GSE
GundSeal
The GCL Composite Lining Solution

The GSE GundSeal GCL Design Manual

From June 1999 through October 2001, GSE in conjunction with industry design and academic professionals developed the industry's first GCL design guidance document, the GSE GundSeal Design Manual. This comprehensive 370 page design manual features design methodologies and procedures for utilizing the GSE GundSeal geomembrane supported GCLs in a wide range of composite liner (geomembrane-clay) applications. It presents state-of-the-practice design principles related to hydraulic performance evaluation, slope stability analysis, construction issues, and durability issues in utilizing GSE GundSeal and GCLs in bottom liner systems, caps, ponds, and secondary containment lining applications.

The design methodologies developed and data presented are based on the input and expertise of industry professionals, including Richard Thiel (Thiel Engineering), David Daniel (University of Illinois at Urbana), Richard Erickson (GSE Lining Technology, Inc.), Ed Kavazanjian (GeoSyntec Consultants) and J.P. Giroud (J.P. Giroud, Inc.). GSE would like to acknowledge the co-authors for their support and creativity in preparing this state-of-the-practice design guidance document.

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FOREWORD

The information contained in this design manual was compiled by the joint efforts of several individuals, and sponsored by GSE Lining Technology, Inc.

The scope, organization, and content of the manual were generally conceived by Richard Thiel, Thiel Engineering, and Richard Erickson, GSE Lining Technology. Richard Thiel generated most of the material for the chapter on slope stability (Chapter 3), the chapter on installation (Chapter 4), and the applications related to bottom lining systems, cover systems, and surface impoundments (Chapters 5, 6, and 7). Dave Daniel, University of Illinois at Urbana, generated most of the material regarding the hydraulic theory (Chapter 2) that was subsequently used in design approaches in the applications (Chapters 5 through 8). Richard Erickson developed the material related to the application of secondary containment (Chapter 8). Ed Kavazanjian, GeoSyntec Consultants, provided all input related to seismic stability referred to in various chapters (primarily Chapter 3) and Appendix G. J.P. Giroud, J.P. Giroud, Inc., provided derivations and rational design approaches for complicated hydraulic issues related to intimate contact and leakage in encapsulated designs utilizing GundSeal (Chapter 2, and Appendices D and E).

A number of reviewers provided comments and contributions that are gratefully acknowledged. In particular, valuable technical and organizational comments were received by Greg Richardson (G.N. Richardson & Assoc., Raleigh, North Carolina). Additional peer review by industry professionals were contributed by:

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Mike Driller, California Dept. of Water Resources, Sacramento, California
Bob Gilbert, University of Texas, Austin, Texas
Bob Mackey, S2Li Inc., Maitland, Florida
Stefan Melchior, melchior + wittpohl Ingenieurgesellschaft, Hamburg, Germany
Rob Swan, SGI Testing Services, Atlanta, Georgia

Contributions from GSE staff included: the execution of the manual and its format, figures, and CD-ROM attributed to Jackie Nguyen and Adrian Baxter; preparation of design details performed by Don Sharkey; and technical review performed by Ed Zimmel, David Vieraitis, and Yong Prachoomdang. Several other GSE staff contributed to the final presentation of the manual as well.

The authors and contributors to this manual hope it provides a useful document for state-of-the-art and practice approaches to designing with geosynthetics in general, and GundSeal in particular. As with any practical design guide, future innovations and understandings are bound to supercede what was once documented as the leading-edge. Design practitioners are encouraged to stay current with the state of the practice as it evolves, and to contact GSE if they perceive useful modifications or additions for this manual.

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Chapter 3
GUNDSEAL DESIGN FOR SLOPE STABILITY

Stability is a concern whenever a slope is built that will challenge the shear strength of the materials within and below it. An evaluation of whether a slope will remain stable or not requires an understanding of the slope geometry, the unit weights and shear strengths of the materials within and under the slope, pore pressures that may be caused by liquids and gases, and external loadings such as vehicles or seismic events. The practice of evaluating the interim and global stability of slopes is usually performed by civil engineers with specialization in geotechnical engineering.

Geosynthetics, such as geomembranes and GCLs, often provide a preferential slip plane along which a slope failure may occur. Landfills and containment impoundments are good examples of engineered structures that require slope stability evaluations specifically focused along geosynthetic and soil interfaces. The containment bottom lining system considers slope stability of the waste mass along the bottom of the liner system. A cover system considers veneer slope stability of the cover soil layers on top of the liner system. Surface impoundments incorporate additional hydraulic and buoyant forces acting on the lining system created from elevated liquid levels. These stability concerns focused on bottom liner and veneer cover systems are shown schematically in Figure 3.1.

Figure 3.1a Bottom Liner Slope Stability Concerns.

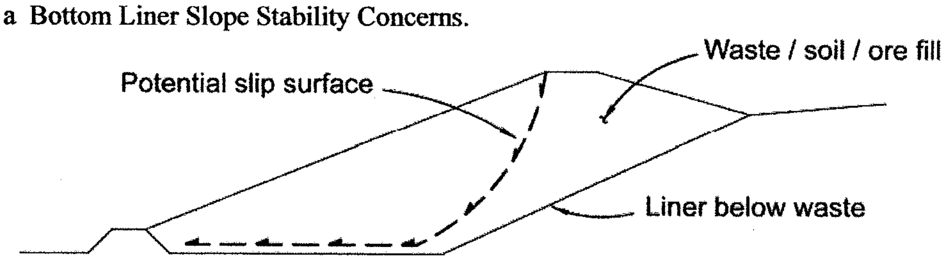

Figure 3.1b Veneer Soil Cover Slope Stability Concerns.

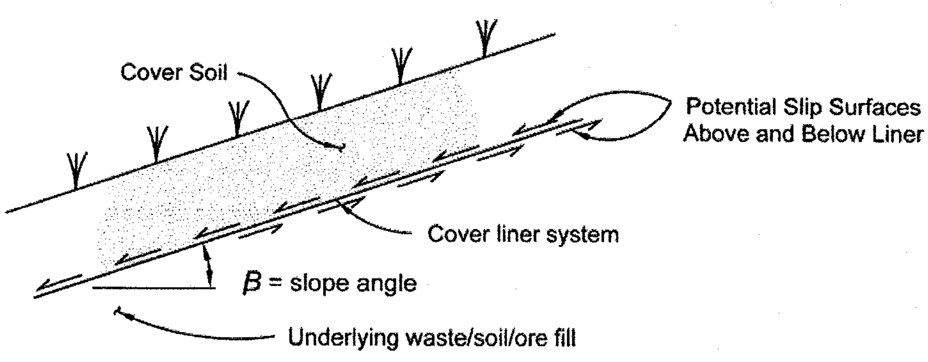

Figure 3.1 Stability Concerns Related to a) Bottom Liners and b) Veneer Cover Systems.

The basic geotechnical principles used to evaluate the slope stability of these different configurations are the same. It is useful to consider them separately, however, primarily because the magnitudes of the forces are much different. Bottom liner systems are typically under a wide range of relatively high-normal loads compared to veneer cover systems, which are typically under a narrow range of lower normal loads. The project-specific range and distribution of normal loads has a significant effect on the shear strength parameters to be used for many materials. For example, the global stability in bottom liner systems in many instances is derived from the resistance in the base of the facility and not from the side slopes. Therefore, in the case of bottom liners, improving stability is often a function of increasing the resistance along the base of the lining system versus the side slopes. In the case of veneer covers and very low normal loads, stability is particularly sensitive to relatively small changes in fluid pore pressures caused by such events as rainfall or landfill gas buildup below the cap liner.

The remainder of this chapter will discuss slope stability considerations when designing with GundSeal for containment bottom liners and veneer systems. The discussion will be focused on the shear strength parameters related to the bentonite coating of GundSeal. Although not specifically discussed in this manual, the adjacent geomembrane and other soil and geosynthetic interfaces should be considered in a similar manner, as these areas typically present the most critical materials and interfaces for slope stability analyses.

3.1 Slope Stability Considerations

3.1.1 *Bottom Liner Stability Analysis*

Slope stability analysis is most commonly assessed using a computer analysis that evaluates the limit equilibrium of a two-dimensional cross section. Less sophisticated analyses can be performed using hand-calculation methods or charts. Hand calculations are an effective analysis tool because they often provide a clearer understanding of the critical aspects of the problem, and mistakes in geometry and assumed failure planes are less likely. Hand analyses are often performed for a simple geometry, such as the cover veneer systems described in Chapter 6 and surface impoundment liners described in Chapter 7. Hand analyses are time-consuming for more global deep-seated failure scenarios, however, and many iterations are often required to locate the most critical slip planes. A common approach is to provide a hand check on one of the most critical surfaces that has been analyzed by a computer program. A good summary of slope stability using hand calculation approaches is provided by Abramson et al. (1996).

The basic steps for performing a slope stability analysis are:

1. Determine the geometry of the slope surface and the subsurface profiles of the different materials and interfaces that will affect the analysis. Select the two-dimensional cross sections that appear to be the most critical. This requires judgment and experience. The process usually involves overlaying a drawing showing the bottom liner contours with a drawing showing the fill contours. Combinations of high and/or steep outer slopes with the most unfavorable bottom liner slopes are typically chosen for evaluation. There may be several different possibilities that could produce the most critical section. Therefore, if in doubt, it is prudent to analyze several different sections.

2. Determine the properties of all materials and layers that will affect the analysis. The most important properties are usually material unit weights and shear strength parameters.

3. Determine the pore pressures that may exist or develop, including both liquid and gas pressures.

4. Assess the stability of the slope using the computer program. A special note of caution is warranted in that the search for the critical failure surface(s) requires inputting correct modeling constraints into the computer program. Experience and judgment are essential to determine where and how to ask the program to perform its search in analyzing critical interfaces and materials. Incorrect searches for the critical surface can lead to completely erroneous results. A complete discussion of these topics is beyond the scope of this manual.

As discussed previously, geosynthetics utilized in bottom liners and cover systems often provide a preferential slip plane or interface that may define the critical surface within a slope stability analysis. Therefore, the type of slope stability analysis often used with typical lining and cap containment systems is called a "block" analysis. A generic schematic of this type of analysis is presented in Figure 3.2. In this case, the central block of the containment facility profile is on a critical surface defined by the bottom lining system. The active and passive wedges of the sliding mass may have slip surfaces that go through the waste/fill or, at the toe, through native materials adjacent to the containment facility.

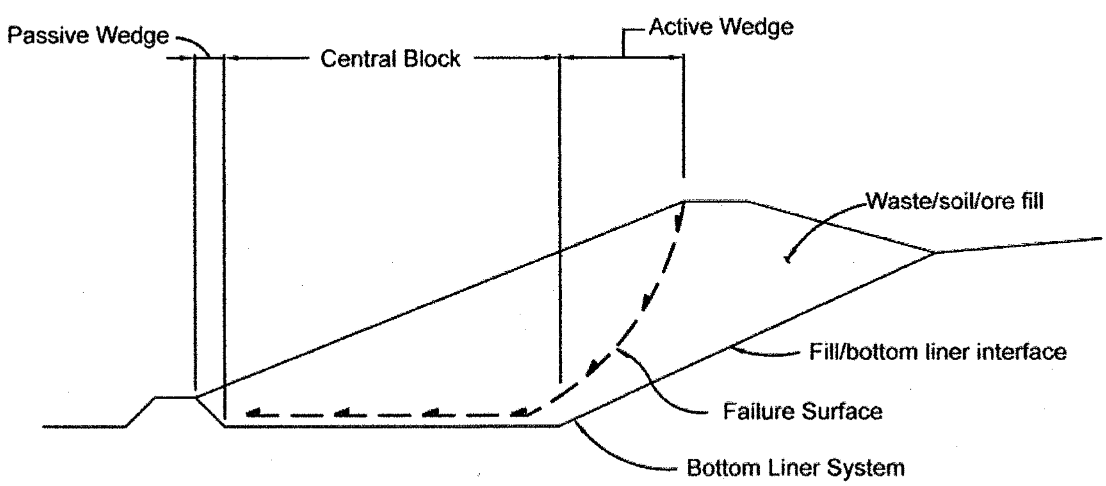

Figure 3.2 Typical Bottom Liner and Cover System Block Stability Analysis.

Sometimes the entire critical slip surface is along the lining system. The labels "central block", "active wedge", and "passive wedge" are actually nomenclature borrowed for illustration purposes from a simplified hand method for performing this type of slope stability analysis (Abramson et al., 1996; and NAVFAC, 1982). Two-dimensional limit-equilibrium computer programs, which are the most commonly used methods of analyses, generally use a variation of the method of slices.

The most popular methods of limit equilibrium used in computer analyses of landfill liner systems are the *Spencer Method* and *Simplified Janbu Method* of slices. Spencer's method, which satisfies both force and moment equilibrium, is generally considered the most accurate when executed properly. The Simplified Janbu method generally yields slightly more conservative values for the factor of safety. Since significantly more computation time is required for analyses of potential failure surfaces using Spencer's method, the most efficient practice commonly used by design engineers is to first investigate a number of potential failure surfaces using Janbu's method. The failure surfaces for analysis are selected using computer-generated random surface generation techniques. Once critical potential failure surfaces have been identified, they can then be analyzed using Spencer's method. The reasonableness of the Spencer solution should then be evaluated through examination of the line of thrust calculated by the computer program.

Note that the Simplified Bishop method of slices, which is popular in stability evaluations of earthen slopes, is generally not appropriate for evaluating landfill stability on liner systems because this method is only appropriate for circular failure surfaces.

More sophisticated analyses can be performed that consider three-dimensional effects, and finite element analysis techniques can be employed as well. *Three-dimensional analyses* have often been used on forensic studies for back-calculation of failures (e.g., Stark and Eid, 1998). *Finite element analyses* have been used primarily to assess stress distributions and deformations in slopes (Duncan, 1996).

While both of these more sophisticated approaches are valid, they require considerably more time and effort than two-dimensional analyses. Their increased complexity and the need for more refined data input generally does not justify their use for a typical landfill, cover, or pond project design. Even for a complex geometry, analysis of numerous two-dimensional cross sections is usually sufficient to assess slope stability.

Properly performed two-dimensional analyses will always give a lower factor of safety (e.g., be a more conservative design) than three-dimensional analyses (Duncan, 1996). Therefore, the guidance and suggestions presented in this manual for performing slope stability analyses on bottom lining and cap systems are geared towards methods that use two-dimensional limit-equilibrium computer programs. Two-dimensional stability analyses software programs available to design engineers include UTEXAS3 (Shinoak Software, 1996), SLOPE/W (Geo-Slope International, 1996), and STABL [(Siegel, 1975) later updated to PCSTABL5 (Carpenter, 1986)]. Data and recommendations presented in this manual are based on two-dimensional stability analyses performed utilizing the PCSTABL5 analysis program.

3.1.2 *Factor of Safety*

The factor of safety *(FS)* against sliding is generally defined as:

$$FS = \frac{\text{Shear strength along the slip surface}}{\text{Shear stress along the slip surface}}$$

The shear strength at any point along the slip surface is a function of the effective normal stress (the total normal stress minus the pore water pressure) at that point. The computer program typically divides a two-dimensional cross section into a series of vertical slices, with the base of the slices positioned along the assumed slip interface. The program then calculates the average normal stress at the base of each slice. Using user-defined shear-strength parameters for each of the materials, and user-input pore pressures for each slice, the shear strength along the slip interface is then calculated. The shear stress required for equilibrium is subsequently calculated by the computer program. The ratio of the estimated shear strength to the shear stress required for equilibrium is then presented as the factor of safety.

This relatively straight-forward process becomes more complicated during earthquake loading. When a low permeability soil interface is saturated, the shear strength of the interface does not change in response to changes in the normal stress induced by the earthquake load because the duration of transient loading is inadequate time to relieve excess pore pressures. Under these conditions, the interface must be assigned an "undrained" shear strength that is a function of the pre-earthquake equilibrium normal stress.

3.1.3 *Shear Strength Definition*

Figure 3.3 illustrates a non-linear envelope, which is typical for many soils and geosynthetic interfaces. Sometimes the non-linearity is slight, and a straight-line approximation over the entire load range under consideration is valid. This is often true for very narrow load ranges such as those considered for cover veneer systems. Sometimes the non-linear shear failure curve is very significant, such as for GundSeal when its shear strength characteristics are evaluated over a broad range of normal loads indicative of bottom lining systems.

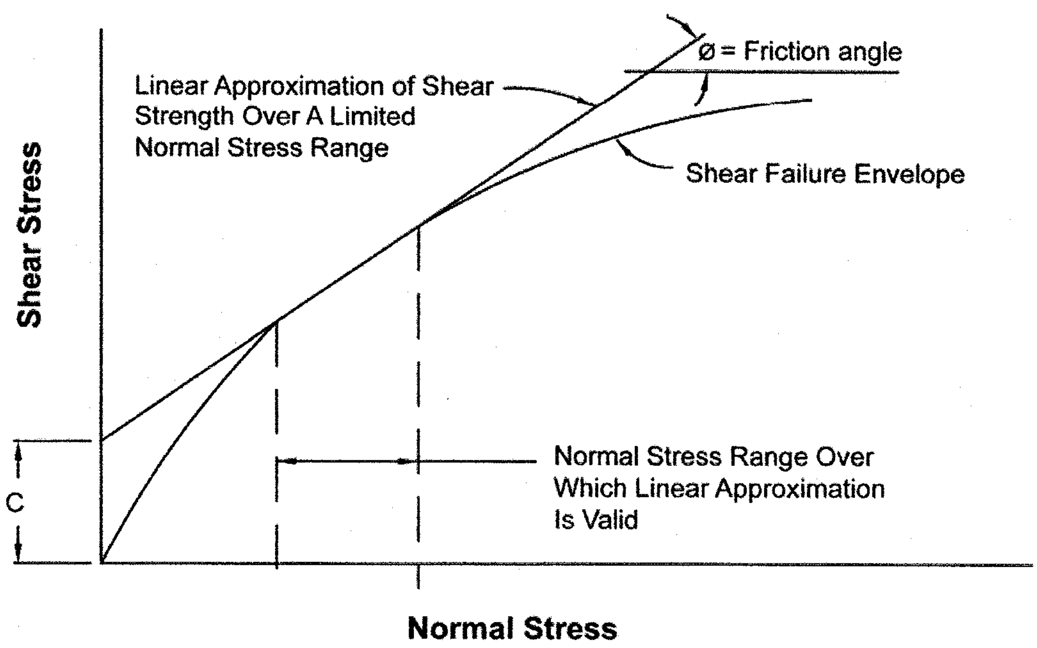

Figure 3.3 Typical Shear Failure Envelope for Soil and Geosynthetic Materials.

If the shear strength curve of the materials evaluated is non-linear with respect to normal load, then special consideration should be made regarding definition of the shear strength parameters within a specific normal load range. Many computer programs only allow linear shear strength parameters to be input. These parameters are identified as a friction parameter (normally referred to as ϕ or δ) and a cohesion (or adhesion) parameter (normally referred to as *c* or *a*). 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 3.3.

The friction parameter is related to the slope of the line (slope = $\tan\phi$ or $\tan\delta$), the cohesion parameter 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 3.3.

Most available two-dimensional computer programs for slope stability allow for the entire envelope (linear or not) to be input as a series of points along the shear strength envelope. If the computer program only allows consideration of linear shear strength envelopes (which is a common situation), 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 user should assign different strength parameters to different zones of the material or interface according to the normal loading it theoretically experiences. In a given geometric cross section, the delineation of different normal-load zones for non-linear materials is usually calculated by hand. The example in Appendix F utilizing PCSTABL5 (Carpenter, 1986) stability analysis software illustrates this method in detail.

For the special case of a saturated low permeability soil (e.g., bentonite) interface subject to seismic loading, the shear strength of the interface is characterized by a friction angle of zero and a cohesion equal to the undrained shear strength. The undrained shear strength is evaluated based upon the pre-earthquake normal stress and the shear strength envelope described in the previous paragraphs. If the pre-earthquake normal stress varies along the interface, the interface must be broken up into segments of relatively constant normal stress, each of which is assigned the appropriate undrained strength. The example in Appendix G illustrates the application of the undrained shear strength concept in a seismic stability analysis.

3.1.4 *Shear Strength Measurement*

For geosynthetic lining systems, the internal and interface (i.e. friction resistance) shear strength is normally determined 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.

The test is normally performed for several different normal loads, typically a series of three individual tests at specified normal load conditions. The normal load and shear forces are converted to stresses by the given area over which shear occurred, typically a 12 in x 12 in (300 mm x 300 mm) sample. The peak and post-peak (or residual) shear stresses are plotted on a graph, and a best-fit straight line or curve is fit through the data to represent the shear strength envelope.

Several factors can influence the shear strength of GundSeal as well as any other available bentonite based GCL products. The most important factors are discussed below.

3.1.4.1 Rate of Shear Displacement. The typical default shear rate for direct shear testing with geosynthetics as presented in ASTM D 5321 is 0.04 in/min (1.0 mm/min). For testing hydrated GCLs, ASTM D 6243 provides guidance on attaining consolidated drained conditions that should avoid the build-up of excess pore pressures.

The effect of the shear displacement rate on the shear strength of the sodium bentonite layer of GundSeal was evaluated by Eid and Stark (1997) at a normal stress of 355 psf (17 kPa) under both dry and hydrated conditions. Their results indicated that shear rates do not affect the residual shear strength of GundSeal in either the dry or hydrated state. They also do not affect the peak strength of dry GundSeal as long as the shear rate is less than or equal to 0.04 in/min (1 mm/min). The shear rate appeared to affect the peak strength of hydrated GundSeal, however, with decreased shear rates resulting in lower shear strength. Their slowest test shearing rates were at 0.0006 in/min (0.4 mm/min).

All of the dry shear strength data reported in this chapter were created at shear rates less than or equal to 0.04 in/min (0.10 mm/min). Most of the available data regarding the shear strength of hydrated sodium bentonite was performed at a shear rate less than 0.004 in/min (0.1 mm/min). Additional information regarding shear rates that were used to develop shear strength envelopes for GundSeal is presented in Section 3.2.

Neither this strain rate nor the "fast" strain rate typically used in practice is representative of the high strain rates associated with earthquake loading. At the present time, the higher shear strength that would be expected for earthquake-like strain rates is considered an unquantified additional margin of safety in seismic design. An engineer wishing to take into account the increased shear strength associated with earthquake rates of loading or wishing to quantify this additional safety margin should conduct his own laboratory test program at the appropriate strain rate and confining pressure.

3.1.4.2 Bentonite Moisture Content. Daniel (1993a) demonstrated that for normal loads up to 3,000 psf (150 kPa), the shear strength of bentonite is highest at a moisture content of approximately 35%. At a water content of 50%, the shear strength is similar to that at full hydration. Therefore, sodium bentonite is typically considered hydrated at a moisture content of 50% and above.

The dry shear strength data reported in this manual were developed on GundSeal in an as-received condition, which is typically a moisture content of 25%. The hydrated shear-strength data presented were from tests performed on samples that had been fully hydrated. For designs utilizing GundSeal as a one-product composite (geomembrane-bentonite) liner, the hydrated shear strength should be used for design purposes. For designs using GundSeal in an encapsulated mode (geomembrane-bentonite-geomembrane), the designer may wish to use either a prorated shear strength (for static design) or a prorated yield acceleration (for seismic design), as discussed in Section 3.3.

3.1.4.3 Normal Stress. The most common strength-related errors in computer slope stability analyses stem from using strength parameters that do not correspond to the normal load conditions and resulting stress level at the surface being analyzed (Lambe et al., 1989). It is generally unconservative to extrapolate linear strength envelopes beyond the limits for which they were defined. It is, therefore, important that shear test data be acquired under normal loading conditions representative of the conditions being analyzed.

The shear strength envelopes presented in this chapter for GundSeal are non-linear over the normal load range from zero to 28,000 psf (1,400 kPa). Both linear equations and non-linear hyperbolic equations are shown for the curves that were fit to the data. The example of slope stability analysis described in Section 5.4.3 illustrates how the non-linear shear strength envelopes can be discretized into linear segments over defined normal stress ranges.

3.1.4.4 Location of the Shear Plane. For the GundSeal product, the shear failure interface can occur either along the geomembrane/bentonite interface or through the bentonite itself. Most of the available direct shear data presented and used in this manual for high-normal loads were derived from testing performed in 2001 on GundSeal manufactured with a well-textured HDPE geomembrane backing. Most of the direct shear data reported in the literature for low-normal loads, which are also presented in this manual, were obtained from direct shear testing on GundSeal manufactured with either a smooth-surfaced HDPE geomembrane backing or with a geomembrane less-textured than was used to generate the 2001 test data.

Direct shear test observations indicate that a well-textured geomembrane fully mobilizes the shear strength of the bentonite in GundSeal. The shear failure plane is typically located in proximity to the peaks of the textured surface of the geomembrane, and strength is controlled by the internal shear strength of the bentonite.

In general, where shear strength and slope stability is of concern, a well-textured geomembrane backing should be specified for GundSeal. The list of GundSeal products is presented in Appendix B which outlines the available HDPE and LLDPE smooth and textured geomembrane backings. The standard textured geomembrane backing for GundSeal is 30 mil (0.75 mm) HDPE with thickness ranging up to 80 mil (2.0 mm) for composite liner applications.

For purposes of maximizing the interface shear strength between the bentonite and a textured geomembrane, the definition of "well-textured" means that the geomembrane has an asperity height of at least 15 mil (0.4 mm) measured in accordance with the test method GRI-GM12.

3.1.4.5 Peak vs. Post-Peak vs. Residual Shear Strength. The highest level of shear strength measured in a direct shear test under a given normal load is defined as the peak strength. With continued shear displacement, there is typically a loss of strength. The shear strength at any given displacement past the point of peak strength is referred to as "post-peak strength". The strength at which there is no further strength loss with continued displacement is called the "residual strength". Many of the most common direct shear devices do not allow enough displacement to occur to measure true residual strength (e.g., see Stark et al., 1996). Therefore, it is usually not technically correct to refer to end-of-test conditions as representing the "residual" strength, but it is more correct to refer to that as "post-peak" strength and specify the amount of displacement.

In the case of GundSeal, Eid and Stark (1997) have shown that the post-peak displacement usually achieved in many common direct shear boxes of 2.0-2.2 in (50-60 mm) is adequate to realize residual shear strength for the bentonite portion of GundSeal in both the dry and hydrated state. Therefore, the post-peak shear strengths referenced in the remainder of this manual for GundSeal will be referred to as residual.

Residual strengths are relatively certain because, unlike peak strengths, small variations (such as moisture content and shear rate) have very little impact on the true residual strength. When the residual strengths are mobilized on a slope with a factor of safety near one, any deformations will not occur quickly. Even if the actual *FS* is slightly less than one due to differences in geometry or loading conditions between the analysis and the actual slope, the slope will not catastrophically fail. The slope can be monitored and subsequent measures can be taken to reduce the deformation rate if deemed necessary. Regardless of how high the *FS* is with peak strengths, if the *FS* with residual strengths is significantly less than one (such as 0.8 or 0.9), there is the potential that the slope will fail suddenly and undergo large and possibly catastrophic deformations. Therefore, in addition to analysis assuming engineering design criteria and project specific slope conditions, a 'worst case' stability analysis should also be conducted to verify hydrated conditions and residual shear strengths on critical slopes produces a $FS \geq 1.0$.

The decision as to whether to use peak, post-peak, or residual shear strength for any given slope stability analysis is up to the designer. In general, if there are potential construction, operation, or design conditions that might allow 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, for example, then the design should use the post-peak or residual shear strength for that interface. In general, for geosynthetic liner systems that exhibit a significant post-peak strength loss (e.g., "brittle" materials), as discussed by Gilbert and Byrne (1996), it may be advisable to verify that the slope stability has a safety factor greater than unity for residual shear strength conditions on the critical interface.

3.1.4.6 Hydration Liquid. Most of the shear strength data for sodium bentonite and GCL products reported in the literature or in this manual were generated with tap water as the hydrating liquid. Limited direct shear test data with normal loads up to 1,440 psf (69 kPa) utilizing various hydrating fluids by Leisher (1992) indicate no significant difference in the shear strength of GundSeal whether the hydrating liquid is tap water, distilled water, or typical landfill leachate.

3.1.5 *Technical Issues Related to Veneer Cover Stability Analysis*

The following characteristics should be considered in design for static slope stability of veneer systems.

a. The failure of veneer covers can be due to two potentially critical slip planes: above and below the geomembrane. They need to be checked independently. The major differences in calculating the factor of safety for these two surfaces, separated by only a geomembrane, is how pore pressures are calculated, and the potential tensile forces in the geosynthetics above the potential failure plane.

b. The critical condition for a failure plane above the geomembrane is usually that in which the cover soils are saturated. This can occur from intense or prolonged precipitation. In this case, the soil cover on the sideslopes would be fully or partially saturated, and may experience pore pressure effects that tend to reduce slope stability. Lateral drainage layers are incorporated in cover system designs to control the amount of pore pressure buildup that may occur.

c. The effectiveness of drainage layers can have a critical influence on slope stability. In particular, if the drainage layer has inadequate capacity, becomes plugged or is unable to discharge through an outlet, the design assumption of drainage is invalid, and the slope may not be stable.

d. Cover slopes are finite in height, and therefore passive resistance at the toe of the slope can be a significant resisting force that should be considered in the analysis. In the same vein, the use of a tapered soil cover, where the soil is thicker at the toe of the slope than near the crest, is a good design solution to increase stability. Tapered covers may not be practical for long cover slopes, and would become more practical the shorter the slope length.

The basic steps for performing a static slope stability analysis for a veneer cover are:

1) Determine the geometry of the critical side slope. This will typically be one of the highest and/or steepest of the slopes.

2) Determine the properties of all materials and layers that will affect the analysis. The most important properties are usually unit weights and shear strength parameters. The most important unit weight is that of the cover veneer soil. The saturated unit weight is the most critical.

3) For a potential failure plane on top of the GundSeal, the critical shear strength is the interface between the soil and the GundSeal geomembrane. Textured geomembrane can be specified for the side slopes to increase this interface strength (Section 3.1.4.4).

4) For a potential failure plane in the bentonite portion of GundSeal below the geomembrane, select the appropriate shear strength for GundSeal. If GundSeal is used as a single composite liner, select the hydrated shear strength values from direct shear test data presented in Table

3.1 for low normal loads. If the installation is on relatively dry subgrade soils that are above the active capillary fringe zone, the peak hydrated shear strength would be a reasonable design assumption. If the installation is on moist subgrade soils, or on landfills that produce gas, and the GundSeal is covered in a reasonable period of time after deployment (within 5 days), the upper range of hydrated post-peak shear strength would be conservative for most designs. Hydrated strengths closer to the lower range of post-peak shear strength (i.e. data from Fox et al., where the bentonite was allowed to hydrate under almost no normal load before consolidating and shearing) would be appropriate where the GundSeal was installed on a wet subgrade and/or was not covered with soil in a timely manner. These are considered conservative parameters as demonstrated later by the Cincinnati field test case which was installed on reportedly moist to wet subgrade soils.

5) Select the appropriate shear strength for the cover soil's toe resistance. A friction angle of 30° should be conservative for most soils at low normal loads that have at least 50% granular fraction (sands or gravels).

6) Determine the pore pressures that may exist. The analysis for a potential failure <u>above</u> the geomembrane should consider seepage parallel to the slope, which can be analyzed in terms of either seepage forces or pore pressures (Lambe and Whitman, 1969). A design approach described by Thiel and Stewart (1993) can be used to design the lateral drainage layer. The analysis for a potential failure <u>below</u> the geomembrane should consider the potential for landfill gas pressure, and pore pressures caused by relatively rapid increases in the cover soil unit weight (e.g. sudden saturation) as described in Liu et al. (1997).

7) The design approach used in the examples presented in this manual for veneer stability generally follows Koerner and Soong (1998). The examples presented are in terms of how a hand-calculated solution would be performed. An attempt has been made to describe the fundamental mechanics of the analysis as presented in the hand examples. The hand examples seek to preserve, as much as possible, the physical meaning of the analytical variables by carrying them through the computations and using an iterative solution. Koerner and Soong (1998) present precise quadratic solutions with detailed geometric input that can be programmed onto a spreadsheet. The final equations that they used yield the same results, but are much less intuitive because they have been highly manipulated for ease of computer programming.

Figure 3.4 shows a schematic cross-section of a typical cover slope. It is assumed that the analysis is performed on a "unit width" of this cross-section. Typically, the "unit" is either one foot or one meter, depending on whether the problem is being solved in English units or SI units.

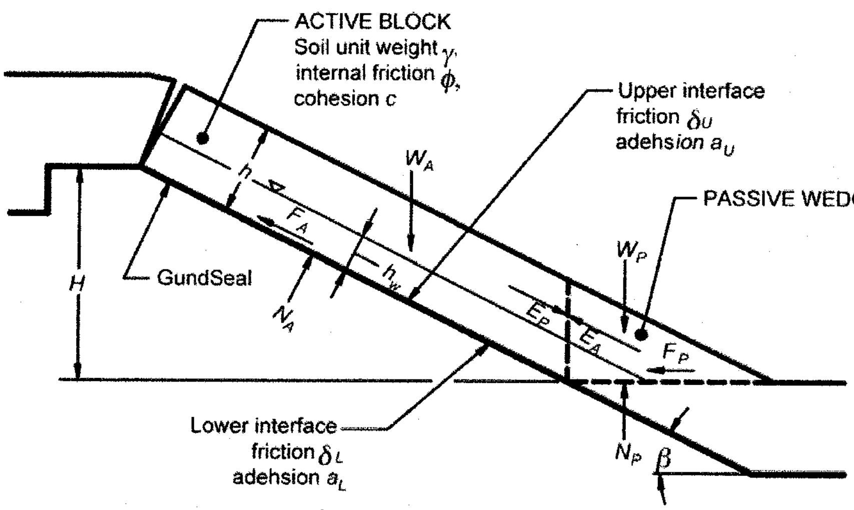

Figure 3.4 Generic Veneer Slope Problem.

where:

h = thickness of cover soil above liner
β = liner slope angle from the horizontal
H = slope height, measured from the top of the anchor trench to the top of the next lower bench or toe
W = weight of block (subscripts A and P denote active or passive block)
γ = unit weight of cover soil at a given moisture condition
ϕ = internal friction of cover soil
c = cohesion of cover soil
h_w = depth of saturation of cover soil
δ = friction angle of the interface (subscripts U and L denote upper or lower interface)
a = adhesion strength parameter of interface (subscripts U and L denote upper or lower interface)

The classic analysis for a veneer cover of finite length considers the stability of two soil blocks (denoted active and passive). The active block is the mass of soil on the slope. The passive block (or wedge) is the triangular area at the toe that will have to shear horizontally for failure to occur. It is assumed that there is a vertical boundary between the two blocks, with equal and opposite forces, E_A and E_P (lbs. or N), acting on these blocks. The direction of the forces is assumed parallel to the slope. These assumptions, while they may not be rigorously accurate, are accepted as standard practice in the profession for purposes of veneer stability analysis.

The values for friction δ and adhesion a for the upper and lower interfaces (designated by subscript $_U$ or $_L$) are dependent upon the specific interface evaluated within the liner system. For evaluating GundSeal and GCLs, the critical interface is generally associated with the bentonite layer. Thus, for overlapped and encapsulated GundSeal, the critical interface evaluated generally consists of the overlying/underlying geomembrane and the bentonite layer.

Figure 3.5 shows free-body diagrams (FBD) of the forces acting on the active and passive blocks. New dimensions defined on the FBD are that the length of the slope is denoted L_1, and the length of the bottom side of the passive wedge is denoted L_2. The approach now will be to write equations of static equilibrium for the active and passive blocks that include a provision for a Factor of Safety *(FS)*. The goal is to determine what is the value of *FS*. This will be achieved by developing equations of static equilibrium separately for the active and passive blocks in terms of the unknown inter-block forces E_A and E_P. Since $E_A = E_P$, these terms will cancel out resulting in a single equation that can be solved for *FS*.

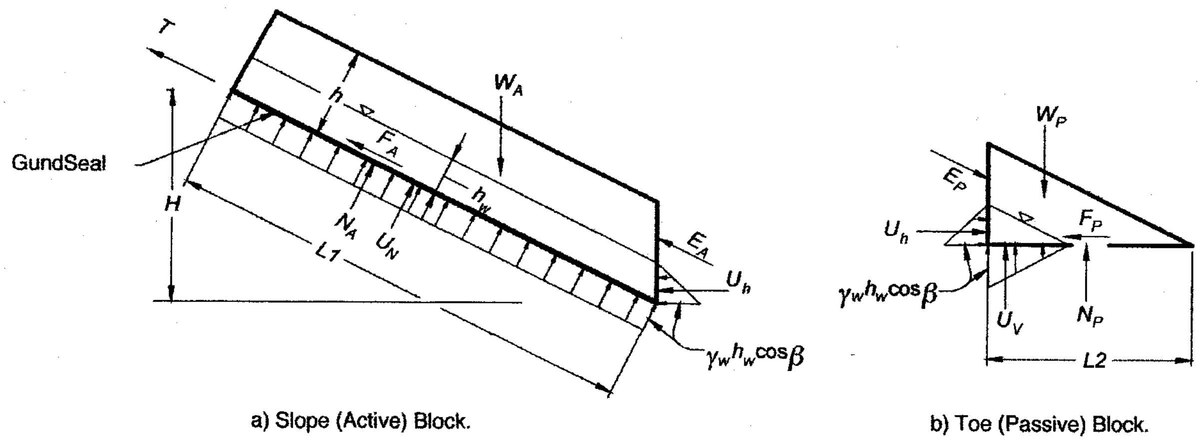

Figure 3.5 Free Body Diagram of a) Active Block and b) Passive Block on a Veneer Slope.

where:

F = resultant frictional force (subscripts A and P denote active or passive block)
N = resultant normal force (subscripts A and P denote active or passive block)
U = resultant hydrostatic force (subscripts N and H denote normal to slope or horizontal)
E = inter-block reaction force (subscripts A and P denote active or passive block)
$L1$ = length of slope of active block
$L2$ = length of slope of passive block
γ_w = unit weight of water
T = allowable tensile force in the geosynthetics <u>above</u> the assumed slip surface

<u>Active Block Forces</u>

1. The weight of the active block is W_A (lbs or N). It can be calculated simply by measuring the cross-sectional area of the active block and multiplying by γ. If the slope is partially

saturated then a blended unit weight between the moist and saturated unit weight should be used. Note that the veneer stability formulae presented by Koerner and Soong (1998) appear so complicated simply because of the laborious trigonometric expressions required to accurately describe the areas of the active and passive blocks. In a hand-written solution, these areas are easily estimated by drawing the cross-section to scale and directly measuring the area.

2. For analyses of the upper interface of the GundSeal geomembrane, the hydrostatic force on the slope, U_N, caused by partial saturation of the cover soils is calculated as

$$U_N = \gamma_w (h_w \cos\beta)\cdot(L_1) = \gamma_w (h_w \cos\beta)\cdot\left(\frac{H}{\sin\beta}\right) = \frac{\gamma_w h_w H}{\tan\beta} \tag{3.1}$$

3. The horizontal hydrostatic force, U_h, at the interface between the active and passive blocks (acting in equal and opposite directions on the two blocks) is:

$$U_h = \left(\frac{1}{2}\right)\gamma_w (h_w \cos\beta)\cdot\left(\frac{h_w}{\cos\beta}\right) = \frac{\gamma_w h_w^2}{2} \tag{3.2}$$

4. The effective force normal to the slope, N_A (lbs or N), is a result of the weight of the active block, the normal hydrostatic uplift force, and the normal component of the horizontal hydrostatic force shown on the free body diagram for the active block. It is calculated as

$$N_A = W_A(\cos\beta) - U_N + U_h(\sin\beta) \tag{3.3}$$

(The values for U_N and U_h would be zero if the analysis was for the lower bentonite interface in terms of water pressure. U_N could exist, however, in the form of a gas pressure.)

5. The value of the maximum shear force F_{Amax} that resists sliding on the slope depends on the normal force, N_A, and the interface friction and adhesion. It is calculated as:

$$F_{Amax} = N_A(\tan\delta) + (a)(L_1) \tag{3.4}$$

(The subscript $_U$ or $_L$ would be used with δ and a, depending whether the analysis was for the upper or lower interface).

Since we are not interested in mobilizing the maximum shear force, common practice is to apply a factor of safety *(FS)* to the shear strength of the active block. Therefore, the allowable, or mobilized, shear strength is written as:

$$F_A = \frac{F_{A\max}}{FS} = \frac{N_A(\tan\delta) + (a)(L_1)}{FS} \tag{3.5}$$

6. For the active block, forces can be balanced in the direction of E_A such that

$$E_A = W_A \sin\beta - U_h \cos\beta - T - F_A \qquad (3.6a)$$

or

$$E_A = W_A \sin\beta - U_h \cos\beta - T - \frac{N_A \tan\delta + (a)(L_1)}{FS} \qquad (3.6b)$$

The subscript $_U$ or $_L$ would be used with δ and a, depending whether the analysis was for the upper or lower interface.)

T is the <u>allowable</u> tensile force of geosynthetics that are <u>above</u> the assumed failure plane. For example, for a potential failure surface on the lower side of the geomembrane component of GundSeal, the allowable tensile force of the GundSeal geomembrane could be considered in the stability analysis. For HDPE geomembranes, it is recommended that the allowable long-term design strain be limited to something less than the yield strain (Koerner, 1998; Giroud et al., 1993).

The term "allowable" is used to qualify the tensile force used in the calculations. This means that the designer has already determined an allowable working stress in the geosynthetic, and no additional factor of safety would be applied to that value.

The deformations required to mobilize the allowable tensile force, T, will generally exceed the peak strength for the underlying interfaces. Therefore, whenever a value for T is used, it is recommended that the values of δ and a represent residual strength values for the slip planes.

Because welds and scratches perpendicular to the direction of stress may cause localized stress concentrations, the average allowable strain must be reduced to account for strain concentrations. Giroud et al. (1993) demonstrated how a scratch 10% of the thickness of the geomembrane could reduce the average yield strain by 66%. Using ASTM D 4885 (wide width tensile test for geomembranes) as a starting point, the yield strain for HDPE geomembranes under these conditions is approximately 20%. Using a factor of safety of ≥5.0 might suggest an average allowable tensile strain of 4% for the HDPE geomembrane backing of GundSeal installed on a slope. Additionally, since there typically will be no welds in the GundSeal geomembrane on the slope, this amount of allowable tensile strain is deemed conservative.

If other geosynthetics are installed over the GundSeal, such as a protective geotextile cushion or a reinforcement geotextile or geogrid, the allowable tensile force of those materials could be added to the geomembrane. Note that the separate tensile forces in layered geosynthetic systems should all be at a compatible strain.

<u>Passive Block Forces</u>

1. The weight of the passive block is W_P and is calculated in the same manner as W_A.

2. The vertical hydrostatic uplift force on the passive wedge, U_V, caused by partial saturation of the cover soils, is calculated as

$$U_V = \frac{U_h}{\tan\beta} \tag{3.7}$$

3. The horizontal hydrostatic force, U_h, acting on the passive block at the interface between the active and passive blocks, is the same as was calculated for the active block.

4. The effective force normal to the bottom of the passive wedge, resulting from the vertical component of the other forces shown in the FBD in Figure 3.4, is N_P. It is calculated as

$$N_P = W_P + E_P(\sin\beta) - U_V \tag{3.8}$$

5. The value of the maximum shear force F_{Pmax} that resists sliding of the passive wedge depends on the normal force, N_P, and the interface friction and cohesion of the soil. It is calculated as the lesser of

$$F_{Pmax} = N_P(\tan\phi) + (c)\,L_2 \tag{3.9}$$

Since we are not interested in mobilizing the maximum shear force, common practice is to apply the same factor of safety (*FS*) to the shear strength of the passive wedge as is applied to the active block. Therefore, the allowable, or mobilized, shear strength is written as:

$$F_P = \frac{F_{P\max}}{FS} = \frac{N_P(\tan\phi) + (c)(L_2)}{FS} \tag{3.10}$$

The forces can be balanced in the horizontal direction for the passive wedge as

$$\sum F_x = 0 = F_P - E_P(\cos\beta) - U_h \tag{3.11}$$

The expressions for N_P (Equation 3.8) and F_P (Equation 3.10) can be substituted into Equation (3.11) and rearranged to solve for E_P as:

$$E_P = \frac{(W_P - U_V)(\tan\phi) + (c)(L_2) - (FS)U_h}{(FS)\cos\beta - \sin\beta\ \tan\phi} \tag{3.12}$$

(This is the first equation whose terms do not readily appear to relate to any physical meaning, but this was unavoidable for an exact solution.)

<u>Solution for *FS*</u>

Since $E_A = E_P$, we can set equations (3.6b) and (3.12) equal to each other and solve for *FS*. The equation can be arranged in the form of $0 = A'(FS)^2 + B'(FS) + C'$ where the exact solution is in the form of a quadratic equation, which can be solved explicitly as described by Koerner and Soong (1998) as presented below. It can also be solved iteratively by trying different values of *FS* until $E_A = E_P$ from equations (3.6b) and (3.12).

$$FS = \frac{-B' + \sqrt{B'^2 - 4A'C'}}{2A'} \tag{3.13}$$

where:

$$A' = U_h \sin^2\beta + (W_A \sin\beta - T)\cos\beta$$

$$B' = (\sin\beta)(\tan\phi)(U_h \cos\beta + T - W_A \sin\beta) - \cos\beta(N_A \tan\delta + a\frac{H}{\sin\beta}) - (\tan\phi)(W_P - U_V) - (c)(\frac{h}{\sin\beta})$$

$$C' = (\sin\beta)(\tan\phi)(N_A \tan\delta + a\frac{H}{\sin\beta})$$

Note, however, the toe resistance force, E_A or E_P, cannot exceed the lateral passive pressure provided by the cover soil. This consideration becomes more significant as the slope being analyzed becomes longer and flatter. Therefore

$$E_{A(\max)} = \frac{\gamma h^2}{2} K_P \tag{3.14}$$

where γ = unit weight of cover soil, h = cover soil thickness; and K_P = coefficient of passive earth pressure for the cover soil. The value of K_P can be estimated for cohesionless soils according to standard soil mechanics principles as

$$K_P = \tan^2(45 + \phi/2) \tag{3.15}$$

where ϕ = the internal friction of the cover soil. For example, if ϕ = 30° then K_P = 3.

If the value of E_A calculated in Equation (3.14) is greater than the value calculated by the iterative process, then the factor of safety should be recalculated using Equation (6.6b), where the value for E_A is obtained directly from Equation (3.14). The solution would be written as

$$FS = \frac{N_A \tan\delta + (a)(L_1)}{W_A \sin\beta - U_h \cos\beta - T - E_A} \tag{3.16}$$

3.2 GundSeal Shear Strength

Typical shear strength parameters for the bentonite coating of GundSeal are presented in this section. These shear strengths are related exclusively to the internal shear strength of GundSeal, that is, to the bentonite coating and its interface with the geomembrane portion of GundSeal. The shear strength of the interfaces of the outer surface of GundSeal's geomembrane with another material is not discussed in this manual.

under design normal loads ranging up to 30,000 psf (1,435 kPa). Equations are presented for both peak and post-peak conditions.

5. Appendix H presents supporting direct shear test data and related analysis and summary of results as performed by GeoSyntec Soil-Geosynthetic Interaction Testing Services. This GundSeal direct shear testing program was conducted for the development of this manual.

The reader is referred to the referenced sections in the manual for related information and performance data to serve as supporting documentation for project specific design slope stability considerations. This section expands on issues related to slope stability and design with GundSeal utilized in bottom liner applications.

5.4.1 *Regulatory Considerations*

Many states and jurisdictions have requirements for certain minimum static factors of safety, seismic factors of safety, or maximum allowable seismic deformations regarding site-specific stability.

5.4.1.1 Factor of Safety. The most typical requirement for static stability is to meet a factor of safety of ≥1.5. The origin of this value was the empirical result of analyzing the relative success and failure of dams that have been constructed. Experience proved that when an analysis was performed correctly, assuming reasonable and prudent material properties, an earthen structure with a factor of safety of 1.5 can be expected to remain stable even though some of the actual structure geometry and material properties may have varied from those assumed in the analysis.

The essential operative words in stability analysis are *"performed correctly"*. The safety margin in a "factor of safety" exists to account for the unknowns or unpredicted deviations from the original design assumptions. The safety margin is not supposed to account for errors in the analysis, or inappropriate geometric and material property assumptions.

Reduced factors of safety are sometimes used for short-term conditions, interim conditions, or conditions where the impact of a failure would not threaten life or the environment and where the failure is easily repaired. Factors of safety on the order of 1.25 to 1.3 are frequently used for such situations. Other possible reasons for a reduced factor of safety include situations where the material properties, geometry, and pore pressures have been defined with a high degree of confidence or where the factor of safety is based on lower bound, large deformation, "residual" shear strengths. Factors of safety as low as 1.1 are sometimes used for very short-term interim conditions (e.g., during construction) if they are based upon lower bound shear strengths (e.g., fully softened residual strengths). Designs incorporating reduced factors of safety generally require additional documentation to demonstrate their adequacy. In any case, it is recommended that the designer check that the stability factor of safety be greater than unity using the residual shear strength of the interface with the lowest peak strength.

When using GundSeal, this should be checked assuming the bentonite is fully hydrated, even for encapsulated designs.

5.4.1.2 <u>Seismic Stability.</u> As discussed in Sections 3.3.1 and 3.3.2, seismic stability is generally based upon the concept of allowable deformations. Seismic deformations calculated as a function of the yield acceleration using Newmark-type deformation analyses are compared to allowable values to establish the adequacy of a design. However, the seismic deformation calculated in these analyses is merely an index of the seismic performance and not a quantitative estimate of the anticipated deformation in the design earthquake. Similar to the factor of safety, allowable calculated seismic deformation values used in practice have been empirically established by analyzing slopes, embankments and landfills that have performed satisfactorily when subjected to strong earthquakes. Also similar to the factor of safety, allowable deformations depend upon the impact of a failure, the ease of repair, and conservatism inherent in the shear strengths assigned to the components of the system.

An additional factor in evaluating the allowable calculated seismic deformation is the conservatism inherent in the seismic response analysis used to evaluate the potential for amplification of seismic motions and the attenuation of motions due to spatial and temporal coherence. Most guidelines for allowable calculated seismic deformations used in practice today are based on analyses performed using equivalent linear seismic response analyses, such as the computer program SHAKE [Schnabel et al. (1972); Idriss and Sun (1992)]. Seismic deformations on the order of 6 to 12 in (150 to 300 mm) calculated using equivalent linear response analyses and large deformation shear strengths are generally assumed to indicate no significant damage to a landfill liner (or cover) system. Seismic deformations on the order of 3 ft (900 mm) calculated using equivalent liner response analyses are generally assumed to indicate limited deformations in the design event. This level of deformation is generally assumed appropriate for cover systems which are easily repairable.

If the simplified seismic response analyses described in U.S. EPA (1995) are used, these deformation limits may be conservative. However, if this simplified U.S. EPA analyses indicate unacceptable deformations, current practice is to conduct a more sophisticated response analysis (e.g., SHAKE) rather than to increase the acceptable calculated deformation. Both the simplified and equivalent liner analyses are de-coupled analyses in which response calculated assuming no permanent seismic deformation is used to calculate induced permanent seismic deformation. The allowable calculated deformations cited above may not be appropriate for state-of-the-art fully coupled two and three dimensional seismic response and deformation analyses which attempt to make precise estimates of the deformation that will actually occur in the design earthquake.

In some jurisdictions, seismic design may still be based upon the pseudo-static factor of safety. However, specifying a pseudo-static factor of safety is meaningless unless it is specified in conjunction with a seismic coefficient. Because the appropriate seismic coefficient depends on the intensity of the design earthquake, the seismic coefficient is often specified as a function of the peak ground acceleration (PGA) associated with the design earthquake. If the factor of safety is equal to or greater than 1.0 for a seismic