

PEAK VS RESIDUAL SHEAR STRENGTH FOR LANDFILL BOTTOM LINER STABILITY ANALYSES

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ABSTRACT

The decision whether to use peak or residual shear strengths for a stability analysis must be made in the context of a specific design situation. Yet even when the specific situation is defined, the decision of whether to use peak or residual shear strength is often unclear. In general, if there are potential construction, operation, or design conditions that might cause relative displacement between layers, then a post-peak or residual shear strength for the layer having the lowest peak strength is appropriate. If seismic analyses predict deformation on a given interface, then the design should use the post-peak or residual shear strength for that interface. For bottom liner systems, where stress distribution along the liner system is very complex, it is advisable to verify that the slope stability has a factor of safety greater than unity for residual shear strength conditions along the critical interface.

INTRODUCTION

This paper is concerned with the forces that support a landfill on its liner system, and the shear strength of geosynthetic interfaces that keep the mass from sliding. Figure 1 schematically portrays the shear forces that work to keep the waste mass from sliding. If sliding occurs, the surface along which sliding would occur is called the critical surface, or potential slip plane. Bottom liner systems that use geosynthetics often have their critical surface along one of the geosynthetic interfaces. The shear strength of these interfaces can usually be measured by means of laboratory testing. These interfaces often realize their peak shear strength within a small amount of relative displacement (on the order of 25 mm), after which their shear strength decreases. Typically, after 50 to 300 mm of relative displacement, the shear strength is reduced to a steady minimum value, which is called the residual shear strength of that interface. Figure 2 shows a typical shear stress-displacement curve for a geosynthetic interface.

Over the life of a landfill the following activities occur: the liner system is built; waste is placed; settlement occurs; a final cover system is installed; and settlement and degradation of the waste continues. Each of these phases of the landfill's life produces different combinations of normal and shear stresses on the liner system. Landfill leachate and gas, which can create destabilizing pore pressures, are by-products of the landfill, and are removed with varying degrees of efficiency. The primary questions addressed in this paper are:

Limit-equilibrium analyses of varying complexity that have been developed are available to design practitioners. One of the first approaches was the Ordinary Method of Slices developed by Fellenius. Later refinements were presented by Bishop, Janbu, Morgenstern and Price, Spencer, and others. A review of these methods is beyond the scope of this paper, and the reader is referred to Abramson et al. (1996) and Duncan (1996) as a starting place for a comparison of the various limit-equilibrium methods. The author would, however, offer three points from his own practice as to which method to use for performing stability analyses of bottom liner systems:

- The Bishop method is generally not applicable when analyzing bottom liner system geometries because it was developed for circular failure surfaces. The critical slip plane for liner systems is often a translational block that is non-circular.
- Spencer's method, which is now commonly available in computer codes, is considered more rigorous and complete in its analysis than the simplified Janbu method, which is commonly used for block analyses. Spencer's method is computationally more intensive, however, and may be difficult to use for random searches for a critical failure surface, even with modern computers. It is also less stable and can yield incorrect results unless the line of thrust results are checked by the user. Therefore, a good practice is to search for the critical surface using Janbu's simplified approach, and then perform a final check on the stability using Spencer's method. Usually, but not always, Janbu's method will result in a slightly higher factor of safety.
- The approach developed by NAVFAC (1982) for translational block analyses is often a good and appropriate method for performing a hand-check on the computer results for a 2-D translational block failure along a bottom liner system.

Identification of Critical Slip Plane

The most typical requirement for static stability is to meet a specified factor of safety. Just what constitutes an appropriate factor of safety will be discussed later in this paper. The idea is that if the stability analysis is performed correctly with the proper input variables, the factor of safety should provide a level of confidence that the slope will in fact be stable.

The essential operative words in the above paragraph relating to stability analyses is that they are "*performed correctly*". The safety margin in a factor of safety exists to account for unknown or unpredicted deviations from the original design assumptions. It is not, however, supposed to account for errors in the analysis, or incorrect geometric and material property assumptions.

When performing a correct analysis the critical slip plane for analysis must be identified correctly. An experienced geotechnical engineer is usually required in order to

select the critical cross-sections for analysis of a slope. Even for experienced practitioners, though, it is not always obvious which section is the most critical, and several trials generally need to be performed. For very complicated geometries, as described in the previous section, multiple 2-D sections may need to be weighted in order to simulate a 3-D analysis, or the more complex 3-D analysis can actually be performed.

In addition to selecting the proper cross-section, it is also important to search for and select the correct critical slip plane within that cross-section. In peer-reviewing slope stability analyses performed by others, the author has found errors in which the designer had correctly identified the critical cross-section, but incorrectly identified the critical slip plane within that cross-section. He found others, too, in which the designer had conceptually identified the correct slip plane, but failed to code the computer program to correctly place the slip plane at the correct interface within the liner system. The effects of such errors was to drop from an ignorantly-blissful factor of safety of 2 to 3, to an uncomfortable factor of safety of less than 1.1.

When the critical slip plane is along the liner system, the critical surface is always the one that has the lowest peak strength. If residual strengths are used in the analysis, they should reflect the surface that has the lowest peak shear strength, because that is the one that will govern deformations.

Pore Pressures

Next to gravity, pore pressures (most pervasively those caused by liquid as opposed to gas) are the single most prevalent factor contributing to slope stability failures. They are also among the most overlooked elements in slope stability analyses. Schmucker and Hendron (1998) illuminate this problem when they state that "Very little is known at this time regarding the generation and distribution of pore pressures in MSW landfills."

The one area where evaluating the influence of pore pressures on slope stability has been well focused has been in the design of dams. For this reason there have been few dam failures due to the neglect of pore pressures, with dam failures in the past century generally being caused by other factors (e.g. liquefaction or piping). Pore pressures are not commonly included in landfill analyses. Yet most (or at least many) of the dramatic landfill failures reported in the industry can be attributed to pore pressures that built up either in the foundation, due to waste loading, or in the waste itself, due to leachate buildup or leachate injection. Examples are the Rumpke landfill failure (see Schmucker and Hendron, 1998, who attributed the failure in part to leachate buildup caused by an ice dam at the toe), and the Dona Juana landfill failure (see Hendron et al., 1999, who attributed the failure to high-pressure leachate injection).

When performing slope stability analyses, designers should consider the potential for unanticipated pore pressures. Unanticipated conditions may occur in landfills due to clogging of the leachate collection systems, or aggressive leachate recirculation in the waste mass. Additional discussion of this issue is provided by Koerner and Soong (2000). Further discussion later in this paper describes how pore pressures could lead to a localized exceedence of peak strength, leading ultimately to a progressive failure.

Selecting and Measuring Material Shear Strengths

Shear Strength Definition. Figure 3 illustrates a non-linear shear strength envelope, which is typical for many soil and geosynthetic interfaces. Sometimes the non-linearity is slight, and a straight-line approximation over the entire load range under consideration can be valid. This is often true for very narrow load ranges such as those considered for cover veneer systems. At other times this non-linearity is quite significant, especially when shear strength characteristics are evaluated over the broad range of normal loads indicative of bottom lining systems.

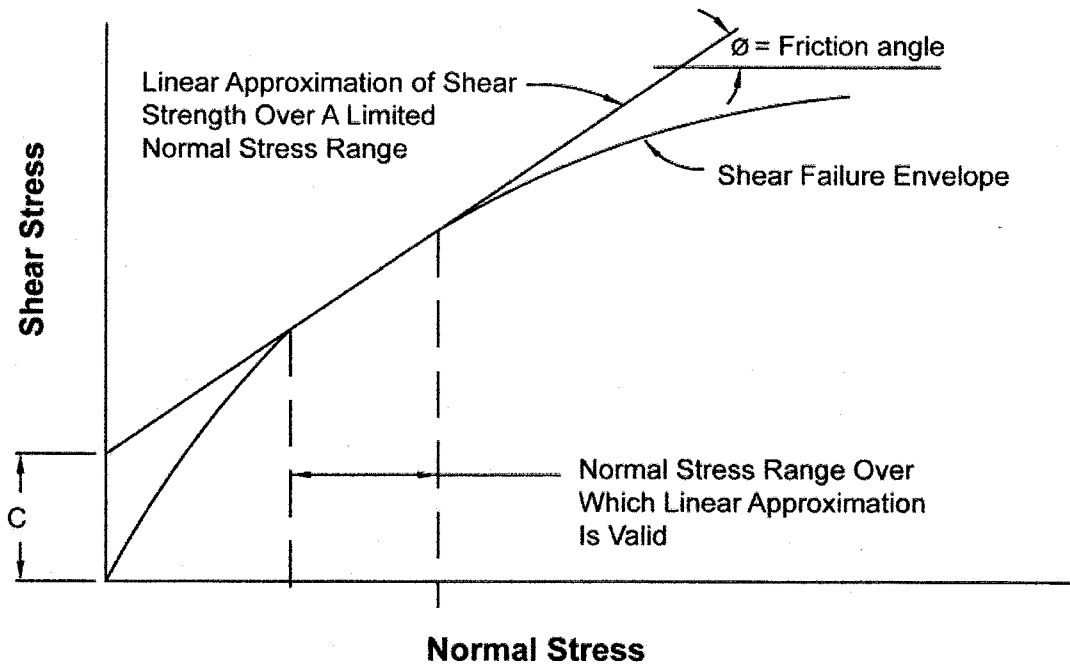


Figure 3 - Typical Shear Failure Envelope for Soil and Geosynthetic Materials.

If the shear strength curve of the evaluated materials is non-linear with respect to normal load, then special consideration should be given to defining the shear strength parameters within a specific normal load range. Many computer programs only allow the input of linear shear strength parameters. These parameters are normally identified as a friction parameter (ϕ) and a cohesion (or adhesion) parameter (c). It is useful to

recognize that these are often only mathematical parameters that describe the shear strength of a material or interface over a specific normal load range. The shear strength parameters are demonstrated in Figure 3.

Draft European Standards, and other publications (e.g. Koerner and Daniel, 1997) suggest that the apparent cohesion of a shear strength envelope can be ignored. As stated by Jones and Dixon (1998): "This assumption can have a significant effect in that the shear strength for any particular normal stress will be quoted as being lower than measured... It is possible that the failure envelope may curve to the origin at very low normal stresses, in which case ignoring the apparent cohesion will result in over conservative results." If we recognize that the values of the parameters ϕ and c are only mathematical tools used to describe the measured or estimated shear strength over a given normal load range, we can discount statements that advocate that cohesion can be ignored.

The friction parameter (ϕ) is related to the slope of the line (slope = $\tan\phi$), the cohesion parameter (c) is the y-intercept, and the normal load range is the abscissa range over which the straight-line approximation of the shear strength envelope is valid. Use of the shear strength parameters outside of the normal load range for which they were defined is generally non-conservative, as illustrated in Figure 3.

If the computer program only allows the consideration of linear shear strength envelopes, the shear strength envelope for non-linear materials should be discretized into a series of straight-line approximations for different normal load ranges. Furthermore, where the critical slip surface runs through a material or interface that exhibits a non-linear strength envelope, the designer should either use a computer code that allows input of a non-linear shear strength envelope, or assign different strength parameters to different zones of the material or interface according to the normal loading it theoretically experiences. For computer codes that do not allow non-linear shear strength envelopes, the delineation of different normal-load zones for non-linear materials is usually calculated by hand. This procedure is outlined in detail by Thiel et al. (2001).

Shear Strength Measurement. For geosynthetic lining systems, the internal and interface shear strength is normally determined by using the direct shear test in accordance with ASTM D 5321. For GCL internal and interface shear strength evaluation, direct shear testing is conducted in accordance with ASTM D 6243. In these direct shear tests, the geosynthetic material and one or more contact surfaces, such as soil or other geosynthetics, are placed within a direct shear box. The specimens are hydrated, consolidated, and placed under a constant normal load in accordance with the ASTM procedures, along with any project-specific testing clarifications/instructions from the design engineer. A tangential (shear) force is applied to the materials, causing one section of the box to move in relation to the other section. The shear force needed to cause movement is recorded as a function of horizontal displacement.

exceeded. An example of a ductile geosynthetic interface is a smooth PVC geomembrane against a geotextile (see data published by Hillman and Stark, 2001). Also, MSW waste is generally considered a ductile material in terms of shear strength (Kavazanjian, 2001).

As a progressive failure develops, the shear stresses are redistributed within the slope. This often involves the slow deformation of the failing mass over time, followed by an abrupt slide. If the critical plane supporting a slope is brittle, and for some reason part of it is stressed past its peak strength, then that part quickly becomes significantly weaker, which means it can carry less of the load. That in turn puts more of the load on other parts of the critical plane, which may in turn cause another part of that plane to become overstressed and exceed its peak strength. The continuation of this process is called progressive failure. At some point the entire system becomes overstressed and an abrupt failure occurs. This is the concern when there is a brittle interface.

Progressive failures have been characteristically noted for stiff clays, as described by LaRochelle (1989): "We have come to realize that we cannot count on the peak strength in this strain-softening material either for short- or long-term stability." Past landfill failures have been attributed to this same phenomenon (Schmucker and Hendron, 1998; Mazzucato et al., 1999; Stark et al., 2000), which holds significant potential for future failures (Gilbert and Byrne, 1996).

POTENTIAL CONDITIONS THAT MAY LEAD TO PROGRESSIVE FAILURE

Several reasons are provided below which explain why the peak strength of a bottom liner interface might unexpectedly be exceeded.

Non-Uniform Stress Distribution and Strain Incompatibility

Perhaps one of the most compelling reasons to be concerned about progressive failure in liner systems is that the stress distribution along the liner interface is not known. "It is impossible to obtain all of the necessary information in most cases" to perform a rigorous analysis of a progressive failure process (Tiande et al. 1999). "It is difficult to determine the available shear resistance along an interface exhibiting strain-softening behavior. It may be unsafe to assume that peak strength is available, while it may be excessively conservative and costly to assume that only the residual strength is available" (Gilbert and Byrne, 1996).

The complexities of stress distribution are affected by the type of loading and by pore pressures. According to Li and Lam (2001) "... the development of progressive failure will also be different depending on whether failure is triggered by a rise in water table [*insert by author: namely, leachate*] or an increase in external loading [*insert by author: namely, continued waste stacking*]".

Reddy et al. (1996) present a most interesting finite-element modeling study that evaluates the stress distribution and deformations along a landfill liner system for an assumed landfill geometry. Their study compares smooth and textured interfaces for different stiffnesses of waste. Although their analysis did not model strain-softening behavior of the interfaces, the results provide valuable insight into stress and strain distribution. Some of the conclusions from their study are:

- The stiffness of the waste influences the distribution of interface stress and shear displacements. Stiffer waste puts more stress and strain on side slopes (especially the lower part of the slope). Softer (more compressible) waste puts more stress on the base liner below the highest part of the waste, and more strain accumulation towards the toe. The overall factor of safety, however, is not affected by the waste stiffness, assuming that no strain-softening of the interface shear strength occurs.
- The smooth interface with 11° friction reached its peak strength in a number of places along the interface in their example, even though the global factor of safety was 1.5. The textured interface did not approach its peak strength anywhere along the interface in their example, but had a factor of safety of over 4. This means that a typical stability evaluation that results in a factor of safety of 1.5 may actually result in areas of the critical interface achieving their peak strength and possibly going into a reduced post-peak strength.

A finite element study was performed by Filz et al. (2001) who reached conclusions similar to those obtained by Reddy et al. (1996). Filz et al. (2001) provided a compelling demonstration that a smooth clay-geomembrane interface exhibiting strain-softening characteristics might be inappropriate to analyze based on peak shear strengths. They showed that the distribution of mobilized shear stresses was not uniform along the base and side slope, and would result in progressive exceedence of peak strength. Their comparative analyses demonstrated that whereas a limit-equilibrium analysis based on peak strengths might result in $FS = 1.6$, the finite-element analysis would suggest impending failure (i.e. $FS = 1.0$). The same problems analyzed using residual shear strengths in limit-equilibrium analyses resulted in an average $FS = 0.94$. Furthermore, for a finite-element analysis to show $FS = 1.5$, the limit-equilibrium analysis based on peak strengths needed to show a FS of about 2.2, and the limit-equilibrium analyses using residual shear strength resulted in $FS = 1.3$.

Differences in the relative stiffnesses of the overlying waste as compared to that of the liner interface are also cited by Gilbert and Byrne (1996) as a significant potential cause of deformations along the liner interface that could lead to residual shear strengths.

Similar suppositions are made by Stark et al. (2000), who postulate that strain incompatibility between MSW and underlying interfaces can lead to progressive failure, as they believe was the underlying cause of the Rumpke landfill failure. The weaker lower interfaces may achieve post-peak strengths before the MSW ever achieves peak

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strength. After peak strength of the interfaces is achieved, the peak strength of the MSW may be mobilized at a time when the strength of the interfaces is reduced to the residual value. They state: "The greater the difference between the stress-strain characteristics of the MSW and the foundation soil or geosynthetic interfaces, the smaller the percentage of [peak] strength mobilized in the MSW and underlying materials."¹

Unexpected Increases in Pore Pressure

The typical effect of pore pressures is to decrease the effective normal stress, which in turn decreases the effective shear strength, even as the shear stress that is driving instability remains unchanged. When pore pressures are introduced, the effective shear strength may be reduced to the point that the peak shear strength at that location is exceeded, at which point progressive failure can begin. This was what Schmucker and Hendron (1998) concluded was the triggering mechanism for the Rumpke landfill failure.

Seismic Loading

With seismic loading there is certainly the potential for deformation to occur along the critical failure plane, which can reduce the strength of the critical interface below its peak strength. In this regard the design practitioner needs to assess the potential for this type of deformation and, if the design earthquake is expected to produce deformation greater than about 20 mm, then the residual strength of that interface must be considered.

Construction Deformation

Construction conditions frequently result in temporary stability conditions with lower factors of safety than the completed fill scenario. To the author's knowledge, the effect of preliminary interface deformation at low normal loads on the subsequent shear strength at higher normal loads has only been documented in one recent study by Esterhuizen et al. (2001). They showed that for a smooth clay-geomembrane interface, deformations at low normal loads would partially, but not fully, reduce the peak strength of the interface at higher normal loads. They provide a very interesting "work-softening" model to describe this behavior in a manner that can be used in a finite-element analysis. Although their model fits the data very well, it is only applicable to the specific clay and geomembrane used for their study, and it is not known at this time how well their approach would work for other interfaces. This is an area for further research.

¹ For years now the author has heard the statement that the strain incompatibility between waste and liner systems could be a major consideration in selecting appropriate shear strengths. It is interesting, however, that some of the literature reports surprisingly low amounts of deformation required to reach the peak strength of the waste; on the order of only 40 mm for rigid-body deformation. See, for example, Eid et al. (2000), Stark et al. (1998), Mazzucato et al. (1999). Also Kavazanjian (2001) states his belief that strain compatibility with MSW is not nearly as significant an issue as has generally been supposed, based on direct- and simple-shear test results that show that the strains and deformations required to reach peak strength are comparable to those required for most soils.

- Relative differences in stiffness between waste and liner materials.
 - Unexpected pore pressures.
 - Seismic loading.
 - Deformation during construction.
 - Waste settlement.
 - Foundation settlement.
 - Aging and creep of the geosynthetics.
- Exceedence of peak strength in a brittle interface can result in progressive failure.
 - Based on field observation, most facilities designed with aggressive interface shear strengths are not experiencing post-peak shear strength, which means that the working shear stress is probably less than or equal to the peak strength. Only a few examples of progressive failure along geosynthetic interfaces have occurred in the industry, and these have not been along highly brittle interfaces, which means that the projects did not have high factors of safety to begin with, even assuming peak interface strengths.
 - Several design approaches have been used over the years and the standard-of-practice is evolving. In the United States a preferred approach has not yet clearly emerged.

RECOMMENDATIONS FOR PRACTICE

- Designers and CQA firms should conduct material-specific testing of interfaces to verify that the materials specified and/or supplied for a project are realistic and meet the design requirements. Whoever commissions the testing should possess a skilled familiarity with the design objectives as well as the testing technique.
- Designers should attempt to position the critical slip plane above the primary geomembrane to the extent feasible for a given project. If a double-sided textured geomembrane is required for construction or operational stability, attempt to specify more aggressive texturing on the under side of the geomembrane.
- Using peak shear strengths on the landfill base, and residual shear strengths on the side slopes appears to be a successful state-of-the-practice in many situations.
- Designers should consider evaluating all facilities for stability using the residual shear strength along the geosynthetic interface that has the lowest peak strength. This would be an advisable risk-management practice for designers, even if the FS under these conditions is simply greater than unity.

- Regardless of the design assumptions, specify soil spreading by pushing up-slope only, and require close monitoring of LCRS and operations soil placement on slopes during construction to verify that relative shear displacement does not occur during construction. Exceptions to this practice should be allowed only with field tests and CQA verification.
- If LCRS or operations soils are placed as part of landfill operations, designers should assume the worst and automatically assume residual side-slope shear strength conditions will occur (and extra leakage rates as well). The reason for this is that construction by landfill operators is usually not controlled and monitored closely.
- Check stability for a potential leachate buildup, especially near the toe of the landfill.

RECOMMENDATIONS FOR FURTHER RESEARCH

- More finite element analyses at an academic level, such as those performed by Reddy et al. (1996) and Filz et al. (2001) would be warranted, to gain a better understanding of the threshold beyond which localized stress distributions might cause exceedence of peak shear resistance. Refinements in the analyses would include modeling the strain-softening behavior of the geosynthetic interfaces, and checking different types of interfaces and geometries. The results of these analyses might prove useful for establishing guidelines as to when peak strengths might be exceeded and when they might be maintained. Ultimately, the author envisions correlations between the FS determined by limit equilibrium analyses, ratios of peak interface strengths to waste fill strengths, and relative stiffnesses (somewhat as proposed by Gilbert and Byrne (1996), but more specific and less general), being used to estimate when and where peak vs. post-peak strengths would be reached at the interfaces.
- The monitoring of slope deformation on geosynthetic interfaces that are being buried by waste is recommended. One fairly easy way to do this would be to use the simple tell-tale technique employed for the Cincinnati cover demonstration project (Koerner et al., 1996), though this would require participation by landfill owners and operators. This avenue of research echoes that suggested by Gilbert and Byrne (1996), who state: "Future research should focus on measuring deformations and mobilized shear resistances in existing waste containment facilities."
- The monitoring of pore pressures in the LCRS above liner systems, with the reporting of the worst-case conditions, would provide valuable information regarding long term conditions in landfills. Unfortunately, any high pressures would likely result in a permit violation at many facilities, so it is improbable that