Slope Stability Sensitivities of Final Covers

R. Thiel, Thiel Engineering, Oregon House, California, USA

ABSTRACT
The geotechnical aspects of final cover system slope stability follow the same principals used for other geotechnical stability problems. What is unique and important to recognize relative to final cover systems is their sensitivity to relatively small changes in loading, slope angle, pore pressures or shear strengths that make them more susceptible to sliding failures. This extra sensitivity shows up in the failure rates observed for bottom liner failures versus final cover failures in the lining containment industry. While there may be only less than a dozen publicized bottom liner failures in the United States over the past 20 years, there have been dozens of veneer cover system failures that have occurred. When analyzing a final cover system for slope stability, design practitioners might consider an evaluation of the project reliability in addition to a factor-of-safety approach.

1. INTRODUCTION

Final cover systems for environmental projects, such as landfills, can be characterized as a veneer soil layer of thickness "t" on a surface sloped at angle β to the horizontal, as shown in Figure 1. The critical slip surface is considered to occur on a barrier layer, such as a geomembrane. The tangential weight, $W_A \cdot \mathbf{sin}(\beta)$, of the soil mass is the driving force that might cause the soil veneer to slip. The resisting forces against slippage are (a) the shear strength, F_A, between the soil mass and the slip surface; the toe buttressing force, F_P, at the toe of the slope; and if the failure surface is below geosynthetics then there could be a tensile force, T, from the geosynthetics.

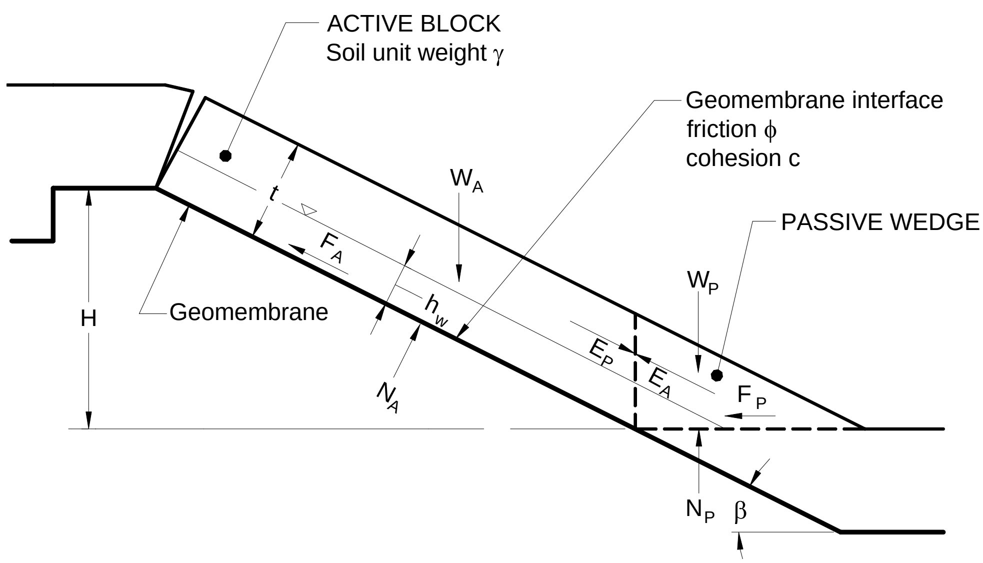

Figure 1: Schematic of veneer cover system with geomembrane liner.

For very short slopes the toe of the slope actually provides a buttressing effect, and can be considered a significant element of the veneer stability. In general the buttressing toe force starts becoming insignificant when the vertical height H of a slope is more than about 30 times the thickness of the veneer cover soil. Thus, for example, a designer could ignore toe buttressing forces on a 0.5 m thick cover for slope heights greater than about 12 m high, without being unduly conservative.

The goal is normally to provide all of the slope resistance with shear forces and not to put the geosynthetics in tension. Unless a tension element, such as a geogrid or high-strength geotextile, is specifically designed to help support the cover system, the industry standard is to ignore any tensile strength contribution from geosynthetics in the veneer stability. Except for unusual situations, experience has shown that it is usually not cost-effective to design for veneer reinforcement on slopes, and better designs can usually be found that transmit all of the forces by shear. Furthermore, as slopes become longer, the contribution of tensile reinforcement becomes less and less significant, the same as for toe buttressing.

The shear strength along the critical interface is often a function of the effective normal force acting on this interface. This relationship between shear strength and normal force is called the shear strength envelope, and is often expressed in terms of the Mohr-Coulomb parameters of “friction angle” (ϕ) and “cohesion” (c). These terms are nothing more than mathematical expressions of the slope of the shear strength envelope and the “y-intercept”. Since the actual shear strength envelope is often not a perfectly straight line, but is curved, as illustrated in Figure 2, it is important that the Mohr-Coulomb parameters be defined within a specific normal load range, as it is always un-conservative to extrapolate the envelope, either to lower normal loads or to higher ones. The shear strength on the interface can be reduced by pore pressures, u, because pore pressures serve to reduce the effective normal force acting on the interface. Thus, flowing water or landfill gas pressures would be destabilizing forces.

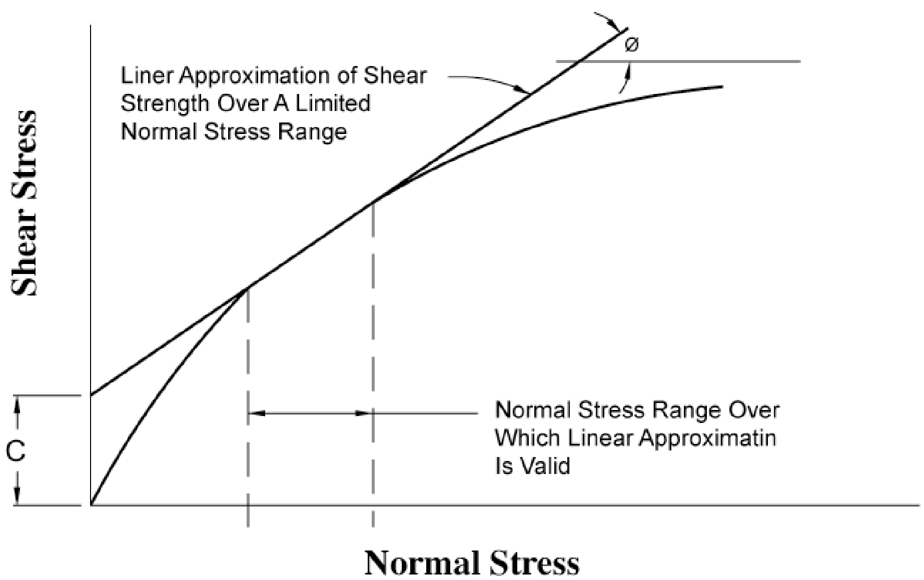

Figure 2: Mohr-Coulomb straight-line interpretation of curvilinear strength envelope.

2. COVER VENEER SYSTEM STABILITY ANALYSES

The analytical approach for the evaluation of landfill cover system stability has been described extensively in the literature by Koerner and Soong (1998), Thiel and Stewart (1993), Giroud et al. (1995a and 1995b), Thiel (1998), Kavazanjian (1998) and others. These and other literature citations include evaluations of toe buttress forces, tapered slopes, seismic forces, equipment forces, tensile forces, gas pressures, and water seepage effects on veneer cover stability.

A simple, brief review of cover stability equations for an “infinite slope” is presented here to provide a context for the remainder of the paper. An “infinite slope” is one whose surface is parallel to the sliding surface, with no consideration for either toe or tensile forces. Often an infinite-slope analysis is appropriate for many projects, and provides a simple evaluation whereby the key aspects of a problem can be evaluated. A diagram for an infinite soil slope on a geomembrane is illustrated in Figure 3.

For a purely frictional interface under dry conditions, with an angle of the slope to the horizontal of β, the factor of safety (FS) against sliding is

$$\mathrm{FS} = \frac{\tan\phi}{\tan\beta} \qquad [1]$$

If the slope with soil thickness t is exposed to precipitation, and water builds up on the liner and flows in the cover soil parallel to the slope at a depth h, the revised expression for FS on the top surface of the geomembrane is calculated as

$$FS = \frac{\left[(t-h)\gamma_s + h\,\gamma_{sat} - h\,\gamma_w\right]}{\left[(t-h)\gamma_s + h\,\gamma_{sat}\right]} \bullet \frac{\tan\phi}{\tan\beta} \qquad [2]$$

where γ_s = unit weight of the unsaturated soil; γ_{sat} = unit weight of the saturated soil; and γ_w = unit weight of water. Note that in the case where the soil is fully saturated, i.e., when h = t, then the equation reduces to

$$FS = \frac{\gamma_b}{\gamma_{sat}} \bullet \frac{\tan\phi}{\tan\beta} \qquad [3]$$

where γ_b = buoyant unit weight of the soil. Since the buoyant unit weight of soil is approximately half of the total saturated unit weight of soil, we can see that the factor of safety gets cut in half when the veneer soil is saturated. This means that a veneer cover designed with a factor of safety of 2 under dry conditions will have a factor of safety of 1 (imminent failure) under saturated conditions. This veneer failure mechanism is commonly observed on highway road cuts where the surficial soil weathers over a hard subsoil, and fails during heavy rains.

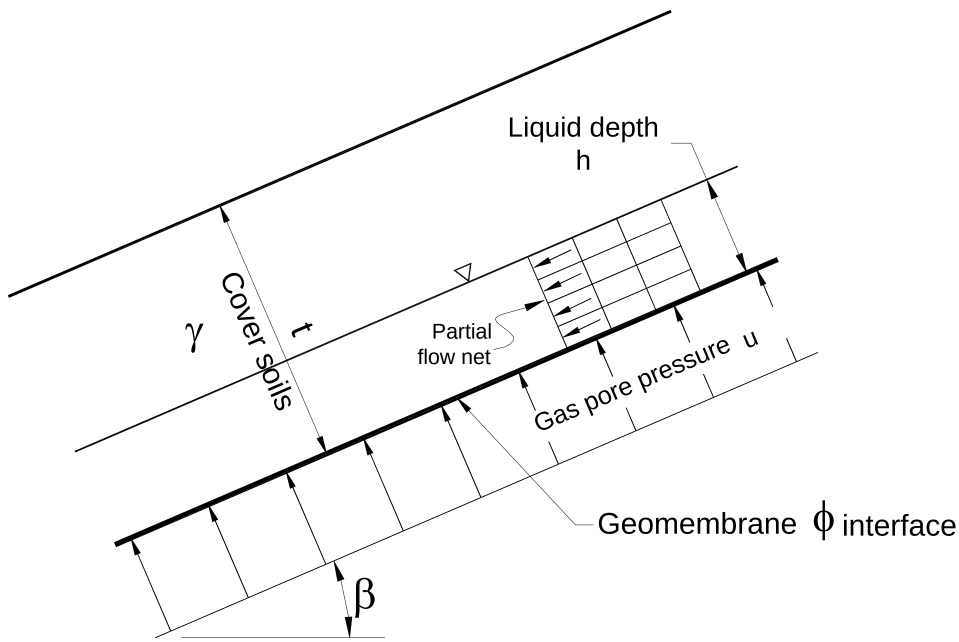

Figure 3: Infinite slope schematic for veneer cover soil showing forces occurring both above and below the geomembrane.

Pore pressures on the top side of the geomembrane do not affect the stability of the interface on the underside of the geomembrane. However, gas pressures from below would affect the stability of the lower interface. The effect of a gas pressure, u, on the stability of the lower interface would be as follows:

$$FS = \frac{\left[t\,\gamma_s \bullet \cos\beta - u\right]}{\left[t\,\gamma_s \bullet \sin\beta\right]} \bullet \tan\phi \qquad [4]$$

3. COVER VENEER SYSTEM SENSITIVITY EXAMPLE

Graphs of the changes in factor of safety of veneer cover system stability versus changes in different parameters were presented by Koerner and Soong (1998). They presented a base case cover system at a slope angle (β) of 18.4 degrees, a slope length (L) of 30 m, a cover soil thickness (t) of 300 mm with a moist unit weight (γ_s) of 18 kN/m^3, a saturated unit weight of 21 kN/m^3, and an interface friction angle (ϕ) on the sliding surface of 22 degrees. Using a quadratic formula solution that included toe buttressing resistance they calculated FS = 1.25 for this case. Using

Equation 1 above provides FS = 1.21. Note that for this example the height of the slope (10 m) is approximately 33 times the soil cover thickness, which is on the border line where toe buttressing is of relative significance. Although the current author would enjoy using these same base-case conditions for this paper, this is not reasonable because Koerner and Soong (1998) did not consider gas pressures from below as one of their scenarios, and any reasonable consideration of landfill gas pressures (e.g. 1 kPa) will indicate FS < 1 for their base case, which represents slope failure. This little exercise alone serves to make the point of this paper!

The main contribution of this paper compared to the other literature references, especially the Koerner and Soong (1998) paper, is a spotlight on the high level of sensitivity that final cover systems have to relatively small changes in various parameters. The approach used in this paper is through a simple reliability analysis as outlined by Duncan (2000), wherein a Taylor series method is outlined in a manner accessible to design practitioners that are not steeped in statistical theory. The main steps of this method, as presented by Duncan (2000), are as follows:

1. Determine the most likely values of the parameters involved and compute the factor of safety using whatever equations you wish. This is called the most likely factor of safety, or F_{MLV}.

2. Estimate the standard deviations of the parameters that involve uncertainty. Various methods may be used to arrive at these values. Where there is little data on which to base a standard deviation, Duncan (2000) suggests that the standard deviation, σ, can be estimated if the design practitioner can estimate the highest conceivable value (HCV) and the lowest conceivable value (LCV) of the parameter under consideration. Using the "Three-Sigma Rule" the standard deviation can then be estimated by dividing the difference between the HCV and the LCV by 6:

$$\sigma = \frac{\mathrm{HCV} - \mathrm{LCV}}{6} \qquad [5]$$

3. Compute the factor of safety with each parameter increased by one standard deviation and then decreased by one standard deviation from its most likely value, having the other parameters equal to their most likely values. The difference in the factors of safety using the plus-σ and minus-σ values for a given parameter is termed ΔF. A separate ΔF is calculated for each desired parameter, and the standard deviation of the factor of safety, σ_F, is computed using a Taylor series technique as follows:

$$\sigma_F = \sqrt{\left(\frac{\Delta F_1}{2}\right)^2 + \left(\frac{\Delta F_2}{2}\right)^2 + \left(\frac{\Delta F_3}{2}\right)^2 + \ldots} \qquad [6]$$

4. Compute the coefficient of variation of the factor of safety, V, as follows:

$$V = \frac{\sigma_F}{F_{MLV}} \qquad [7]$$

5. Using the values of F_{MLV} and V the lognormal reliability index, β_{LN}, can be calculated as:

$$\beta_{LN} = \frac{\ln\left(\frac{F_{MLV}}{\sqrt{1+V^2}}\right)}{\sqrt{\ln(1+V^2}} \qquad [8]$$

6. The reliability, R, is then calculated using the NORMDIST function in Excel, with β_{LN} as the argument. The probability of failure, P_f, is equal to 1-R.

The approach described above can be programmed very simply onto an Excel spreadsheet, and a designer can quickly become sensitive to the reliability of a particular design. This approach can be used for almost any model whose variables can be defined with estimated standard deviations.

For our case of a veneer cover on a slope, a design example is used where the key design parameters and selected standard deviations are presented in Tables 1 and 2 for potential sliding surfaces above and below the geomembrane. Note that other parameters could have been included as well (e.g. variation in unit weights of cover soils, equipment forces, etc.). The list selected is adequate to make the point of this paper.

Table 1. Veneer slope stability design parameters and estimated standard deviations for failure above the liner.

Parameter	MLV	HCV	LCV	σ	Comments
cos β (angle in degrees)	0.9489 (18.4°)	0.9285 (21.8°)	0.9622 (15.8°)	0.0056	There can be high variations in slope angle on landfills. Assume may vary from 1(V):2.5(H) to 1:3.5.
tan ϕ (angle in degrees)	0.5095 (27°)	0.6494 (33°)	0.3839 (21°)	0.0443	This range could represent peak and post-peak strengths, bentonite extrusion from GCLs, or some similar variation. Duncan's (2000) largest criticism of selecting the HCV and LCV was that engineers have a tendency to make the range too small.
t (mm)	500	550	470	13.3	This range represents typical construction earthwork tolerances.
h (mm)	3	500	0	83	See text for more discussion.

Of all the parameters listed in Table 1, the most difficult to reconcile is the depth of the head buildup over the liner. The design factors affecting the maximum head buildup are (a) the hydraulic conductivity of the overlying soil, (b) the transmissivity of the drainage layer, (c) the slope angle, and (d) the spacing between outlet drains. Oddly enough, climatic conditions generally do not matter because for most sites, except for the most arid, there are usually design-rain periods and events that could cause saturation of the cover soils for more than a 30-minute period, which is typically long enough to precipitate critical drainage conditions that could lead to a failure. The design condition that the liquid thickness, h, in the drainage layer be less than the thickness of the drainage layer is absolutely essential to the proper functioning and stability of a cover system. This point is clearly made and emphasized by Giroud et al. (2000), and a design methodology to accomplish this was first clearly presented by Thiel and Stewart (1993). For this example, a geocomposite is used, which only allows a maximum head buildup of about 5 mm, leaving little room for error. Often, with drainage geocomposites, if they fail then there is a high probability of total failure, meaning that the full system will have a drainage backup. In a cover system this can easily lead to a total slope stability failure, as governed by Equation 3. This is illustrated in Figure 4 where a graph of drainage layer transmissivity versus safety factor and submergence ratio shows how the safety factor drops very steeply over a short range of drainage transmissivity. The reason this happens is that as soon as the drainage layer becomes full, it touches the overlying saturated cover soil, and the depth of saturation jumps from the top of the geocomposite all the way up to the top of the final cover soil. These types of failures have occurred on several projects where (a) the hydraulic conductivity of the cover soil was under-estimated, (b) the length of the slope was too long, or (c) the transmissivity of the drainage layer was inadequate.

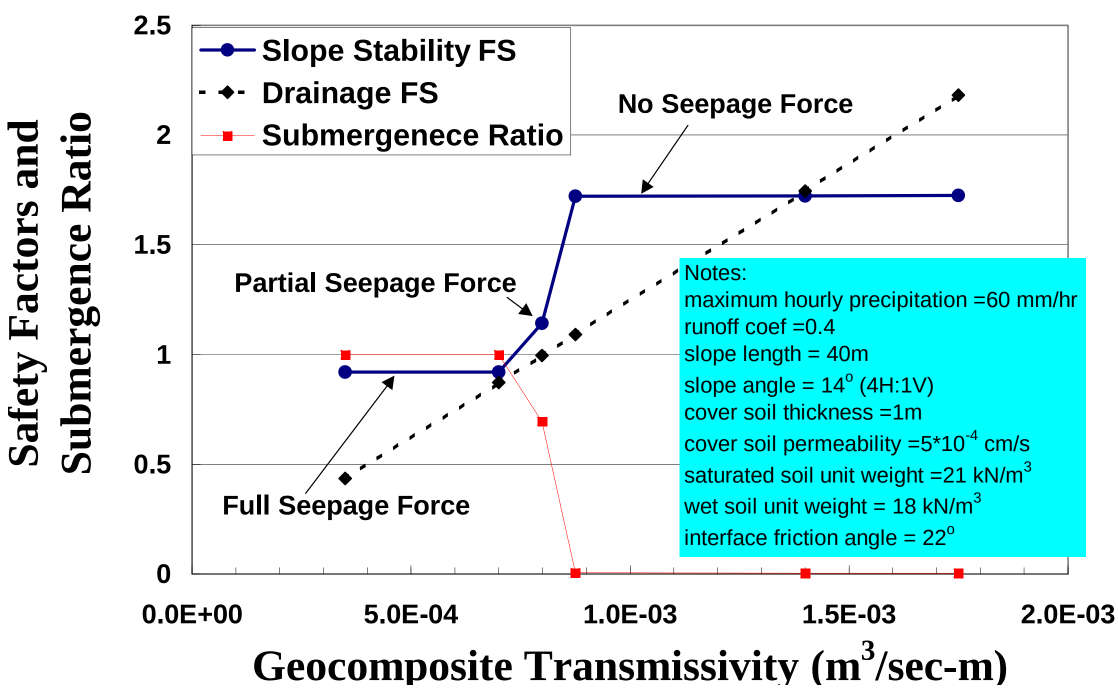

Figure 4: Quantum jump in seepage force over a small range of drainage layer transmissivity.
(Figure courtesy of Greg Richardson)

Table 2. Veneer slope stability design parameters and estimated standard deviations for failure below the liner.

Parameter	MLV	HCV	LCV	σ	Comments
cos β (angle in degrees)	0.9489 (18.4°)	0.9285 (21.8°)	0.9622 (15.8°)	0.0056	There can be high variations in slope angle on landfills. Assume may vary from 1(V):2.5(H) to 1:3.5.
tan ϕ (angle in degrees)	0.5095 (27°)	0.6494 (33°)	0.3839 (21°)	0.0443	This range could represent peak and post-peak strengths, bentonite extrusion from GCLs, or some similar variation. Duncan's (2000) largest criticism of selecting the HCV and LCV was that engineers have a tendency to make the range too small.
t (mm)	500	550	470	13.3	This range represents typical construction earthwork tolerances.
u (kPa)	1	4	0	0.67	A discussion of the typical and upper values for gas pressure from a landfill is provided in Thiel (1998).

Reliability analyses were performed using the parameters given in Tables 1 and 2. Equation 2 was used in conjunction with Table 1 to calculate the factors of safety for a potential sliding surface above the geomembrane. Equation 4 was used in conjunction with Table 2 to calculate the factors of safety for a potential sliding surface below the geomembrane. The results of the spreadsheet calculations for the factors of safety are presented in Table 3, and the reliability and probability of failure calculations are presented in Table 4.

Table 3. Spreadsheet calculation of factors of safety.

Condition	Moist soil unit wt (kN/m3)	Sat soil unit wt (kN/m3)	Thick-ness t (m)	Liquid depth h (m)	Friction angle phi	Slope angle beta	Gas pressure u (kPa)	FS
Above GM								
MLV	18	21	0.5000	0.003	27.00	18.40	0	1.527
FS+σ for co	18	21	0.5000	0.003	27.00	**17.35**	0	1.626
FS-σ for cos	18	21	0.5000	0.003	27.00	**19.39**	0	1.443
FS+σ for tar	18	21	0.5000	0.003	**28.98**	18.40	0	1.660
FS-σ for tan	18	21	0.5000	0.003	**24.95**	18.40	0	1.394
FS+σ for t	18	21	**0.5133**	0.003	27.00	18.40	0	1.527
FS-σ for t	18	21	**0.4867**	0.003	27.00	18.40	0	1.527
FS+σ for h	18	21	0.5000	**0.083**	27.00	18.40	0	1.397
FS-σ for h	18	21	0.5000	**-**	27.00	18.40	0	1.532
Below GM								
MLV	18	21	0.5000	**-**	27.00	18.40	1.00	1.352
FS+σ for co	18	21	0.5000	**-**	27.00	**17.35**	1.00	1.441
FS-σ for cos	18	21	0.5000	**-**	27.00	**19.39**	1.00	1.277
FS+σ for tar	18	21	0.5000	**-**	**28.98**	18.40	1.00	1.470
FS-σ for tan	18	21	0.5000	**-**	**24.95**	18.40	1.00	1.235
FS+σ for t	18	21	**0.5133**	**-**	27.00	18.40	1.00	1.357
FS-σ for t	18	21	**0.4867**	**-**	27.00	18.40	1.00	1.347
FS+σ for u	18	21	0.5000	**-**	27.00	18.40	**1.67**	1.232
FS-σ for u	18	21	0.5000	**-**	27.00	18.40	**0.33**	1.473

4. DISCUSSION OF RESULTS

The results presented on Table 4 indicate that the most likely factor of safety on the top surface of the geomembrane is F_{MLV} = 1.53, and the probability of failure P_f = 0.013%. This represents about a 1:7,700 probability of failure, which would generally be considered excellent. According to Whitman (1984) this would be just below the range of reliability acceptable for large dams. The results even here can be somewhat deceiving, because if the geocomposite drainage layer did fail and the head level exceeded 5 mm (the thickness of the geocomposite), then the head would immediately jump to the full thickness of the cover soil, and the factor of safety would drop below unity, implying a slope failure. This quantum jump in the liquid depth is not conveniently modeled by the simplistic reliability model suggested by Duncan (2000) and presented herein. Note that there have been dozens of veneer slope stability failures in the industry, with the vast majority of the failures being attributed to overloading (or under-designing) of the drainage layer.

The results for the bottom surface of the geomembrane indicate F_{MLV} = 1.35, and the probability of failure P_f = 1.71%. This represents a 1:58 probability of failure. According to Whitman (1984) this would be in the range of reliability acceptable for pit mine slope stability, where occasional failures are tolerated. For environmental closure projects this level of reliability would generally be considered unacceptable. Thus it can be seen that the influence of landfill gas pressures on the veneer stability is perhaps unacceptable, and in this case it would be warranted to apply the design procedures recommended by Thiel (1998) to address this situation to achieve an acceptable level of reliability.

5. CONCLUSIONS

Final cover systems are very sensitive to relatively small changes in normal force, pore pressures, and shear strengths. A simple factor-of-safety approach to modeling a veneer cover system may hide the true risks of a proposed project.

As with any design, no level of analytical sophistication can replace good, basic, sound engineering. The author has been involved in failure evaluations where the pre-construction calculated factor of safety "exceeded" 1.5, and has peer reviewed several projects where fundamental modeling errors were made when evaluating the slope stability of geosynthetics lined systems. These errors were easily corrected by applying simple principles in an appropriate manner. Thus, before diving into a reliability calculation, it is important to make sure that the system is appropriately modeled to allow calculation of a good, basic factor of safety that accounts for all significant loadings and resistances.

The procedures outlined in this paper are not new or original to this author. The procedures are not intended to make the design practitioner's work more complicated or cumbersome, but to make designers more aware of how they can account for the sensitivity of cover veneer systems to small changes in design parameters.

One common question asked by owners, designer, and regulators is: what is an acceptable factor of safety. In geotechnical stability analyses, a value of F_{MLV} = 1.5 has long been an industry standard. For certain temporary or non-critical situations there is a legitimate tendency to consider how much lower the factor of safety can be, and still be acceptable. Understanding the sensitivity of final cover systems to small changes in design parameters, and applying a simple reliability model, can provide a useful tool for all parties to understand the increased risk (and potential cost) versus the benefits of using a lower F_{MLV}.

Table 4. Spreadsheet calculation of reliability, R, and probability of failure, P_f, for the design example.

		Slope angle β			Friction angle ϕ			Cover thickness t			Liquid depth h							
CONDITION	**F_{MLV}**	**FS-σ**	**FS+σ**	**ΔF_1**	**FS-σ**	**FS+σ**	**ΔF_2**	**FS-σ**	**FS+σ**	**ΔF_3**	**FS-σ**	**FS+σ**	**ΔF_4**	**σ_F**	**V_F**	**lognormal reliability index β_{LN}**	**Reliabil-ity**	**Proba-bility of failure Pf**
Sliding above geomem-brane using Eqn. 2	1.527	1.443	1.626	0.183	1.394	1.660	0.266	1.527	1.527	-	1.397	1.532	0.135	0.175	0.115	3.6491	99.99%	**0.013%**
		Slope angle β			Friction angle ϕ			Cover thickness t			Gas pressure u							
Sliding below geomem-brane using Eqn. 4	1.352	1.277	1.441	0.164	1.235	1.470	0.235	1.347	1.357	0.010	1.232	1.473	0.241	0.187	0.139	2.1186	98.29%	**1.706%**

Pf	**Scenarios**
10.000%	Upper end of what is considered acceptable for pit mine slope stablity, where occaisional failures are tolerated.
5.000%	Mid range of what is considered acceptable for pit mine slope stablity, where occaisional failures are tolerated.
1.000%	Considered acceptable for building foundation settlement, or for fixed ocean drill rigs.
0.100%	Considered in the mid range for fixed ocean drill rigs.
0.010%	Considered in the upper range acceptable for dams.
0.001%	Considered in the lower range acceptable for dams.
	- After Whitman, *Evaluating Calculated Risk in Geotechnical Engineering* , ASCE 1984

REFERENCES

Duncan, J.M. (2000). Factors of Safety and Reliability in Geotechnical Engineering, *J. of Geotechnical and Geoenvironmental Engineering,* ASCE, V126 N4: 307-316.

Giroud, J.P., Williams, N.D., Pelte, T., and Beech, J.F. (1995a). Stability of Geosynthetic-Soil Layered Systems on Slopes, *Geosynthetics International*, IFAI, V2 N6: 1115-1148.

Giroud, J.P., Bachus, R.C., and Bonaparte, R. (1995b). Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes, *Geosynthetics International*, IFAI, V2 N6: 1149-1180.

Giroud, J.P., Zornberg, J.G., and Zhao, A. (2000). Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers, *Geosynthetics International*, IFAI, V7 N4-6: 285-380.

Kavazanjian, E. (1998). Current Issues in Seismic Design of Geosynthetic Cover Systems, *Sixth International Conference on Geosynthetics*, IFAI, Atlanta, GA, USA: 219-226.

Koerner, R.M. and Soong, T.Y. (1998). Analysis and Design of Veneer Cover Soils, *Sixth International Conference on Geosynthetics*, IFAI, Atlanta, GA, USA: 1-23.

Thiel, R. (1998). Design Methodology for a Gas Pressure Relief Layer Below a Geomembrane Landfill Cover to Improve Slope Stability, *Geosynthetics International*, Vol. 5, No. 6: 589-616.

Thiel, R.S. and Stewart, M.G. (1993). Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest, *Geosynthetics '93,* IFAI, Vancouver, B.C., USA: 1131-1134.

Whitman, R.V. (1984). Evaluating Calculated Risk in Geotechnical Engineering. *J. of Geotechnical Engineering*, ASCE, V110 N2: 145-188.