PEAK VERSUS RESIDUAL STRENGTH FOR WASTE CONTAINMENT SYSTEMS

The Problem – waste containment systems contain numerous slip surfaces along which shear displacement (strain softening) can occur. When these surfaces are sheared, a peak shear strength is mobilized at a small displacement (typically 0.5 inches) and then the shear strength decreases to a residual strength with continuing displacement. In this paper, Gilbert provides four design guidelines to help address this problem. The third guideline is the concept that is most commonly misunderstood.

A common misconception is that the residual strength for a containment system is the minimum residual strength among all components in the system. This causes some designers to hesitate to use needlepunched geosynthetic clay liner (GCLs) because they may have a low residual internal shear strength, similar to that of pure bentonite. However, the residual strength can only be mobilized if the peak strength is exceeded. Therefore, the residual strength for the system is the residual strength of the component with the lowest peak strength.

An example of this concept is shown in Figure 3. This double liner system has a primary composite liner, consisting of a textured geomembrane over a reinforced GCL, underlain by a leak detection system with a drainage geocomposite. Direct shear tests were run on each of the possible slip surfaces. The peak strength for the interface between the GCL and the drainage geocomposite is less than the internal GCL peak strength; thus, the reinforcing fibers in the GCL are not stressed to failure and there is little shear displacement. Therefore, the residual strength for this system is that for in the interface between the GCL and the drainage composite and not the minimum residual strength occurring internally within the GCL. In a sense, the interface between the GCL and the geocomposite protects the GCL from undergoing significant internal shear displacement.

This concept is important because in design because it can be used to prevent significant strain softening from occurring in the barrier material(s) by purposely creating a weaker interface above the barrier. Sometimes referred to as "base isolation", this idea was written about over 10 years ago by Von Pein and Prasad (1990). It has been used in numerous landfills, including earthquake-prone California, over the past decade. And it was successfully put to the test in the Northridge earthquake as documented in a GFR article (see CETCO TR-211).

The factors of safety used in Gilbert's paper were a matter of debate between engineers, academicians and regulators at the 15th Geosynthetic Research Institute Conference. However, there was consensus that the residual strength for the system is the residual strength of the component (typically an interface) with the lowest peak strength. Consequently, because of needlepunched GCL's relatively high peak internal shear strength, an engineer can easily design so that the residual internal shear strength of the GCL is not a critical issue.

References:

Von Pien, R.T. and Prasad, S., "Composite Lining System Design Issues", *Proceedings of the 4th GRI Conference*, Philadelphia, December 1990.

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ABSTRACT

The following guidelines are provided to address the question of whether to use peak or residual strengths in the design of liner and cover slopes for waste containment systems:

1. Slopes in liner and cover systems should generally be designed so that the factor of safety (FS) against sliding is greater than or equal to one with the residual strength for the system.
2. If a slope is designed using a factor of safety less than one with the residual strength for the system, then the consequences and probability of sliding should be considered explicitly and accepted by all stakeholders (the owner, operator, regulator and designer).
3. The residual strength for a liner or a cover system is not necessarily the minimum residual strength among all of the components in the system.
4. Both peak and residual strengths for all interfaces in a system are needed for design.

THE PROBLEM

Waste containment systems contain numerous slip surfaces along which strain softening can occur. When these surfaces are sheared, a peak strength is mobilized at a small displacement (typically several millimeters) and then the strength decreases to a residual strength with continuing displacement (e.g., Figure 1). The problem with strain softening is that the strength that will be available in the field is uncertain. Should the slope be designed assuming that the peak strength is available along the entire slip surface? Should it be designed assuming that only the residual is available? These are not easy questions to answer because the consequences of relying on peak strengths can be large (a possible slope failure if the peak strength is not available) and the consequences of relying only on residual strengths can also be large (flat slopes and lost air space).

In this paper, I provide four design guidelines to help address this problem.

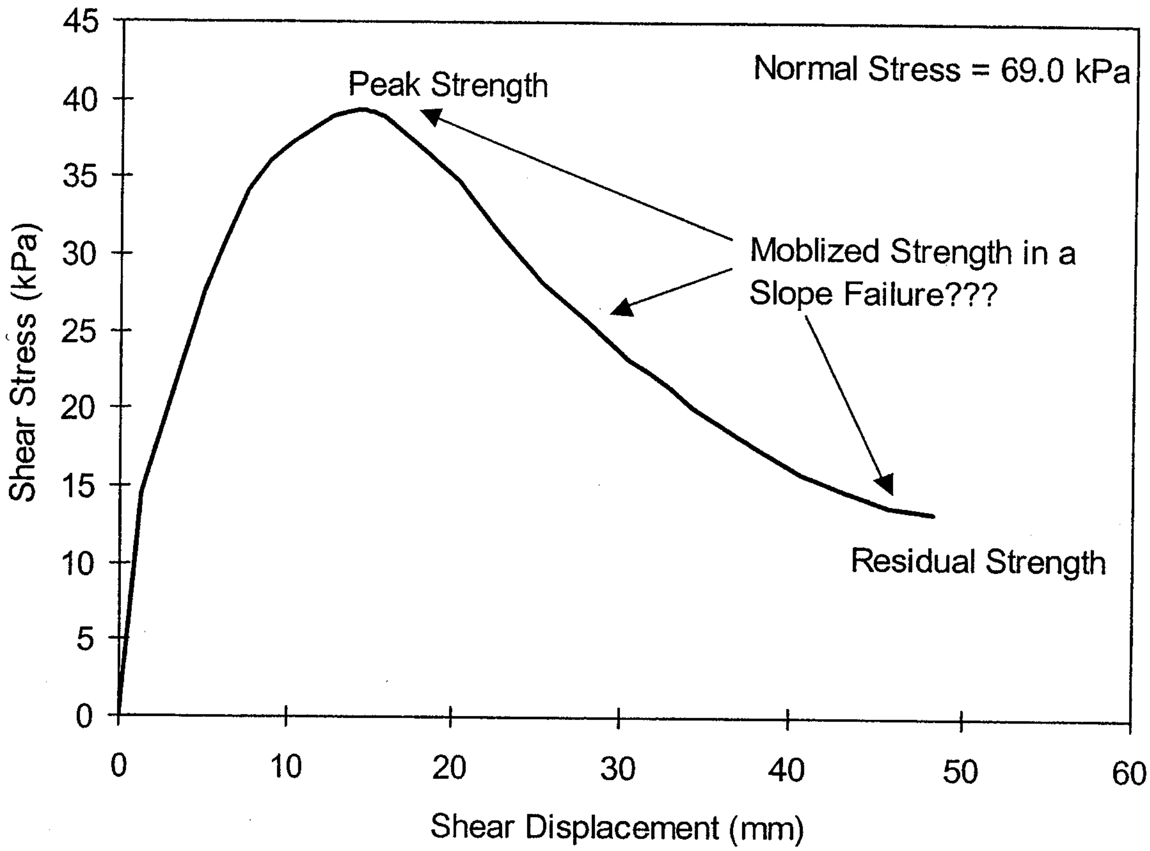

Fig. 1 Example of Strain Softening (Data for Interface between Textured Geomembrane and Reinforced GCL from Gilbert et al. 1996)

GUIDELINE 1 – GENERALLY USE FS ≥ 1.0 WITH THE RESIDUAL STRENGTH FOR SYSTEM

Why not allow a FS < 1.0 with residual strengths?

The first reason for not allowing a factor of safety less than 1.0 with residual strengths is the potential for failure. There are numerous examples since the inception of "soil mechanics" where cuts and slopes in natural strain-softening soils failed when the factors of safety were less than 1.0 with residual strengths. Nearly 40 years ago, Skempton (1964) described seven case histories in his Rankine lecture and Bjerrum (1967) subsequently described an additional nine in his Terzaghi lecture. There are now numerous examples in containment systems with the same result. If the Kettleman Hills landfill had been designed using this recommended approach, then it would not have failed (e.g., Byrne et al. 1992 and Stark and Poeppel 1994). Hence, it is well established that the potential for failure exists if a slope is designed using a factor of safety less than 1.0 with residual strengths.

The second reason for not allowing a factor of safety less than 1.0 with residual strengths is the consequence of failure. A slope that requires more than the residual strength to be stable will accelerate and experience sudden, large deformations if it fails because the mobilized strength will drop to the residual strength (Figure 1). This sudden release of energy during strain softening can have significant consequences. For example, the waste in the Kettleman Hills failure moved more than 10 m along the liner system, destroying the liner system and causing tens of millions of dollars of damage. Koerner and Soong (2000) provide additional examples of high-consequence failures. Both the consequence of and the potential for a slope failure are minimized using a factor of safety greater than or equal to 1.0 with the residual strength.

Why not require a FS > 1.0, such as 1.3 or 1.5, with residual strengths?

There is little benefit in general to using a factor of safety greater than 1.0 with residual strengths. First, the consequences of a slope failure are relatively small if the slope is designed using a factor of safety equal to 1.0 with residual strengths because there is little potential for large deformations due to strain softening. If the slope is not stable, then the rate of movement will be very small, inclinometers and horizontal benchmarks can be used to monitor the movement, and ample time will be available to decide whether and how to stabilize the slope.

Second, the potential for failure is generally already very small for a factor of safety equal to 1.0 with residual strengths and it will not decrease very much with a higher factor of safety. The uncertainty in the stability of a slope, which is why we conventionally use a factor of safety greater than 1.0 in design, is generally dominated by uncertainty in the shear strength. For example, the measured peak shear strength in a laboratory test can be affected by many different variables such as the rate of shear, the compacted moisture content and density for soils, small surface features at interfaces, and the size and orientation of the test specimen. However, the residual strength is much more of a fundamental material property; it depends primarily on the chemical composition of the soils and geosynthetics. Hence, there is substantially less uncertainty in estimating a residual strength compared to estimating a peak strength. Furthermore, the main source of uncertainty that does exist in using a residual strength is what the mobilized strength will actually be in the field. Since it will likely be greater than or equal to the residual strength (most likely somewhere in between the residual and peak strengths), there is already a measure of conservatism built into using a factor of safety equal to 1.0 with residual strengths.

GUIDELINE 2 – CONSIDER AND ACCEPT RISKS IF USING FS < 1.0 WITH RESIDUAL STRENGTHS

Can a slope be stable using a FS < 1.0 with residual strengths?

Yes! In most case histories of failure with strain-softening materials in natural slopes and in containment systems the mobilized strength at failure was somewhere between the residual and peak strength. For example, the factor of safety with residual strengths at the time of failure ranged from 1.0 down to 0.7 in the case histories with natural soils described by Bjerrum (1967). As another example, the factor of safety with residual strengths was about 0.8 for the Kettleman Hills failure, meaning that the mobilized strength was about halfway between the residual and the peak strength (Byrne et al. 1992).

The potential for mobilizing residual strengths in the field depends on several factors. In order to mobilize the residual strength, the peak strength has to be exceeded locally. This localized stress will cause local displacement that may lead to a progressive strain softening and subsequent failure of the slope. Gilbert and Byrne (1996) present a simple model that relates the mobilized strength to the slope geometry and the properties of materials in a containment system slope. This model indicates that the potential for mobilizing residual strengths increases as the applied stress increases, as the length of the slip surface increases, as the stiffness of the material above the slip surface decreases, and as the rate of strength reduction with displacement increases. The general conclusion drawn from application of this model is that containment slopes that require either a buttress force at the toe or a tension force at the crest (e.g., geosynthetics in tension) for stability are likely to mobilize residual strengths, whether they are in a cover or in a liner. Conversely, the potential for mobilizing residual strengths would be lowest in covers that are designed so that the applied stress is less than the minimum peak shear strength in the system (i.e., an infinite slope that is stable).

What affects the risk of a failure?

The risk of failure is obtained by multiplying the probability of failure by the consequence of failure (e.g., Liu et al. 1997). Figure 2 provides qualitative guidance on how the risk of failure is affected by the type of slope for slopes designed using a factor of safety less than one with residual strengths.

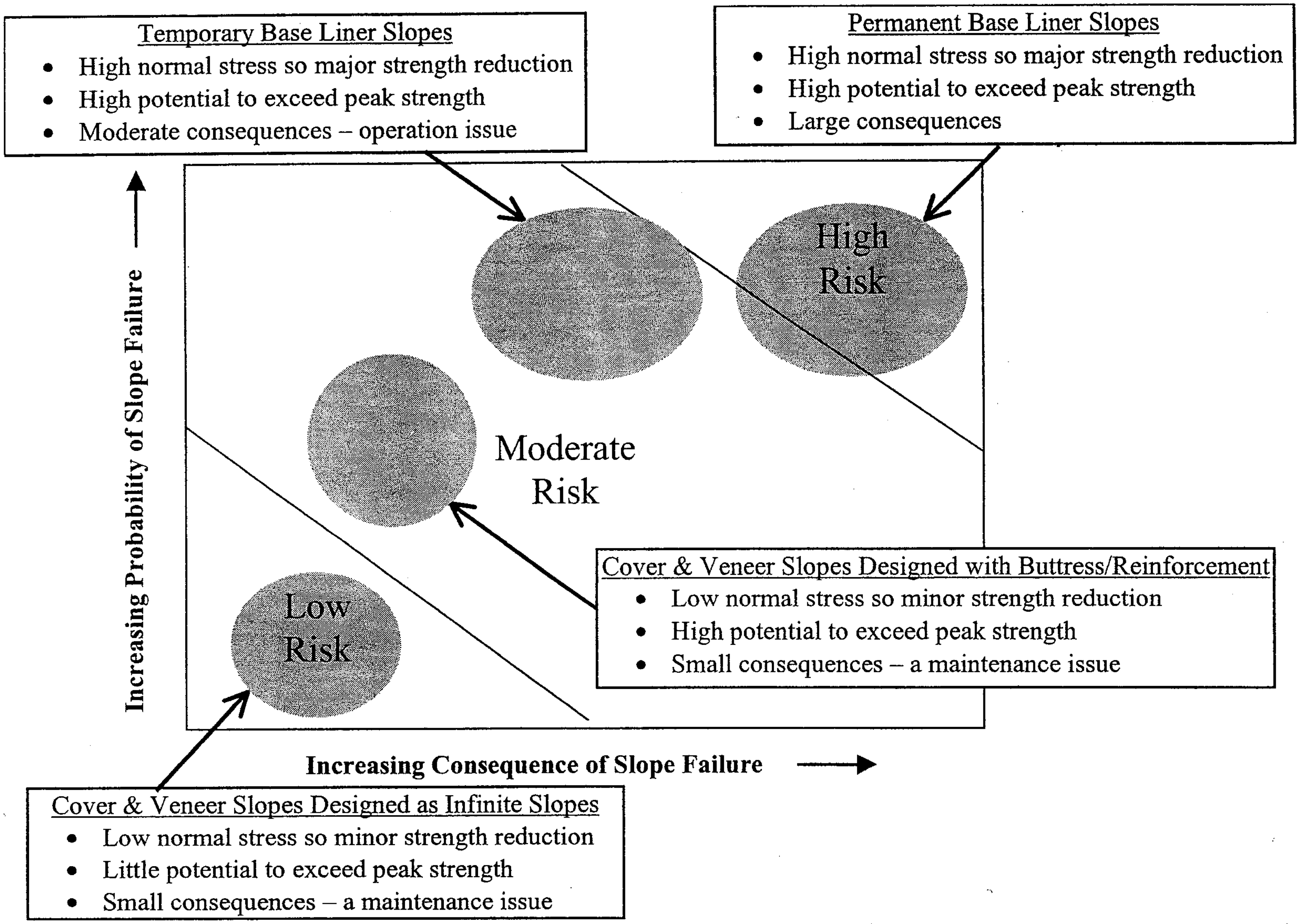

Fig. 2 Risk of Slope Failure versus Type of Slope for Slopes Designed Using FS < 1.0 with Residual Strengths

Cover or veneer slopes designed as infinite slopes (that is no reliance on a toe buttress or geosynthetic tension) will generally have the smallest risk if they are designed using a factor of safety less than one with residual strengths. First, the relative potential for strain softening is the lowest for these types of slopes. There will be a minor reduction in strength from peak to residual for most materials in cover or veneer slopes because the normal stresses acting on the slip surface are low. Also, the applied shear stresses are not likely to exceed the peak shear strength because there is little potential for deformation in the slope. The one exception might be if the slope is subjected to earthquake loading. Second, the consequences of a slope failure are small. The volume of material involved in a failure will be relatively small, and sliding is typically considered to be a maintenance problem versus a "design failure" requiring corrective action.

Permanent base liner slopes will generally have the highest risk of failure if they are designed using a factor of safety less than one with residual strengths. The relative potential for strain softening is the greatest with these types of slopes because the high normal and shear stresses acting on the slip surface typically mean that (1) there is a major reduction in strength from peak to residual and (2) the peak shear stress is exceeded by the applied shear stress at some location along the slip surface. Also, the consequences of a slope failure are large due to the large volume of material involved and the large release of energy at the onset of failure.

Table 1 provides an alternative perspective on the information in Figure 2. The higher the risk of a slope failure, the more justification that should be required to use a factor of safety less than one with residual strengths.

Table 1 Level of Justification Required to Use FS < 1.0 with Residual Strengths

Type of Slope	Risk of Failure	Justification Required
Cover & Veneer Slopes Designed as Infinite Slopes	Low	Low
Cover & Veneer Slopes Designed with Buttress/Reinforcement	Moderate	Moderate
Temporary Base Liner Slopes	Moderate	Moderate
Permanent Base Liner Slopes	High	High

Why would you accept a greater risk of failure?

There is a benefit associated with being able to use steeper slopes in the containment system - the steeper the slopes the greater the volume of air space and the greater the profit. Therefore, consideration of the risk of failure must also include consideration of the benefits: acceptance of a risk of failure means that the risk is acceptably small <u>compared</u> to the benefits.

As a simple and conservative rule of thumb, an "acceptable" probability of slope failure in the lifetime of the slope can be obtained from the following equation:

$$\text{"Acceptable" Probability of Slope Failure in Lifetime} \leq \frac{\$1{,}000}{\text{Cost of Slope Failure (\$)}}$$

Results are summarized in Table 2. This "acceptable" probability of failure gives a risk of slope failure (the probability times the cost) that is $1,000, which would generally be a manageably small risk to accept.

Table 2 Rule-of-Thumb Guidance on
"Acceptable" Probability of Slope Failure

Cost of a Slope Failure in Liner or Cover	"Acceptable" Probability of Failure in Lifetime of Slope
$10,000	0.1
$100,000	0.01
$1,000,000	0.001
$10,000,000	0.0001
$100,000,000	0.00001

Larger probabilities of slope failure could be acceptable if the benefits are deemed by all stakeholders to balance the risk.

In summary, in order to design a slope using a factor of safety less than one with residual strengths, both the probability and the consequence of the failure should be considered, discussed and accepted by all stakeholders including the owner, operator, regulator and designer.

GUIDELINE 3 – RESIDUAL STRENGTH FOR A SYSTEM IS NOT NECESSARILY THE MINIMUM RESIDUAL STRENGTH IN THE SYSTEM

A common misconception is that the residual strength for a containment system is the minimum residual strength among all components in the system. However, the residual strength can only be mobilized if the peak strength is exceeded. Therefore, the residual strength for the system is the residual strength of the component with the lowest peak strength.

An example of this concept is shown on Figure 3. This double liner system has a primary composite liner, consisting of a textured geomembrane over a reinforced geosynthetic clay liner (GCL), underlain by a leak detection system with a drainage geocomposite (nonwoven geotextiles bonded to a geonet). Direct shear tests were run on each of the possible slip surfaces: between the textured geomembrane and the GCL; within the GCL; and between the GCL and the geocomposite. The most significant strain softening occurs internally within the GCL due to failure of the reinforcing fibers at large displacements. The textured geomembrane interface with the GCL has a greater peak strength than the peak internal strength of the GCL; the failure surface moves into the GCL and the same strain softening occurs. However, the peak strength for the interface between the GCL and the drainage geocomposite is less than that for the GCL; the reinforcing fibers in the GCL are not stressed to failure and there is little strain softening. Therefore, the residual strength for this system is that for the interface between the GCL and the drainage geocomposite and not the minimum residual strength for slippage occurring internally within the GCL. In a sense, the interface between the GCL and the geocomposite protects the GCL from undergoing strain softening.

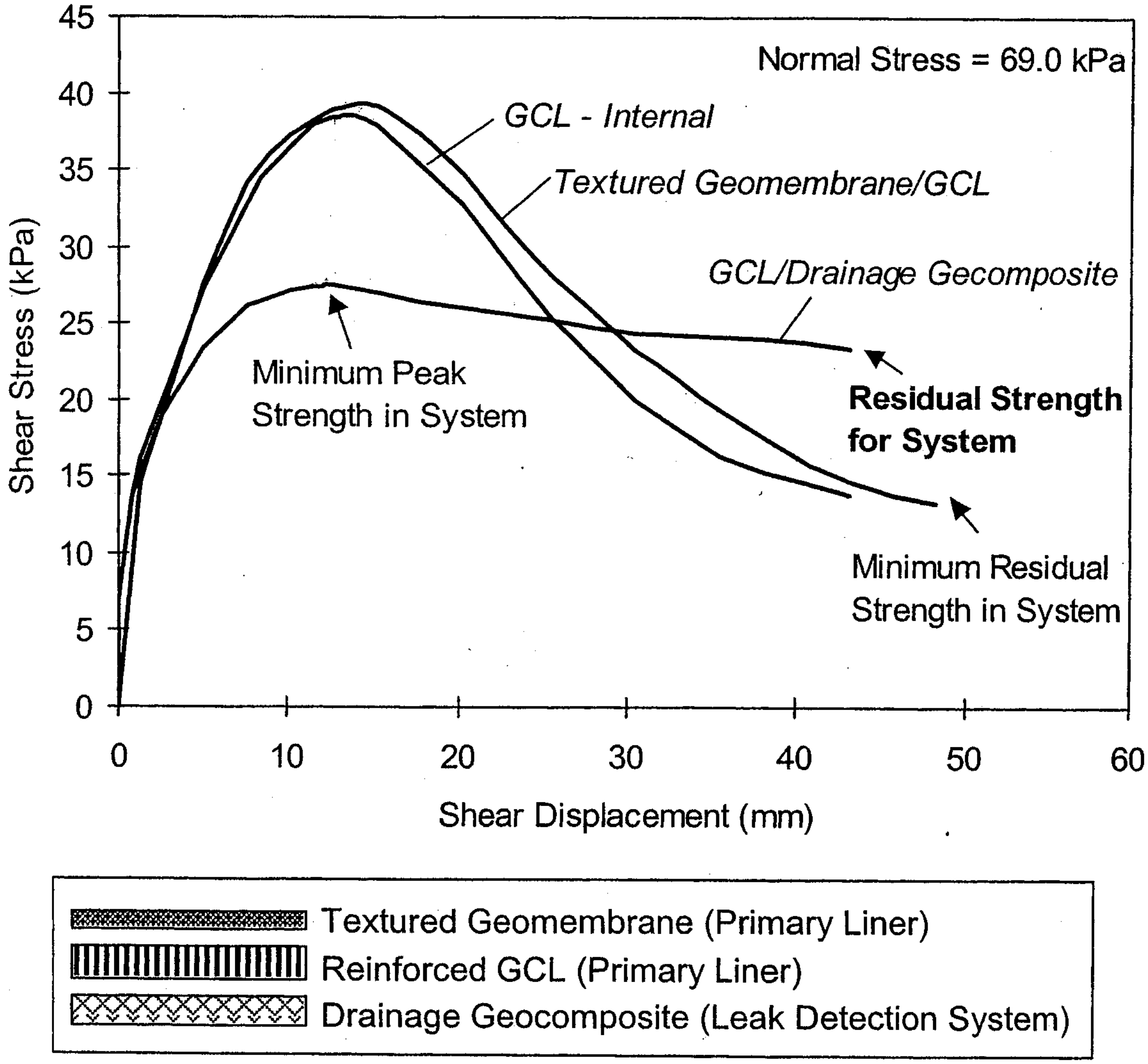

Fig. 3 Example of Residual Strength for a System (adapted from Gilbert et al. 1996)

This concept of residual strength for the liner or cover system is important. First, the residual strength for the system will be greater than the minimum residual strength in most cases. Therefore, the recommendation to design slopes using a factor of safety greater than or equal to 1.0 with the residual strength for the system is not necessarily more onerous than using peak strengths in design. In fact, in the example on Figure 3, the residual strength for the system is not much smaller than the minimum peak strength even though some of the components exhibit significant strain softening. Second, this concept is important in design because it can be used to prevent significant strain softening from occurring within the containment system even if there are components within the system that are susceptible to strain softening. Therefore, liner and cover systems can be designed to minimize the problems associated with strain softening.

GUIDELINE 4 – BOTH PEAK AND RESIDUAL STRENGTHS ARE NEEDED FOR DESIGN

Regardless of whether a slope is designed with a factor of safety greater than or equal to one using the residual strength for the system or it is designed with a lower factor of safety by considering and accepting the risks of slope failure, both the peak and residual strengths are needed for all interfaces in the containment system. The only way to know the residual strength for the system is to know the peak and residual strengths for all interfaces in the system. The only way to assess the probability and consequence of a slope failure is to know the residual strength for the system.

A common misconception is that it is difficult and expensive to estimate residual strengths. However, it is actually easier to estimate residual strengths than peak strengths. Skempton pointed this out for clay soils in 1964, concluding that residual strengths are dependent only on the nature of the particles (the mineralogy) and independent of stress history and water content (Skempton 1964).

This same principle can be applied to geosynthetic materials and to interfaces in containment systems. For example, Mitchell et al. (1990) estimated the residual strength for the interface between a nonwoven geotextile and a smooth geomembrane by polishing the surface of the geomembrane with the geotextile before running a small-scale direct shear test with continuous displacement up to 5 mm. The residual strength they measured compared very well to that obtained using a large-scale direct shear test with continuous displacement up to 50 mm (Byrne et al. 1992). As another example, Gilbert et al. (1996) showed that the residual strength for a reinforced GCL is essentially the shear strength of bentonite, for which much published information already exists (e.g., Mesri and Olson 1970). As a final example, Li and Gilbert (1999) showed that the residual strength for the interface between a nonwoven geotextile and a textured geomembrane could be obtained by shearing the interface repeatedly in a small-scale direct shear device. In fact, with this simple approach they were able to reproduce the entire shear stress versus displacement curve obtained using a larger-scale device with a much larger continuous displacement.

SUMMARY

I strongly recommended that slopes in liner and cover systems be designed using a factor of safety greater than or equal to 1.0 with the residual strength for the system. A smaller factor of safety with the residual strength should only be used if the risk of a slope failure is considered and accepted by all stakeholders.

A noteworthy consequence of this recommended approach is that estimates are needed for both the peak and the residual strengths for all components in the containment system. The peak strengths are needed to identify the location of slippage, while the residual strengths are needed to then establish the residual strength for the system. This information is always required to know what the factor of safety is with the residual strength, and to evaluate the risk if a factor of safety less than 1.0 is to be used.

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REFERENCES

Bjerrum, L. (1967), "Progressive Failure in Slopes of Overconsolidated Plastic Clay and Clay Shales," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 93, SM5, 3-49.

Byrne, R. J., Kendall, J., and Brown, S. (1992). "Cause and Mechanism of Failure, Kettleman Hills Landfill B-19, Unit IA." *Proc., ASCE Spec. Conf. on Performance and Stability of Slopes and Embankments - II*, Vol. 2, 1188-1215.

Gilbert, R. B. and Byrne, R. J. (1996), "Strain-Softening Behavior of Waste Containment System Interfaces," *Geosynthetics International*, 3(2), 181-203.

Gilbert, R. B., Fernandez, F. F. and Horsfield, D. (1996), "Shear Strength of a Reinforced Geosynthetic Clay Liner," *Journal of Geotechnical Engineering*, ASCE, 122(4), 259-266.

Koerner, R. M., Soong, T. –Y. (2000), "Stability Assessment of Ten Large Landfill Failures," *Proceedings,* Advances in Transportation and Geoenvironmental Systems Using Geosynthetics, GeoDenver 2000, ASCE, Geotechnical Special Publication No. 103, 1-38.

Li, M. H. and Gilbert, R. B. (1999), "Shear Strength of Textured Geomembrane and Nonwoven Geotextile Interfaces," *Proceedings,* Geosynthetics 99, IFAI, Boston, Mass., 505-516.

Liu, C.-N., Gilbert, R. B., Thiel, R. S. and Wright, S. G. (1997), "What Is an Appropriate Factor of Safety for a Cover Slope?" *Proceedings,* Geosynthetics 97, IFAI, San Diego, California, 481-498.

Mesri, G. and Olson, R. E. (1970). "Shear Strength of Montmorillonite." *Geotechnique,* 20(3), 261-270.

Mitchell, J. K., Seed, R. B. and Seed, H. B. (1990). "Kettleman Hills Waste Landfill Slope Failure. I: Liner System Properties." *J. of Geotech. Engrg.,* ASCE, 116(4), 647-668.

Skempton, A. W. (1964), "Long-Term Stability of Clay Slopes," *Geotechnique,* Vol. 14, No. 2, 77-102.

Stark, T. D. and Poeppel, A. R. (1994). "Landfill Liner Interface Strengths from Torsional Ring Shear Tests." *J. of Geotech. Engrg.* ASCE, 120(3), 286-293.