable transmissivity being determined under simulated condition for 100-hour duration using the following formula per GRI-GC8 standard (2001)

$$\theta_{allow} = \theta_{100} \frac{1}{RF_{CR} \times RF_{CC} \times RF_{BC}} \qquad (8)$$

where

$\theta_{allowed}$ = allowable transmisisivity
θ_{100} = laboratory measured transmissivity determined under simulated conditions for 100-hour duration
RF_{CR} = reduction factor for creep to account for long term behavior
RF_{CC} = reduction factor for chemical clogging
RF_{BC} = reduction factor for biological clogging

The creep reduction factor RF_{CR} is based on 10,000 hour compressive creep data and calculated according to the following equation developed by Giroud et. al (2000) where:

$$RF_{CR} = \left[\frac{\left(t_{CO} / t_{virgin}\right) - (1 - n_{virgin})}{\left(t_{CR} / t_{virgin}\right) - (1 - n_{virgin})} \right]^3 = \left[\frac{t_{CO} - \frac{\mu}{\rho}}{t_{CR} - \frac{\mu}{\rho}} \right]^3 \qquad (9)$$

t_{CO} = thickness after load application for 100hours
t_{virgin} = initial thickness
t_{CR} = thickness at the time period of interest (for instance, thickness at 50 year design life, extrapolated from the 10,000 creep curve)
n_{virgin} = initial porosity
μ = mass per unit area of the considered geonet

ρ = density of the polymeric compound used to make the geonet

Range of clogging reduction factors is provided by GRI-GC8. Combining equations (6), (7) and (8), a drainage safety factor, FS_{dc}, of the geocomposite drainage layer can then be calculated as follows:

$$FS_{dc} = \theta_{100} \times \frac{1}{RF_{CR} \times RF_{CC} \times RF_{BC}} \times \frac{\sin\beta}{q_h \times L} \quad (10)$$

The selection of drainage FS-value is dependent upon the design life and criticality of the project, 2 – 3 is recommended by Giroud et al (2000). The combination of drainage safety factor and reduction factors are sometimes called the Long-Term Services Factor.

Transmissivity testing

The in-plane flow capacity of a geocomposite is evaluated using a laboratory transmissivity test (ASTM D-4716). This test is performed using the transmissivity box setup shown on Figure 8. This apparatus allows a range of normal loads and boundary conditions, i.e., soil or geomembrane, to be applied to the face of the geocomposite. The head acting across the 300mm square sample can be varied to create a range of gradients that simulate field slope conditions. The flow gradient, i, is defined as the head divided by the flow length (300mm in the case of ASTM D4716).

Figure 9 shows typical results from a laboratory transmissivity test. In general, transmissivity decreases with increasing normal loads and increases with decreasing flow gradients. Great care must be taken to properly specify the proper boundary conditions and load duration for this test. Boundary conditions on both the upper and lower faces of the drainage composite should reflect the actual material that will be encountered in the field. This allows a proper simulation of intrusion that may occur due to soft soils or large particles in the soils. The flow gradient used in the test must be equal to or larger than the actual anticipated flow gradient. The normal load should be allowed to seat for a time period of not less than 100 hours before performing the flow test. Additionally, the manufacturer of the geocomposite drainage product should provide compressive creep test data indicating the performance of the specific product with a load duration of 10,000 hours. This effectively precludes the long-term collapse of the geonet core.

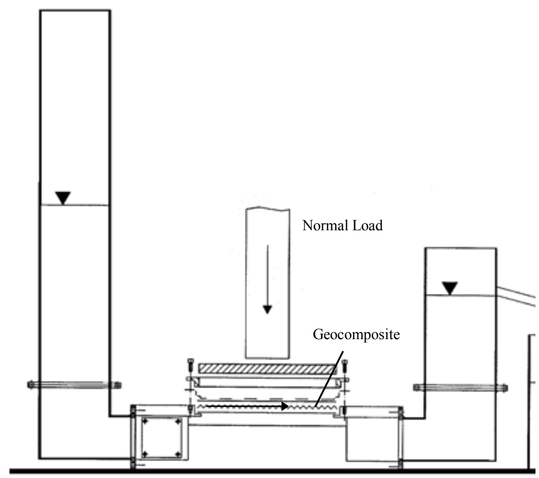

Figure 8 ASTM D-4716 Transmissivity Test

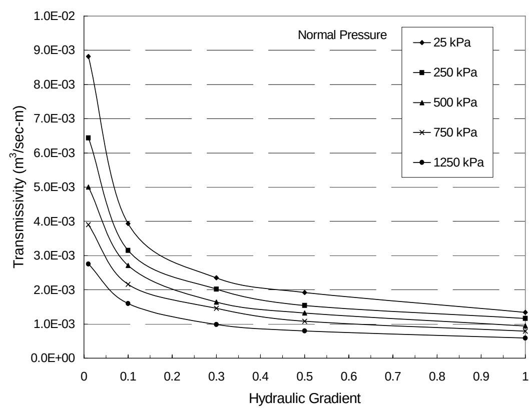

Figure 9 Typical ASTM D-4716 Test Date

Pipe Collection System

While not a ‘geosynthetic’ system, the network of pipes that drain the area collectors have several unique design ‘geo’ considerations. Figure 10 shows a typical collector pipe with an envelop of gravel placed around the pipe. This figure demonstrates two points should be understood by a designer: (1) the envelop of gravel not only filters leachate entering the pipe but significantly increases the normal load that the pipe can carry, and (2) no geotextile is ever placed between the waste and the drainage pipe. Typical leachate is rich in biological activity that can quickly clog a geotextile. Early work by Koerner et al (1993) showed that biological growth due to the leachate quickly and dramatically reduced the permittivity of a geotextile. For this reason, a geotextile may be used beneath the pipe and stone to provide a cushion to protect the geomembrane but it is never used as a filter around the leachate collection pipes.

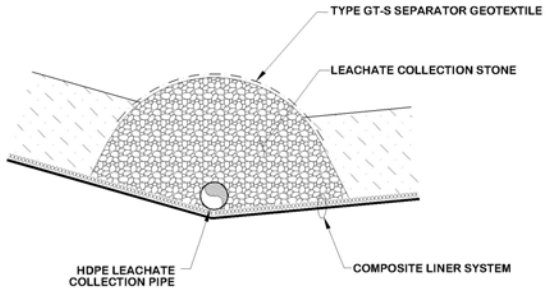

Figure 10 Leachate Drainage Pipe Detail

FINAL COVER DESIGN

Since the 80's, the convention in landfills in the USA is that a landfill cell that has a geomembrane liner must be closed with a final cover that also incorporates a geomembrane. The goal was to ensure that the final cover would allow less surface water infiltration that the anticipated leakage from the liner system. In recent years, this paradigm is changing and the control of gas emissions is now considered more important. This may result in fundamental changes in future designs of final cover systems.

Figure 11 shows the two distinct designs that must be performed for a typical landfill: (1) a flat sloped top-deck design with no slope stability concerns, and (2) the side slopes that may be as step as 3 horizontal-to-1 vertical. The first general rule in final cover design is to prevent water falling on the top-deck from running onto the side slopes. This prevents excessive erosion on the slopes but requires the use of down pipes or swales to carry the water to the base of the landfill. The second general rule in final cover design is to avoid the use of compacted clay layers in the barrier design. Compacted clays will quickly desiccate in the final cover and provide no long term barrier to water or gas migration.

The cover design will usually consist of (1) a layer of soil compacted to provide structural support to the cover system, (2) a geocomposite gas venting layer (for MSW landfill), (3) a geomembrane to limit water and gas migration, (4) a drainage geocomposite drainage layer and (5) a vegetative soil layer to protect the geomembrane, limit erosion, and for esthetic reasons. The geomembrane component will require a rigorous CQA program as previously described for the geomembrane in a liner system.

The design of side slopes presents the added complication of maintaining the slope stability of the veneer system on relatively steep slopes. Surface water percolating through the vegetative soil layer can produce seepage forces acting parallel to the slope if the soil layer saturates.

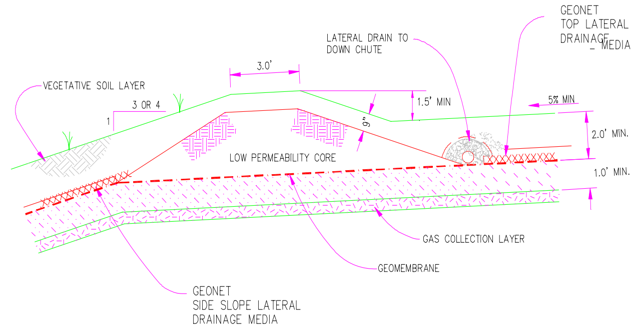

Figure 11 Final Cover Systems

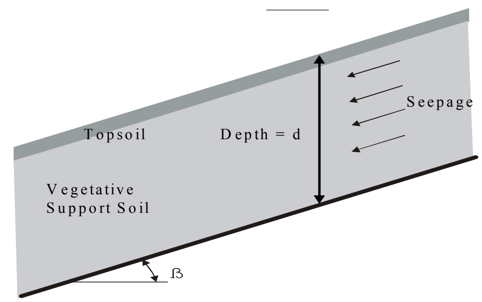

Figure 12 Seepage Forces acting on Side Slopes

If the cover soil fully saturates and the drain layer is inadequate, the slope stability factor of safety is given as:

$$FS = \frac{Resisting\ Forces}{Driving\ Forces} = \frac{\gamma_b d \cos\beta \tan\delta}{\gamma_b d \sin\beta + \gamma_W d \sin\beta} = \frac{\gamma_b \tan\delta}{\gamma_{sat} \tan\beta} \approx 0.5 \frac{\tan\delta}{\tan\beta} \quad (11)$$

where γ_{sat} is the saturated unit weight of the soil and γ_b is the buoyant unit weight of the soil, β is the slope angle, and δ is the interface friction angle.

When such seepage forces are eliminated by using high flow capacity geocomposite drainage layer, the slope safety factor, FS, becomes:

$$FS = \frac{\tan\beta}{\tan\delta} \quad (12)$$

Thus, the use of a geocomposite drainage layer doubles the sliding factor of safety under extreme surface water infiltration.

The geocomposite drainage layer must have sufficient transmissivity to carry the maximum anticipated percolation inflow and adequate interface friction with the adjacent geomembrane and vegetative soils. The design of the pore water pressure drain underlying a saturated cover soil layer was first presented by Thiel and Stewart at the Geosynthetics '93 conference in Vancouver, B.C. The rate of water infiltration into the geocomposite drain can be readily calculated under a unit gradient since the infiltration velocity is equal to the permeability of the vegetative layer. Typical permeability values for vegetative systems range from 1 x 10^{-3} to 1 x 10^{-5} cm/sec. Tighter soils do not allow root penetration and soils looser do not provide adequate water storage. The basic lateral drainage model developed by Thiel is shown on Figure 13.

The quantity of water, Q_{in}, infiltrating into a unit width of drainage composite having a length L is given by

$$Q_{in} = k_{veg} \times L \times 1 \tag{13}$$

The flow capacity of a drainage layer is solved for using Darcy's Law as follows:

$$Q_{out} = k_d \times i \times A = k_d \times i \times (t \times 1) = \left[k_d \times t \right] \times i = \theta \times i \tag{14}$$

where t is the thickness of the drainage layer, i is the flow gradient, and [kt] is transmissivity, θ. For slopes, the gradient i is equal to sinβ, where β is the slope angle. The transmissivity of a geocomposite drainage layer is obtained from laboratory testing as previously described. It is important that the transmissivity be obtained at normal stress levels, boundary conditions, and gradients that reflect actual field conditions.

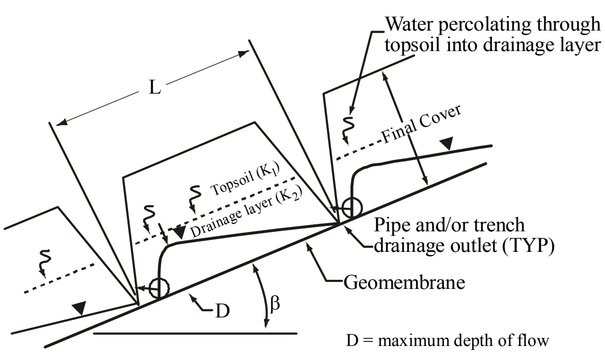

Figure 13 Head Buildup In The Drainage Layer
(Thiel and Stewart, 1993)

It is important to understand the impact of both θ and L on the hydraulic factor of safety. Conveniently, the effective drainage length of the drain can be limited by draining it at each side slope swale commonly used to limit surface erosion. Such swales are commonly 30 to 50 m apart down the slope. The geocomposite drainage layer can be designed to drain into each swale as shown on Figure 14.

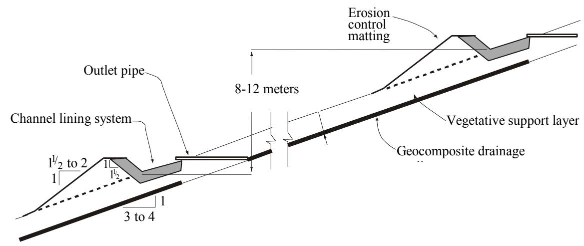

Figure 14 Side Slope Swales and Geocomposite Drains

LESSONS LEARNED FROM FAILURES

Case A

This failure is particularly interesting in that it occurred on a relatively gentle slope. Figure 14 demonstrates that cracks widening between sliding blocks. Figure 15 shows that the vegetative supporting soil was washed to slope base. Initially this failure was thought to a surface erosion problem since the slope was minor. However no erosion 'gullies' running down the slope are visible and the vegetation on the cover is excellent. This led to suspect that something other that run off erosion was occurring.

Figure 15. Cracks widening between sliding blocks

The details of the cover are as follows:

- slope angle = 8.5 degrees, slope length = 94 m
- cover profile: 15cm top soil, 45cm silty sands (k=5*10^{-4} cm/sec), single bonded geocomposite drainage net, and a smooth HDPE geomembrane.
- geocomposite transmissivity = 8*10^{-4} m^2/sec

Figure 16. Vegetative support soils washed to slope base

HELP analyses indicated that the topsoil and sand did not saturate and a peak flow into the geocomposite of 2.5 cm per day, r $=2.9\text{x}10^{-5}$ cm/sec. Thus the peak flow into the geocomposite was calculated $=2.7*10^{-5}$ m^3/sec-m ($2.9*10^{-7}$ m/sec * 94 m * 1). The drainage capacity of the geocomposite is calculated = 1.2 $*10^{-4}$ m^3/sec-m ($8*10^{-4}$ m^3/sec-m * sin(8.5°)). This results in a predicted factor of safety of $1.2*10^{-4}/2.7*10^{-5}$ =4.5. However, inspection of the failed cover clearly indicated that the cover had saturated. Thus the flow into the geocomposite should not have been calculated using HELP model and should have been calculated using the unit gradient design. This produces a peak inflow into the geocomposite of $4.7*10^{-4}$ m^3/sec-m ($5*10^{-6}$ m/sec * 94 m * 1) and an actual factor of safety of $1.2*10^{-4}/4.7*10^{-4}$ =0.26! Clearly the drainage layer was under-designed and the final cover was subject to saturation. As a note, this cover was 'repaired' by removing all materials over the geomembrane and rebuilding with a larger capacity geocomposite and perforated pipes that reduced the effective collection length of the geocomposite to approximately 30 m.

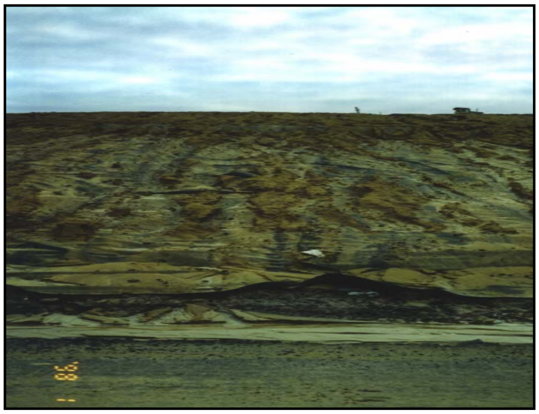

Figure 17. Massive Soils Loss on Slopes

Case B

Massive sliding of cover soils occurred after a major storm dropped 120mm of rain on an East Coast municipal solid waste landfill cap construction project. The rainfall occurred within a span of 5 to 6 hours and damaged approximately 14 hectares of cover. Figure 17 shows massive cover soil loss along the slope, and Figure 18 demonstrates landfill gas pressure built-up under the geomembrane.

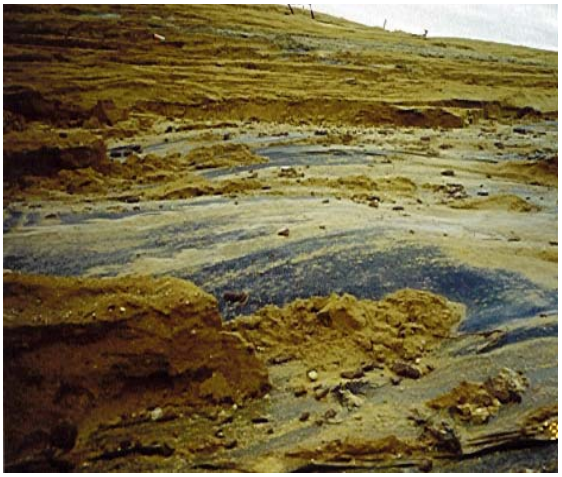

Figure 18. LFG buildup under the geomembrane

Investigation showed that the failure likely resulted from one or more of the following mechanisms:

(a) Inadequate transmissivity in the drainage layer, leading to excessive pore water pressures in the cover soil. Evaluation of the failure is based on the following field conditions that existed at the time of failure:

3:1 slope, β=18.4°
Slope length =122 m
Cover soil permeability, $k=5*10^{-3}$ cm/sec
Saturated unit weight of soil γ_{sat}=17.6 kN/m^3
Transmissivity of the composite lateral drainage layer, $\theta=3.5*10^{-4}$ m^2/sec
Geocomposite/texture geomembrane interface friction angle δ=22°

Field observations and laboratory testing indicated that the in-place soil was saturated. This soil was composed of fine sugar sand containing a high percentage of silt fines. The Unified Classification for this soil is SP-SM. The soil was to function as a vegetative support layer immediately above the final cover geomembrane and drainage geocomposite. The vegetative support layer was to be covered with 150 mm of topsoil supporting grass. Failure occurred before the topsoil layer and associated grass could be placed. Assuming saturation of the vegetative support sands, the factor of safety for the *drainage capacity*, FS_{dc}, of the geocomposite drainage layer can be calculated by the following equation:

$$FS_{dc} = 3.5 \times 10^{-4} \times \frac{0.32}{5 \times 10^{-5} \times 122} = 0.018 \quad (15)$$

Clearly, with a safety factor of 0.018, the transmissivity of the geocomposite is inadequate. Using equation (11), site conditions at this project results in

$$FS = \frac{\gamma_b}{\gamma_{sat}} \frac{\tan\delta}{\tan\beta} = \frac{(17.6 - 9.8)}{17.6} \frac{\tan 22^o}{\tan 18.4^o} = 0.54 \quad (16)$$

Thus, the slope is unstable if the drainage capacity of the drainage net is exceeded**.** No existing geocomposite drains or geotextiles prove sufficient interface friction to enable a cover soil to remain in place for such a steep slope if the cover soil becomes saturated. However, when seepage forces are eliminated, the slope factor of safety of the cover soil per equation (12) was = $\tan\delta / \tan\beta$ or 1.21. Note that a minimum static sliding factor of safety of 1.5 is typically recommended. Thus, even the non-saturated condition was marginal at this site.

In addition to a lack of adequate transmissivity, there were problems of: (b) inadequate gas venting layer, causing LFG pressure buildup below the geomembrane; and, (c) highly erodible silty sands used in the vegetative support layer, causing soil mass loss, especially during storm events.

The failure of the cover soils highlighted significant design errors and construction sequence problems. Each of the mechanisms evaluated above are sufficient to have caused major damage to the partially constructed cover. With the exception of facilities in arid climates, geocomposite lateral drainage systems must be designed assuming the overlying soils become saturated. Given the unusual weather trends that have dominated the past decade, long-term performance of these facilities must accommodate such weather extremes. The construction problems are related to construction in layers versus full sections. This construction practice leaves very large and highly erodible soil surfaces exposed for extended periods. Severe storms will cause major damage to construction when such practices are used. This is independent of the design adequacy of what is being constructed. Many contractors now limit the area of exposure allowed for erodible soil layers unless the contractor can demonstrate that excessive erosion will be mitigated. Incremental slope stability and soil loss evaluations will force this practice.

Based on the forensic analysis, revised analysis methods and repair techniques for this failure are proposed. These repair techniques include

- Reduction of the effective slope length of the drainage layer
- Increase in transmissivity of the drainage layer.
- Decrease the erosion potential of the soils.
- Increase the capacity of the existing gas collection blanket.

SUMMARY

Geosynthetic components provide the only means for economically containing the waste byproducts of or cultures. Fortunately, these components are economical and made to very high standards. This paper has developed the simple but essential design considerations that must be evaluated for the successful development of a contemporary lined landfill.

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