DRAFT TECHNICAL GUIDANCE DOCUMENT ON STATIC AND SEISMIC SLOPE STABILITY FOR SOLID WASTE CONTAINMENT FACILITIES

PRODUCED BY

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1.0 INTRODUCTION

Locating the critical cross-section and critical failure surface is the first step in the analysis of the static and seismic stability of landfill slopes. Locating the critical cross-section is usually accomplished by analyzing two-dimensional cross-sections through a particular slope. Only cross-sections that represent a realistic, or plausible, sliding direction should be selected. For example, a cross-section depicting the down canyon direction, or bedrock dipping away from the slope, should be selected. Figures 1 through 7 depict typical failure modes of landfill slopes. The two-dimensional cross-sections are usually analyzed using a limit equilibrium slope stability method to estimate the factor of safety. Each cross-section is searched to locate the critical failure surface, (i.e., the failure surface with the lowest factor of safety in that cross-section). The cross-section that yields the lowest factor of safety is referred to as the critical cross-section.

Factors affecting slope stability are:

- Slope geometry, height, inclination, surcharges (additional driving forces), and berms/buttresses (additional resisting forces).

- Shear strength of interfaces, waste, cover materials, foundation materials (related to bearing failure), and effects of seismicity (e.g., soil liquefaction).

- Pore-water pressures/seepage forces.

- Gas pressures.

- Loading conditions including driving forces (e.g., unit weight of waste, compaction of waste, and bailing of waste) and pre- and post-closure conditions (e.g., seismic events, settlement, and expansion).

- Settlement caused by the landfill (e.g., increase in unit weight and infiltration); and the foundation material(s) (e.g., soils or organics).

2.0 STATIC LIMIT EQUILIBRIUM ANALYSES

I. INPUT AND ANALYSIS

For the critical cross section through the landfill, the peak or residual shear strength for a slip surface in a certain material or along an interface between two materials, is represented by a failure envelope. The failure envelope is typically a nonlinear plot of peak or residual shear stress versus effective normal stress obtained from laboratory tests on representative materials. The laboratory tests should be conducted on site specific soils and/or geosynthetics and should simulate the field conditions, including material arrangement, hydration, normal stress, displacement rate, etc. The use of manufacturer generated data should not be used unless verified with site specific testing. This is discussed in more detail in Section 6.0 titled "Shear Strength of Geosynthetic Interfaces."

Each shear test is used to determine a peak and/or residual shear strength for a particular normal stress. Peak and/or residual shear strength from one test can be determined from a plot of shear stress (τ) versus displacement. Several values of peak and/or residual shear stress plotted with the corresponding effective normal stress are used to draw a failure envelope. The Mohr-Coulomb equation of a failure envelope is $\tau_{max\ or\ residual} = c' + \sigma_n' \tan \phi'$, where $\tau_{max\ or\ residual}$ is the maximum (or peak) or residual shear strength, c' is cohesion or vertical intercept of the failure envelope, σ_n' is effective normal stress which is equal to σ_T (total normal stress) minus the u (pore water pressure), and ϕ' is the angle of internal friction or inclination of the failure envelope.

The cohesion is typically zero for the residual failure envelope. In addition, it is recommended that the cohesion be set equal to zero for the peak failure envelope, because the cohesion is not a function of normal stress and thus is assumed to be constant along the entire failure surface in the factor of safety calculation. In summary, it is recommended that the shear strength be represented by a friction angle or the actual failure envelope can be input directly into the slope stability software. The effective stress failure envelope may be different than the total stress envelope for the same slip surface in a certain material or along a certain interface because of the presence of pore-water pressures.

II. TWO-DIMENSIONAL LIMIT EQUILIBRIUM METHODS

Two-dimensional limit equilibrium analysis methods are applicable to practical problems because they can accommodate circular and non-circular slip surfaces and variable soil properties and pore-water pressures in the slope or cross section. Most of the two dimensional limit equilibrium methods divide the slope, bounded by a circular or non-circular slip surface, into a number of vertical slices. For each slip surface - be it along several interfaces and/or within one material - there will be a factor of safety (F.S.) equal to the sum of the resisting forces, shear stresses, or moments of each slice divided by the sum of the opposing/driving forces, shear stresses, or moments of each slice. Duncan (1992) shows that limit equilibrium stability methods that satisfy all conditions of equilibrium, i.e., vertical force equilibrium, horizontal force equilibrium, individual slice moment equilibrium, and overall moment equilibrium, provide the most accurate estimate of the factor of safety. Methods that satisfy all conditions of equilibrium, e.g., Janbu (1957), Morgenstern and Price (1965), and Spencer (1967), will yield factors of safety that are within 5 percent of the correct answer. This small error is substantially less than that which could be caused by an input error. Therefore, the majority of the time and effort should be spent on defining the slope geometry, realistic failure modes/surfaces, pore-water pressures, unit weights, and shear strength information after selecting a stability method that satisfies all conditions of equilibrium.

For circular and non-circular failure surfaces the following assumptions are made in all limit equilibrium methods:

(1) Shear strength is fully mobilized along the failure surface; and

(2) The normal force acts at the center of the vertical slice base.

For a non-circular failure surface in the critical cross section, where a $\tau_{max\ or\ residual}$ failure envelope can be defined for each interface along the failure surface, there is one F.S. for this non-circular slip surface. The parameters c, ϕ, unit weight (γ) and the coordinate dimensions of each mass above the failure surface can be input into the method to obtain the sum of the driving shear stresses for calculating the factor of safety.

Several computer software programs allow a search for the critical non-circular failure surface. Note that a non-circular slip surface can be in the cover system, the liner system, native soils, or at a native soil/rock interface below the liner system.

For a circular failure surface in the critical cross section, be it above, below, or through the non-circular basal slip surface, the computer software can conduct searches for the critical circular slip surface through these masses or combined mass. The c, ϕ, unit weight (γ) and the coordinate dimensions of each mass type are required input. In addition, instructions on which mass or combined masses need to be searched must be input. If a search for a critical circular slip surface yields a slip surface that passes through a liner system, the liner component(s) may have to be analyzed separately to determine if the shear stresses generated cause an undesirable F.S. for the tensile strengths of the liner geosynthetics. However, the critical failure surface is usually along, not through, the geosynthetics due to the low shear strength of soil/geosynthetic and geosynthetic/geosynthetic interfaces.

III. SLOPE STABILITY COMPUTER PROGRAMS

Slope stability computer programs usually utilize one or more of the following two dimensional limit equilibrium stability methods. The main difference between these methods is the conditions of equilibrium that are satisfied and the assumption(s) concerning the inter-slice forces.

- o Ordinary Method of Slices (Fellenius 1927) - avoid using for flat slopes with high pore-water pressures.
- o Bishop's Modified Method (Bishop 1955) - use only for analysis of circular slip surfaces. Bishop's Method does not satisfy horizontal force or individual slice moment equilibrium but is accurate ($\pm$ 5% from the correct factor of safety for circular slip surfaces).
- o Janbu's Generalized Procedure of Slices (Janbu 1957) - satisfies all conditions of equilibrium.
- o Morgenstern and Price's Method (Morgenstern and Price 1965) - satisfies all conditions of equilibrium but is difficult to use.
- o Spencer's Method (Spencer 1967) - satisfies all conditions of equilibrium and is recommended for both circular and non-circular failure surfaces.

- o Force Equilibrium.

Since Spencer's method satisfies all conditions of equilibrium for circular and non-circular surfaces and can be accurately coded into a computer program, it is recommended that Spencer's method be selected for analysis in the software package. A number of software packages utilize Spencer's method, e.g., XSTABL (Sharma, 1995) and UTEXAS2 or UTEXAS3 (Wright, 1986).

For a summary and evaluation of existing limit equilibrium stability methods, including their characteristics, equilibrium conditions satisfied, and assumptions (e.g., characteristics of resultant side forces), see professional papers by Duncan and Wright (1979) and/or Duncan (1992 or 1996).

When selecting a software package consider the stability methods used by the software package. Methods that satisfy all conditions of equilibrium should yield accurate results (i.e., F.S.'s which differ by no more than $\pm$ 5% from the correct F.S.) for most if not all practical conditions.

Additional questions to ask when selecting a slope stability software package include:

- o What slip surface geometry can be analyzed?
- o What are the shear strength options (e.g., non-linear failure envelopes)?
- o What are the pore-water pressure options? (Note that with water below a critical slip surface $\sigma_n' = \sigma_T$ and with water above a critical slip surface $\sigma_n' = \sigma_T - u$.)
- o What are the input and output techniques?

Verification of the factor of safety computed by a slope stability software package using a method of vertical slices can be accomplished using one of the following techniques:

- o Use more than one slope stability program and ensure reasonable agreement between the calculated values of F.S. for the <u>same</u> stability method.
- o Use of slope stability charts.
- o Use of hand calculations.
- o Use a three dimensional stability method and ensure

reasonable agreement between the calculated values of factor of safety for the same stability method. F.S. from a three dimensional analysis is $\geq$ F.S. from two dimensional analysis for appropriate comparisons (e.g., three dimensional Janbu's method versus two dimensional Janbu's method).

IV. FINAL COVER STABILITY ANLAYSIS

The stability of final covers is usually evaluated using an infinite slope analysis. An infinite slope analysis applies to slopes where the thickness of the slide mass is small compared to the slope height. The analysis assumes that movement of the slide mass will occur parallel to the slope. The forces causing movement are due to the weight of the slope materials and seepage forces. The forces resisting movement are usually provided by the geosynthetic and/or soil interfaces. The infinite slope equation is presented below and it neglects toe buttressing forces and the tensile strength of the geosynthetic components.

$$F.S. = \left[\frac{C_i}{\gamma * z * \cos^2 \beta} + \tan \phi_i \left(1 - \gamma_w \frac{(z - d_w)}{(\gamma * z)} \right) \right] * (\tan \beta)^{-1}$$

or

$$F.S. = \left[\frac{C_i}{\gamma * z * \cos^2 \beta} + \tan \phi_i \left(1 - \frac{u}{(\gamma * z)} \right) \right] * (\tan \beta)^{-1}$$

where F.S. = factor of safety, C_i = interface cohesion, ϕ_i = interface friction angle, γ = unit weight of cover material, z = depth to interface, γ_w = unit weight of water, d_w = depth to water, β = slope angle, and , u = pore-water pressure.

Field observations and calculations show that the most important parameter in the infinite slope analysis is the depth of water in the slope, i.e., pore-water pressure. The occurrence of numerous seepage cover failures illustrates the importance of including seepage forces or boundary water pressures in the analysis. These seepage failures occurred even though a drainage layer or drainage composite was installed. In addition, the F.S. is extremely sensitive to values of d_w or pore-water pressure inserted into the equations shown above. Additional details on the infinite slope analysis can be found in "Stability of Geosynthetic - Soil Layered Systems on Slopes" by Giroud et al.

(1995a and b) or “Waste Containment Systems, Waste Stabilization and Landfills” (Sharma and Lewis 1994).

V. RECOMMENDED FACTOR OF SAFETY

The Missouri solid waste regulations do not specify a particular range or value for static factor of safety. However, the recommended value of factor of safety for static conditions is 1.5. As discussed subsequently in Section 3.0, a permanent seismic deformation analysis is preferred to a pseudo-static analysis with a F.S. $\geq$ 1.5. Therefore, a recommended factor of safety for pseudo-static conditions is not presented. This is in agreement with Missouri Site Selection regulations (10 CSR 80-3.010) which requires that sanitary landfills shall not be located in seismic impact zones unless it is demonstrated that all containment structures can resist a permanent cumulative earthquake displacement of six inches or less resulting from the maximum credible Holocene time earthquake event’s acceleration versus time history.

Scenarios to consider when selecting interface strengths for slope stability analyses (Stark and Poeppel 1994):

- o Apply one of the following design scenarios:

  Scenario I - Use residual shear strength for side slope interfaces. Use peak shear strength for basal or relatively flat interfaces and satisfy the recommended factor of safety.

  Scenario II - Use residual shear strength for side slope and basal interfaces and satisfy a factor of safety greater than unity.

- o Interface failure envelopes may be non-linear.
- o Use the entire non-linear failure envelope or a friction angle that represents the applicable range of effective stresses.
- o Model the failure envelope for each interface in the slope stability software by inputting 10 to 20 combinations of shear and normal stress that describe the failure envelope.

VI. THREE DIMENSIONAL ANALYSIS

Since the three-dimensional (3D) factor of safety is greater than the two-dimensional (2D) F.S. for similar conditions and stability methods, only a brief comparison of 3D and 2D limit equilibrium methods is presented:

- o Two dimensional methods of analysis assume a plane strain condition, i.e., no shear forces acting on the side of the slide mass. Slopes are not infinitely wide as assumed in a two dimensional analysis, and thus three dimensional effects influence/increase stability.

- o With resolving side forces on the sliding mass - which cannot be done in a two-dimensional analysis - a three-dimensional method of analysis can identify the orientation of the resultant force or most critical sliding direction. Identification of the critical sliding direction can facilitate identification of the critical cross section for a two dimensional stability analysis.

- o By using a three dimensional method of analysis one can back calculate shear stresses and shear strengths mobilized on the failure surface after a slope failure. Two-dimensional analysis will overestimate the back-calculated shear strength because the side forces are not taken into account.

VII. RECOMMENDED SLOPE STABILITY INFORMATION

The specific contents of a slope stability analysis are sensitive to particular conditions present at an individual site and often need to be assessed on a case by case basis. However, in general, a slope stability analysis for a sanitary landfill should include the following:

A. The rationale, cross-sections, and plan views, for critical slope conditions* that may occur during the <u>excavation and construction</u> of the landfill**.

B. The rationale, cross-sections, and plan views, for critical slope conditions* which may occur during the <u>operation and filling</u> of the landfill**.

C. The rationale, cross-sections, and plan views, for critical slope conditions* which may occur during <u>closure and post closure</u> of the landfill.

D. The rationale for selection of soil and geosynthetic interface strength characteristics, including detailed information from a site specific subsurface exploration, and detailed information from a project specific geosynthetic shear strength testing program.

E. A discussion of the methodology used for the determination of the factors of safety including a summary of the input parameters, e.g., shear strength, unit weight, and pore-water pressure.

F. The physical calculations and/or computer output for the critical conditions of the excavation, intermediate waste, and final slopes.

* Determining critical slope conditions includes investigating both static and dynamic cases for both deep-seated and shallow failure surfaces and for rotational and translational modes of failure.

** Operational and construction practices can have a profound impact upon the integrity of the engineered components of waste containment facilities and should not be overlooked in the design process.

VIII. METHODS OF FIELD EVALUATION OF SLOPE STABILITY

Field evaluation of slope stability includes:

- o Inspecting the top of the slope for something equal to or more severe than tension cracks or vertical offsets.

- o Inspect the bottom (toe) of slope for something equal to or more severe than uplift, heave, or displacement.

- o Determine if the slope is too steep, e.g., steeper than 3 horizontal: 1 vertical. This can be accomplished using surveying techniques or comparing waste grades from semi-annual or annual reports submitted by the owner/operator. This comparison can also be used to determine if any overbuild has occurred.

- o Determine if there is seepage exiting the slope; if there is erosion or excavation at the toe; or if there are any signs of creep (strength loss).

- o If there is movement detected, then install a slope inclinometer, if possible, for determining the slide surface location and velocity of the slide mass movement. This may not be feasible or advised through a composite liner system.

Remedial measures for slope instability may include:

- o Unload the top of the slope (flatten the slope).
- o Buttress the toe of the slope (soil, waste, rock berm, retaining structure, tie backs, shear key, etc.).
- o Increasing shear strength (drainage, textured geosynthetics, grouting, geogrids, etc.).

Features on the ground surface can provide the key to understanding the details of landslide processes and causes. Therefore, particular attention must be paid to these features and detailed notes and sketches should be made. An active landslide may affect existing structures, utilities, and other artificial features in ways that provide insight into the processes and cause of the feature. Cracks in pavement, foundations, and other brittle materials can support inferences about the stress produced by movement of the landslide. The timing of breakage of leachate/liquid and gas lines, drainage ditches, electrical cables, and similar utilities can suggest the sequence of deformation before field observations or measurements. Measuring the tilt of structures or trees assumed to be vertical or horizontal before movement can yield an estimate of the amount of displacement on certain parts of the slope. Of course, titled trees also can be used to ascertain that the slope has moved previously. The key features are the main scarp, lateral flanks, internal morphology, vegetation, cover soil, and toe relationship. The rate of change of these features is extremely important. If the slope is determined to be moving, the rate at which these features are changing should be monitored daily.

Table 1 is a summary of the terrain features frequently associated with slope movements and relationship of these feature to landslides.

Table 1. Morphologic, vegetation, and drainage features characteristic of slope instability.

TERRAIN FEATURE	RELATION TO SLOPE INSTABILITY
Morphology	
Cracks at top of slope	Formation of head scarp
Bulging at toe of slope	Movement of slide mass
Hummocky and irregular slope	Shallow movements or retrogressive slide blocks
Back-titling slope facets	Rotational movement of slide blocks
Vegetation	
Back-titling trees	Rotational movement of slide blocks
Irregular linear changes along slope	Slip surface of translational slides
Disrupted, disordered, and partly dead vegetation	Slide blocks and differential movements in
Drainage	
Areas with stagnated drainage	back-tiling slide blocks and hummocky relief on slope
Excessively drained areas (erosion)	Outbulging of slope
Seepage from slope	Slope face or failure plane outcrop
Interruption of drainage lines	Formation of head scarp
Anomalous drainage pattern	Streams or ditches curving around front of slope

3.0 SEISMIC STABILITY ANALYSES

I. PSEUDO-STATIC ANALYSIS

Historically earthquake analyses have been accomplished by determining a pseudo-static (Terzaghi 1950) F.S. for the critical slip surface determined from the static limit equilibrium analysis. This can be accomplished using a seismic coefficient multiplied by the acceleration due to gravity (g = 32.2 ft./sec.^{2} or 9.81 m/sec.^{2}) that represents the maximum (or peak) horizontal earthquake acceleration from the regulatory maximum earthquake event (A_{max}). A typical A_{max} is a multiple of the peak horizontal bedrock acceleration, which is estimated from the U.S.G.S. Map (Algermissen et al., 1990). The U.S.G.S. map presents contours of the horizontal bedrock acceleration for 10% exceedence in a 250 year earthquake event. The seismic coefficient multiplied by gravity is the maximum horizontal earthquake acceleration acting at the center of gravity of the slide mass.

A permanent seismic deformation analysis should be used instead of a pseudo-static slope stability analysis. This is to satisfy Missouri Site Selection regulations (10 CSR 80-3.010) that require all containment structures to resist a permanent cumulative earthquake displacement of six inches or less. A permanent deformation analysis is also preferred because it provides a direct estimate of field performance, i.e., resulting displacement, while the pseudo-static F.S. yields an indirect estimate. In addition, the limitations of a pseudo-static stability analysis include:

- Large uncertainty in estimating the seismic coefficient.
- Assumes a horizontal force, derived from the seismic coefficient, is permanent and acts in one direction.
- Pseudo-static F.S. is not a single value as calculated above, but varies with time over one earthquake event;
- Permanent deformation theoretically only occurs when pseudo-static F.S. is less than unity.
- Pseudo-static F.S. does not indicate the magnitude of displacement in the slope after shaking. As a result, a pseudo-static analysis cannot be used to determine whether the sum of all displacements will cause structural damage (as required by 10 CSR 80-3.010) or a post-peak strength loss. Pseudo-static F.S. is for one point in time (one earthquake pulse).

II. PERMANENT SEISMIC DEFORMATION ANALYSIS

The procedure developed by Newmark (1965) is commonly used to estimate the permanent seismic deformation of earthen embankments, such as landfills. The following assumptions are made in the Newmark sliding block analysis:

A. The slide mass is a rigid-plastic body (does not deform internally).

B. There is no displacement at accelerations below the yield acceleration (yield coefficient, K_y, multiplied by g).

C. The sliding mass deforms plastically along a discrete basal shear surface when the yield acceleration (K_y multiplied by g) is exceeded. This can be applied to the landfill liner system.

D. Static and dynamic shearing resistances are identical.

E. The effect of dynamic pore-water pressures are neglected. In other words, the materials do not lose strength during shaking (e.g., the extreme loss of strength case would be liquefaction).

F. The yield acceleration (K_y multiplied by g) is not strain dependent and thus remains constant throughout the analysis. Please note here that in reality, however, yield acceleration decreases as deformation increases because the shear strength along the critical slip surface decreases with increasing displacement.

Note that the vertical component of the earthquake acceleration pulses are not considered in the Newmark (1965) analysis. Researchers are currently working on software/methods to address these vertical accelerations and their impact on the permanent seismic deformation.

A cumulative (over several maximum earthquake event pulses) permanent seismic deformation analysis can be accomplished in at least three ways. The appropriate methodology is a function of the project risk and magnitude of permanent deformation. A simple classification procedure is presented below for the three types of analysis:

1. Simplified Deformation Analysis: warranted for

preliminary analysis, routine projects and small landfills with minimal impact.

2. One-Dimensional Deformation Analysis: warranted on medium-sized landfills or when the simplified procedure indicates potential problems, i.e., excessive deformations.

3. Two-Dimensional Deformation Analysis: warranted on large or important landfills where there might be significant impact from slope failure, or when the one-dimensional analysis indicates potential problems, i.e., excessive deformations.

Only the simplified deformation analysis is discussed in detail herein. Extensive discussion of the one-dimensional and two-dimensional deformation analyses are beyond the scope of this document. The following paragraphs describe the simplified procedure for determining the permanent seismic deformation along the critical slip surface:

- Determine yield acceleration, which is the yield coefficient (K_y) multiplied by gravity that will produce a factor of safety of unity, for the critical static slip surface. K_y is usually obtained by a trial-and-error process using the static slope stability software. Values of seismic coefficient are input until the factor of safety equals unity. The seismic coefficient that corresponds to a F.S. of unity equals K_y.

- Determine maximum average acceleration of the slide mass for the static critical slip surface using a seismic response analysis or empirical chart.

- Using the empirical chart presented by Makdisi and Seed (1978), estimate the associated cumulative permanent displacement along the critical slip surface for a selected earthquake event. The Makdisi and Seed (1978) chart (see Figure 8) uses the ratio of the yield acceleration to the maximum average acceleration for a given earthquake magnitude to estimate the permanent deformation of the critical slip surface. In general, if the ratio of yield acceleration to maximum average acceleration for the critical slip surface is greater than 0.8, the cumulative permanent displacement will be less than one (1) centimeter (cm) for earthquake magnitudes ranging from 6.5 to 8.25. Due to the uncertainties in using a simplified chart and limitations of the deformation analysis, the upper bound curve for each earthquake magnitude should be used (see Figure 8).

- A more detailed calculation of cumulative seismic deformation for the regulatory maximum earthquake event can be achieved by double-integrating the acceleration time history for the critical slide mass. The portions of the acceleration time history that exceed the yield acceleration result in permanent deformation.

III. RECOMMENDED PERMANENT DEFORMATIONS

Allowable permanent cumulative deformation along a static critical slip surface is site specific and failure surface specific. For example, the allowable permanent deformation along the base of a "grandfathered", no geosynthetic liner system installed, landfill shall not exceed six (6) inches. The allowable permanent deformation in a geosynthetic lined area shall not exceed six (6) inches (see Missouri Site Selection Regulations, 10 CSR 80-3.010). In Section 7.0 this displacement is limited to four (4) inches when a geosynthetic clay liner (GCL) is utilized in the composite liner system because of the nature of the material. The allowable deformation in a geosynthetic final cover (with and without a GCL) is also limited to six (6) inches. This is in agreement with Missouri Site

Selection regulations (10 CSR 80-3.010) which requires that sanitary landfills shall not be located in seismic impact zones unless it is demonstrated that all containment structures can resist a permanent cumulative earthquake displacement of six inches or less resulting from the maximum credible Holocene time earthquake event's acceleration versus time history.

Dynamic shear strength properties of in-situ waste can be determined via direct geophysical seismic field methods. As a first approximation, the dynamic shear strength of waste can be assumed to be at least 80% of the static shear strength (see Section 5.0). Dynamic shear strength of geosynthetic interfaces can be estimated from static laboratory tests (Yegian and Lahlaf 1992 a and b). The undrained shear strength of native cohesive soils can be estimated using 80 percent of the static/drained laboratory shear strength (Makdisi and Seed 1978). The dynamic shear strength of partially saturated cohesionless soil can be approximated using the drained shear strength. The undrained shear strength of saturated cohesionless soil should be estimated from a liquefaction analysis (see Section 8.0).

Amplification or attenuation of earthquake acceleration pulses are a function of the height of the landfill, landfill topography, length of a critical slip surface, earthquake amplitude, frequency content, and duration, etc. These factors can be considered and evaluated using a seismic response analysis.

Cumulative permanent displacement is a function of amplitude of acceleration, frequency content of the earthquake event, landfill height and stiffness of the waste, and duration of the earthquake event.

The resultant shear stresses generated along a critical slip surface from static or seismic slope stability analysis shall not exceed the tensile strengths or pull-out resistances (i.e., from anchor trenches) of the geosynthetic materials.

In addition to cumulative seismic deformation analysis, it is important to conduct a liquefaction stability analysis to ensure there is no loss of shear strength or increase in compressibility of the native materials underlying the facility. Evaluation of liquefaction potential is discussed in Section 8.0 of this document.

IV. SIMPLIFIED DEFORMATION ANALYSIS

A simplified procedure of modeling an earthquake of specified magnitude and location so as to determine the cumulative earthquake event deformation for each critical slip surface is described below:

1. Estimate peak horizontal bedrock acceleration, A_{max}, for 10% exceedence in 250 years from the Algermissen et al. (1990) map that presents contours of peak horizontal bedrock acceleration.

2. Classify site stiffness. Sites are classified based on shear wave velocity (V_s) using the criteria presented by Borcherdt (1994) and utilized by Richardson et al. (1994). This criteria is shown below.

SITE STIFFNESS CLASSIFICATION (from BORCHERDT, 1994)

Site Classification	Average Shear Wave Velocity
Special Study*	Less than 330 ft/s
Soft	330 to 660 ft/s
Medium Stiff	660 to 1,230 ft/s
Stiff	1,230 to 2,300 ft/s
Rock	Greater than 2,300 ft/s

*Classification includes liquefiable soils, sensitive clays, peats, organic clays, high plasticity clays (PI > 75), and soft soil deposits greater than 120 feet (37 meters) thick.

4. Estimate peak free field (ground surface) acceleration using empirical charts (e.g., Figure 8a) and the site classification or conduct a seismic response analysis. The peak free field acceleration corresponds to the bedrock acceleration propagated upward through the existing soils at the site. A one-dimensional seismic response analysis could be conducted using the computer

program SHAKE (Schnabel et al., 1972) or WESHAKE (Sykora et al., 1992).

4. Estimate peak acceleration at the top of the landfill or cover using an empirical chart (e.g., Figure 9) or a seismic response analysis. The empirical chart or site response analysis can be used to estimate whether the earthquake acceleration will amplify or attenuate through the waste.

   The Singh and Sun (1995) charts in Figure 9 show an approximate relationship between maximum acceleration at the base of the landfill and the landfill crest for waste heights of 100 (Figure 9a) and 200 (Figure 9b) feet and waste shear wave velocities of 400 and 700 feet/second. For 100 feet of waste, acceleration at the crest of the waste is greater than the acceleration at the base. For 200 feet of waste, the crest acceleration is less than that at the base. If the landfill height and shear wave velocity differ significantly from that used by Singh and Sun (1995), a seismic response analysis should be conducted or the relationship proposed by Harder (1991) can be used (see Figure 9c). Harder (1991) presents a relationship for earth dams in which the acceleration recorded at the base (or abutment) of an earth dam is compared with those recorded at the crest of the dam. This upper bound correlation for earth dams can be used to estimate an upper bound for the seismic response of landfills. The basis for applying the upper bound correlation for earth dams to landfills is that it is probably conservative because (1) the two-dimensional geometry of a typical earth dam will probably exhibit a greater response than that of a landfill and (2) the refuse will exhibit a higher damping than that of an earth dam. Figure 9c also shows the correlation proposed by Seed and Idriss (1982) for deep cohesionless sites and the relationship for 100 feet of waste with a shear wave velocity of 700 fps from Figure 9a.

The empirical chart developed for soft soil sites by Idriss (1990) is shown in Figure 10. Richardson et al. (1994) suggest using this chart to represent municipal solid waste (MSW). Use of the median relationship in Figure 10 suggests significant amplification of earthquake acceleration for rock accelerations less than or equal to 0.4g.

5. Determine the maximum value of average horizontal earthquake acceleration, $a_{avg}(z)$ in Figure 11, at the base of the slip surface. This can be accomplished using an empirical correlation chart (e.g., Figure 11) or a one dimensional seismic response analysis software program (e.g., SHAKE or WESHAKE).

   The one-dimensional seismic response analysis is conducted using a column through the critical cross-section of the landfill. The acceleration at the base of the slip surface, $a_{avg}(z)$, is estimated directly from the seismic response analysis. A one-dimensional response analysis requires the dynamic properties of the materials in the column to be input along with an earthquake acceleration time history. Section 5.0 of this document discusses the dynamic properties of waste. The computer model SHAKE assumes that the ground surface is level and soil or waste layers through a critical cross section are horizontal.

   Use of empirical charts to estimate the maximum average horizontal acceleration at the base of the slip surface involves: (1) using an empirical chart, e.g., Figures 8a, 9, and 10, to estimate the peak acceleration at the top of the landfill, $a_{avg}(z=0)$ in Figure 11, and (2) using an empirical chart, e.g., Figure 11, to estimate the average acceleration at the base of the slip surface, $a_{avg}(z)$. In Figure 11, $a_{avg}(z=0)$, and the ratio of the depth of the slip surface (z) to the height of the landfill (h) can be used to estimate the average acceleration at the base of the slip surface at depth z, i.e., $a_{avg}(z)$. The value of $a_{avg}(z)$ divided by gravity can then be substituted for the parameter K_{max} (maximum value of average acceleration coefficient for the critical sliding mass) in Figure 8 to estimate the permanent seismic deformation using K_y and the appropriate earthquake magnitude.

6. Determine K_y by inputting values of seismic coefficients into the static slope stability software until a F.S. of unity is obtained. K_{max} is sometimes used in determining a starting value for K_y in this trial and error process. It should be noted that K_y is constant during the earthquake event chosen and is independent of K_{max}.

   For a landfill cover, an infinite slope analysis can be used to determine K_y. Matasovic (1991) modified the infinite slope equation to include a seismic coefficient as shown below:

$$FS = \left[\frac{C_i}{\gamma * z * \cos^2 \beta} + \tan\phi_i \left(1 - \gamma_w \frac{(z - d_w)}{(\gamma * z)} \right) - k_y \tan\beta * \tan\phi_i \right] * (k_y + \tan\beta)^{-1}$$

   where F.S. = factor of safety, C_i = interface cohesion, ϕ_i = interface friction angle, γ = unit weight of cover material, z = depth to interface/failure surface, γ_w = unit weight of water, k_y = seismic coefficient, d_w = depth to water, and β = slope angle.

7. Use the empirical deformation chart by Makdisi and Seed (1978) in Figure 8 using the ratio of K_y / K_{max} to determine an estimate of the associated cumulative permanent displacement along the critical slip surface for a selected maximum earthquake event. This chart uses the ratio of the coefficients (or accelerations if each coefficient is multiplied by gravity) as related to several earthquake magnitudes, each having a different range of cumulative earthquake deformation for various values of the ratio of K_y / K_{max}.

8. If desired or required one can use a more detailed calculation of cumulative seismic deformation for the regulatory maximum earthquake event by double-integrating the acceleration time history for the critical slip surface at depth z. The effects of the average acceleration time history for the critical slip surface should be used and is calculated from recording over time the shear stress at the critical slip surface divided by the total vertical stress above the slip surface.

To estimate the permanent seismic deformation of a liner slip surface consider the following:

- Peak bedrock acceleration (A_{max}) divided by g is used as K_{max} for a rock foundation site.

- For a soil foundation site the free field acceleration divided by gravity is used as K_{max} and is determined using empirical charts or the SHAKE model.

- Use appropriate K_{max} and K_y and the Makdisi and Seed (1978) empirical correlation chart to estimate permanent deformation.

To estimate the permanent seismic deformation of a cover slip surface consider the following:

- Estimate peak acceleration at the top of the landfill or in cover using empirical charts (e.g., Figure 9 from Singh & Sun, 1995 and/or Figure 10 from Idriss, 1990) or a seismic response analysis. Divide the peak acceleration in the cover by gravity to obtain K_{max}.

- Use appropriate K_{max} and K_y and the Makdisi and Seed (1978) empirical correlation chart to estimate permanent deformation. As described previously, a simplified analysis is warranted for preliminary analysis, routine projects and small landfills with minimal impact.

- Alternatively, use a one-dimensional earthquake response model like or better than SHAKE to estimate the maximum horizontal acceleration at the top of the landfill. Divide the peak acceleration in the cover by gravity to obtain K_{max}. As described previously, a one-dimensional analysis is warranted on medium-sized landfills or when the simplified procedure indicates potential problems, i.e., excessive deformations.

- Alternatively, use a two-dimensional earthquake response computer model like or better than FLUSH (Lysmer et al., 1995) to estimate the time history at the top of the landfill and double integrate the time history to estimate the permanent deformation. As described previously, a one-dimensional analysis is warranted on large or important landfills where there might be significant impact from slope failure, or when the one-dimensional analysis indicates potential problems, i.e., excessive deformations.

- Alternatively, use a three-dimensional earthquake response computer model like or better than TLUSH (Kagawa et al., 1981) to estimate the time history at the top of the landfill and double integrate the time history to estimate the permanent deformation. A three-dimensional analysis is used primarily for research purposes.

To estimate the permanent seismic deformation of a <u>waste</u> slip surface consider the following:

- Estimate peak acceleration at the top of the landfill or in cover using empirical charts (e.g., Figure 9 from Singh & Sun, 1995 and/or Figure 10 from Idriss, 1990) or a seismic response analysis.

- Estimate the average acceleration at base of slide mass using the chart proposed by Kavazanjian and Matasovic (1995) in Figure 11. Divide this average acceleration at the base of the slide mass by gravity to obtain K_{max}.

- Use appropriate K_{max} and K_y and the Makdisi and Seed (1978) empirical correlation chart to estimate permanent deformation.

- Alternatively to the chart proposed by Kavazanjian and Matasovic (1995) in Figure 11, one may use a one-dimensional earthquake response model like or better than SHAKE to estimate maximum horizontal equivalent acceleration (equivalent to the maximum horizontal shear stress/density multiplied by depth) at the base of the critical slip surface. Divide this maximum horizontal equivalent acceleration at the base of the critical slip surface by gravity to obtain K_{max}.

- Alternatively, use a two-dimensional earthquake response computer model like or better than FLUSH (Lysmer et al., 1995) to estimate the time history at the base of the critical slip surface and double integrate the time history to estimate the permanent deformation.

4.0 TENSILE STRESSES IN GEOSYNTHETIC MATERIALS

Resultant force and tensile stress determination methods for geosynthetic materials in liner and cover systems include the following:

1. Unbalanced Shear Method (Giroud and Beech 1989) - This is an infinite slope limit equilibrium method, where the tensile shear stress generated in the geosynthetic is the difference between the resultant applied/driving shear stress above the geosynthetic material and the resultant resisting shear stress below the geosynthetic. If the resultant applied/driving shear stress is less than or equal to the resultant resisting shear stress (F.S. $\geq$ 1), then the resultant tensile shear stress within the geosynthetic is zero. Since it is an infinite slope analysis it is usually conservative. Note, this method should not be used to determine the distribution of tensile stresses among multiple layers of geosynthetics.

2. Limit Equilibrium Method (Koerner and Hwu 1991) - This method is similar to the two-dimensional limit equilibrium methods described in Section 2.0. This method uses horizontal and vertical force equilibrium to define the resultant shear stress conditions along a slope using the critical non-circular slip surface. As a result, this method can incorporate the effects of a toe buttress on the stability, whereas the infinite slope method cannot. If the resultant driving stress exceeds the resultant resisting stress there will be a resultant tensile shear stress within the geosynthetic above the critical slip surface.

3. Displacement Compatibility Method (Long et al. 1993 and 1994) - This is a modified version of the previous two methods where an additional resisting shear stress is determined from a relationship involving the axial compressive stiffness of the soil above the geosynthetic and the axial tensile stiffness of the geosynthetic. There are finite element and finite difference computer models available for this approach for determining tensile stresses in multiple layers of geosynthetics.

Generally higher geosynthetic tensile stresses are determined using the unbalanced shear method versus the limit equilibrium method. The magnitude of the geosynthetic tensile stresses determined from the displacement compatibility method is somewhere in between the other two methods.

In general the effects of settlement on geosynthetics in a side slope liner system include:

- o From the top to the bottom of the side slope, the displacements increase and the tensile stresses decrease in the geosynthetics. From the point where tension in the geosynthetics is zero along the side slope, i.e., the transition point, the geosynthetics move with the waste down-slope.

- o The maximum amount of shear stress applied by the MSW up-slope of the transition point is less than it is down-slope.

- o Maximum displacement of MSW relative to the underlying geosynthetic occurs up-slope of the transition point and gradually tapers off down-slope.

Resultant force and tensile stress determination methods for geosynthetic materials in liner systems when settlement on side slope liner system occurs include the following:

1. Method to Identify Plane (or Planes) of Sliding - Determine the applied shear stress from MSW settlement. Determine interface resistances of the liner system. Compare applied shear stress to resistance of the weakest interface.

2. Specified Displacement Method (Richardson and Koerner 1987) - Determine the axial strain in the geosynthetics, by assuming the geosynthetics displace with the waste. The plane of slippage is assumed to be below each geosynthetic. The displacement or change in length of each geosynthetic occurs from the top of the side slope down to the point where the geosynthetic moves with the waste. Axial strain is the change in length divided by length between the anchor trench and the top of the side slope for each geosynthetic. Plot the tensile stress versus axial strain relationship for its respective plane of slippage. Determine the magnitude of required tensile strength in each geosynthetic by the following relationship:

   Subtract the resultant resisting shear stress below the plane of slippage from the resultant driving shear stress above the plane of slippage. The resultant driving stress above the plane of slippage is derived from the reduction of the applied shear load from MSW settlement by the overlying interfaces resisting shear stresses.

Compare the required tensile strength in each geosynthetic with the peak and residual tensile strength as obtained from the tensile stress versus axial strain relationship for each geosynthetic.

3. Specified Load Method (Giroud and Beech 1989) - Determine the tensile load resulting from waste settlement in each geosynthetic of the side slope liner system. In this method the plane of slippage is below the MSW mass at an interface between two of the geosynthetics of the liner system. Determine the applied/driving shear stress applied on each interface of the liner system. Determine where this applied/driving shear stress exceeds the resisting stress of an interface. Determine the applied/driving shear loads on the geosynthetics above this interface. Compare the load on each geosynthetic with the known tensile strength of the respective geosynthetic to determine tensile failure.

4. Strain Compatibility Method (Long et al. 1994) - Use this method when methods 2 and 3 above result in unacceptable tensile stresses. Predict load in geosynthetics above plane of sliding by requiring loads and displacements to be compatible. Determine the interface plane of slippage as in the specified load method. Estimate the magnitude of unbalanced shear or the applied/driving shear loads on the geosynthetics above this interface. Determine the extension of each geosynthetic to down-slope distance (axial strain) for each geosynthetic. From an estimate of waste settlement along sides of landfill, determine length over the side slope over which the waste will exert downdrag forces on the geosynthetics. This zone of downdrag forces or unbalanced shear between waste and geosynthetics is in the up-slope area of the waste. MSW and geosynthetics move differentially with tensions and strains being generated in the geosynthetics along the length of slope in this zone. Determine tension and strains in geosynthetics components.

For any of the resultant force and stress determination methods for geosynthetic materials, to minimize tensile stresses in the geomembrane the weakest interface should be located above the geomembrane.

5.0 ENGINEERING PROPERTIES OF MUNICIPAL SOLID WASTE

I. STATIC PROPERTIES OF WASTE

The important properties of municipal solid waste (MSW) for stability analyses are composition, particle size/distribution, unit weight (γ), MSW age, moisture content (ω), cohesion (c), friction angle (ϕ), effective overburden stress (σ_{Vo}'), degree of compaction, and shear strength $\tau_{max\ and\ residual}$. However, the essential properties for a stability analysis are γ, c, and ϕ. An example of γ's of MSW versus depth is included in Figure 12. Figure 13 presents a relationship between MSW dry unit weight, normal stress, and degree of compaction. Some of the sampling methods available to estimate γ are: standard penetration test (STP) split spoon sampling; bucket auger, and test pits. References on waste properties are included in Appendix A at the end of this technical bulletin. Figure 14 presents a plot of values of c versus ϕ for MSW. Figure 15 presents a plot of shear strength versus normal stress for MSW from large-scale laboratory testing. This friction angle was then used to estimate the corresponding cohesion. From Figure 15 it can be seen that the shear strength data defines a narrow band with a friction angle, ϕ, of approximately 35 degrees and a cohesion that ranges from 5 to 45 kPa. The data in Figure 15 suggests that a reasonable (solid trend line) combination of c and ϕ for municipal solid waste is 500 psf (25 kPa) and 35 degrees, respectively. The back-calculated shear strength for a recent landfill failure in Cincinnati is also shown in Figure 15 and plots slightly above the trend lines. This suggests that this reasonable combination of c (500 psf) and ϕ (35 degrees) may still be conservative. This combination is slightly higher than combinations published by others such as Singh and Murphy (1990), Kavazanjian et al. (1995), and Houston et al. (1995).

II. DYNAMIC PROPERTIES OF WASTE

Properties important to dynamic analysis of municipal solid waste are shear modulus, damping ratio, and the effect of cyclic strain. Maximum shear modulus (G_{max}) equals the density of the MSW multiplied by the shear velocity (V_s) squared. The energy imparted is equal to two times G_{max} multiplied by the quantity one

plus Poisson's Ratio. Presently there are a number of uncertainties in evaluating dynamic properties of MSW, such as inhomogeneity and lack of data on large or representative samples.

The wave velocities that can be measured in MSW are Rayleigh (surface) wave velocity (V_r), shear wave velocity (V_s), and compression or p wave velocity (V_p). Sixty-seven percent (67%) of the total seismic energy delivered to the waste is from the Rayleigh wave. Twenty-six percent (26%) of the total seismic energy delivered to the MSW is from the shear wave. The remaining 7% is from the p wave. Testing principles for V_r involve the ratio of the amplitude of the Rayleigh wave at depth z to the amplitude of the Rayleigh wave at the surface, and there are horizontal and vertical components of V_r for various Poisson's Ratios.

Laboratory methods for measuring V_r and V_s with depth include cyclic triaxial, cyclic simple shear, and resonant column. Seismic field methods include refraction, surface wave testing, cross-hole method, and down-hole method. Other field methods include analysis of steady state surface waves, and spectral analysis of surface waves (with outputs of time histories and Fast Fourier Transforms).

A trial-and-error surface wave analysis of MSW involves:

1. Assuming a layering profile consisting of V_s for each layer, mass density for each layer, and Poisson's Ratio for each layer.

2. Calculation of a theoretical dispersion curve.

3. Comparison of the theoretical dispersion curve to the experimental dispersion curve to see if they agree and assumed layering profile is representative of actual field condition.

4. If no agreement repeat steps 1 through 3. Generally mass density and Poisson's Ratio for each layer of the profile do not have to be adjusted if no agreement.

V_s generally appears to increase linearly with depth. The older (more compacted) the MSW is, the higher the values of V_s. Figure 16 presents a relationship between shear wave velocity and depth. As a first approximation the dynamic shear strength of waste can be assumed to be at least 80% of the static shear strength of MSW. The relationship between degradation of shear modulus and shear strain in MSW has been proposed by a number of researchers. Idriss et al. (1995) presents degradation relationships based on seismic recordings at the OII landfill near Los Angeles, California (see Figure 17).

6.0 SHEAR STRENGTH OF GEOSYNTHETIC INTERFACES

Laboratory methods to measure geosynthetic interface shear strengths include the following:

- o Large-Scale Direct Shear (12″ by 12″ square).
- o Small-Scale Direct Shear (4″ by 4″ square).
- o Torsional Ring Shear.
- o Pullout Tests (not widely used).

The ASTM Standard Test Method (ASTM D5321) for shear testing of geosynthetics requires the use of a 12 inch by 12 inch direct shear apparatus. This is one reason for the large-scale direct shear apparatus being the most widely used apparatus for shear testing of geosynthetics. Other advantages of the large-scale direct shear apparatus are a large specimen and commercially available from several manufacturers. Some of the limitations of the large-scale direct shear box are cost, limited normal stress, machine friction, load eccentricity, and limited continuous shear displacement. The limitations of the small-scale direct shear apparatus are similar to the large-scale apparatus and the use of a small specimen.

Some advantages of the torsional ring shear test apparatus are: there is continuous shear deformation in one direction; the τ versus displacement relationship for each test can be taken well beyond τ_{max} to $\tau_{residual}$; a constant cross-sectional specimen area; and supervision is usually minimal. Limitations of the ring shear apparatus are: there is no ASTM standard; the specimen is small; anisotropic shearing can occur with some geosynthetics; and availability of the test apparatus.

Generally, three or more direct shear or ring shear tests are conducted at different effective normal stresses to determine the peak and residual failure envelopes. A separate test specimen should be used for each effective normal stress (σ_n'). When conducting a direct shear or ring shear test the unit weight, water content, soil type, and geosynthetic should remain constant, while only σ_n' varies from test to test.

Field tests to measure interface shear strengths can provide representative conditions/results, but are expensive, difficult to control, and the interpretation of the results may be difficult because of unforeseen boundary conditions. Back-analysis procedures can be performed on field tests of a planned failure (e.g., a test pad scenario) and a test without failure, but back-analysis of an unplanned failure is difficult. There are examples of controlled field tests and tilt-table tests available. Field tests must be well planned ahead of time with: extensive analysis, sensitivity studies, determination of properties of soil and all interface strengths; and all important aspects must be documented. There should be more than one test section if possible including a proof or control test. Failure envelopes for materials and interfaces can come from literature, but it is best to determine site-specific failure envelopes for materials and interfaces.

I. NEED FOR SITE SPECIFIC TESTING

Stark and Poeppel (1994) present interface test results for the 1988 slope failure at the Kettlemen Hills Hazardous Waste Repository in California. They found that the τ_{max} or peak failure envelope at low σ_n's (say < 6000 psf) for the site specific geonet/geomembrane interface is the weakest interface (lowest shear strength envelope). For a τ_{max} envelope at higher σ_n's (say > 6000 psf) the site specific clay/geomembrane interface in the liner is the weakest interface (lowest shear strength envelope). This is in agreement with field observations that show the failure occurred along the clay/geomembrane interface in the liner at the Kettlemen Hills site. <u>As a result, site specific interface testing is required to determine the critical interface and the range of normal stresses over which the interface is critical.</u> This is important because the critical interface can change depending on the normal stress.

Stark and Poeppel (1994) also showed there is little difference in $\tau_{residual}$ envelopes for the geotextile/geomembrane, clay/geomembrane, and geonet/geomembrane interfaces at σ_n's less than approximately 3500 psf. For a $\tau_{residual}$ envelope at σ_n's greater than approximately 3500 psf, the clay/geomembrane interface in the liner is the weakest interface (lowest $\tau_{residual}$ failure envelope). By increasing the plasticity index of the clay from 44 to 63 and the liquid limit from 65 to 87, the $\tau_{residual}$ failure envelope of the clay/geomembrane interface increased. (The reason for this increase with increasing clay plasticity is discussed in Section II. titled "Shear Behavior of Clay/Geomembrane Interface".) These conclusions were derived based on testing conducted for the Kettlemen Hills site.

In summary, it is recommended that published and/or manufacturers shear strength data not be used for stability evaluations required in the permit process. In addition, manufacturer's data is usually accompanied by disclaimers that typically state that the information should not be relied upon to determine final design parameters and that project-specific shear testing should be conducted for this purpose. As a result, <u>it is recommended that shear strength testing using project-specific materials under appropriate conditions, including normal stress, moisture content, unit weight, hydration, and shear procedure, be conducted.</u>

Ideally site specific interface testing should be conducted during the design phase and before submission of the permit application. However, rapid changes in geosynthetic products and pricing may preclude determination of site-specific materials in the design phase. As a result, it may be acceptable to utilize conservative manufacturer's or published test results during the design phase and conduct site-specific interface testing <u>BEFORE</u> the start of construction. The interface testing conducted prior to construction should be used to ensure that actual interface strengths are greater than the manufacturer's or assumed values used in design. If the manufacturer's values are not obtainable prior to construction, the facility may need to be redesigned.

II. SHEAR BEHAVIOR OF CLAY/GEOMEMBRANE INTERFACE

Stark (1996) showed the $\tau_{max\ or\ residual}$ failure envelopes of the clay/smooth geomembrane or clay/textured geomembrane interface decreases with increasing water content in the clay. The residual interface strength increases with increasing σ_n', while the interface residual shear strength decreases with increasing water content of the clay. Other relationships/behaviors of the clay/geomembrane interface presented by Stark (1996) include:

- o $\tau_{max\ or\ residual}$ failure envelopes decrease with increasing clay compaction water content;

- o Increasing σ_n' increases the $\tau_{max\ or\ residual}$ failure envelopes at a given clay compaction water content.

- o $\tau_{max\ or\ residual}$ failure envelopes are nonlinear.

Clay plasticity has an interesting and somewhat unexpected effect on the peak and residual failure envelopes. Clay/textured geomembrane interface $\tau_{max\ or\ residual}$ failure envelopes associated with high plasticity clay can be greater than clay/textured geomembrane interface $\tau_{max\ or\ residual}$ envelope associated with low plasticity clay. (It should be noted that the $\tau_{max\ or\ residual}$ failure envelopes of high plasticity clay are usually less than $\tau_{max\ or\ residual}$ failure envelopes of low plasticity clay.) For the high plasticity clay, failure occurs in the clay and not at the clay/textured geomembrane interface as observed for the low plasticity clay. Failure occurred in the clay because the high plasticity soil bonded to the geomembrane and filled in the voids around the texturing. The clay, even though it exhibits a high plasticity, is stronger than the low plasticity clay/textured geomembrane interface. This is true for both $\tau_{max\ or\ residual}$ failure envelopes.

Clay/smooth geomembrane or clay/textured geomembrane interface efficiency equals $\phi_{interface}$ / ϕ_{soil} times 100. In general, peak efficiency for a clay/smooth geomembrane or clay/textured geomembrane interface is about 55%. The residual efficiency for the clay/smooth geomembrane or clay/textured geomembrane interface is about 35%, which is the same for geosynthetic interfaces.

Chemical immersion effects on interface strength can be illustrated by the example of $\phi_{interface}$ / ϕ_{sand} which increases as the Shore D hardness of the geomembrane material decreases. Shore D Hardness is measured in accordance with ASTM Standard Test Method D2240-85 for each geomembrane polymer to obtain an index value of surface hardness. The Shore D Hardness test is performed simply with a handheld indentation device.

For technical discussion on shear strength characteristics of sand/geomembrane interfaces see reference O'Rourke et al. (1990).

III. SHEAR BEHAVIOR OF TEXTURED GEOMEMBRANE/NONWOVEN GEOTEXTILE INTERFACES

The following describe the effects of textured geomembranes in textured HDPE geomembrane/nonwoven geotextile interfaces as described by Stark et al. (1996):

- Textured HDPE geomembranes usually increase the F.S. by a factor of 1.5 to 2.0 over untextured or smooth HDPE geomembranes.

- For textured geomembrane/geotextile interfaces, the τ_{max} envelope typically exhibits a peak ϕ' of 32°, which is a significant increase over untextured geomembrane/geotextile interfaces (peak ϕ' =9°).

- For textured geomembrane/geotextile interfaces, once τ_{max} is passed in the stress/displacement relationship, the τ decreases with increasing shear displacement until $\tau_{residual}$ is reached. The $\tau_{residual}$ failure envelope for the textured geomembrane/geotextile interface tested by Stark et al. (1996) has an average residual ϕ' of 13°. The magnitude of displacement affects the magnitude of strength loss, and thus interface strength.

- Textured HDPE geomembrane/non-woven geotextile or textured HDPE geomembrane/drainage geocomposite interfaces exhibit a post-peak strength loss of 50 to 60 percent. The post-peak strength loss is primarily attributed to the pulling out or tearing of fibers from the non-woven geotextile and orienting them parallel to shear. However, polishing of the geomembrane texturing may also contribute to the strength loss. [*Note that the drainage geocomposite in this discussion consists of non-woven geotextile over geonet over non-woven geotextile with the geotextiles heat bonded to the geonet.*]

- Shear stresses imposed on the textured geomembrane interface must be resisted in part by the texturing on the surface of the geomembrane. At high normal stresses, the applied shear stress can remove or damage some or all of the texturing. The removal or damage of the texturing appears mainly applicable to textured HDPE geomembranes created by the lamination and impingement techniques. Therefore, site specific laboratory interface shear tests should accurately simulate field conditions to understand the performance of the materials involved. For example, the interface should be tested at the field normal stress with the delivered geomembrane and geotextile to investigate field performance and possible damage to the texturing.

- Textured HDPE geomembrane/non-woven geotextile interface failure envelopes can be nonlinear. It is recommended that the entire failure envelope or a friction angle that corresponds to the appropriate normal stress be used in stability analyses.

- The mass per unit area, polymer composition, fiber type, and/or fabric style of the non-woven geotextile can influence the peak textured geomembrane/non-woven geotextile interface shear resistance. For example, continuous large denier fibers appear to result in a higher peak interface shear strength than staple fibers.

- The residual textured geomembrane/non-woven geotextile interface shear resistance appears to be independent of fiber type, fabric style, and mass per unit area. However, polyester geotextiles appear to yield a higher residual interface shear strength than comparable polypropylene geotextiles.

- Calendering of a non-woven geotextile can increase the textured geomembrane/non-woven geotextile peak and residual interface strengths by 10-20% and 20-30%, respectively.

- A non-woven geotextile mass per unit area of 270 g/m^2 appears to result in higher peak interface strengths than a 540 g/m^2 nonwoven geotextile at normal stresses greater than 100 kPa. This suggests that a lower nonwoven mass per unit area or thickness may be more desirable for liner systems. At normal stresses less than 100 kPa, there appears to be negligible difference between the peak interface strengths with these nonwoven geotextiles.

- Coextruded and laminated textured geomembranes exhibit similar peak shear strengths for geomembranes/nonwoven geotextile interfaces if the texturing is not damaged or removed from the laminated product. If the texturing is damaged or removed, the coextruded geomembrane exhibits a higher peak interface strength.

The coextruded geomembrane appears to yield a higher residual interface shear strength than the laminated geomembrane.

O The presence of a drainage net in a drainage geocomposite does not significantly increase the textured geomembrane/non-woven geotextile interface shear resistance for normal stresses less than 500 kPa. However, the drainage net can facilitate damage to or removal of the texturing from a laminated or impingement geomembrane at normal stresses lower than would be required for texture damage or removal with only a non-woven geotextile.

In summary, textured geomembranes made by the coextrusion method is preferred because the texturing cannot be removed. The roughness of a textured geomembrane can be measured with a Geosynthetics Research Institute at Drexel University micrometer with a pointed or rounded tip.

7.0 SHEAR STRENGTH OF GEOSYNTHETIC CLAY LINERS

In recent years, geosynthetic clay liners (GCLs) are increasingly being chosen to replace or augment compacted clay liners (CCLs) in composite liner and cover systems. Some of the advantages of GCLs over CCLs are: (1) lower and more predictable cost, (2) prefabricated/manufactured quality, (3) easier and faster construction, (4) lack of need for field hydraulic conductivity testing, (5) availability of engineering properties, (6) resistance to the effects of wetting/drying and freeze/thaw cycles, (7) more airspace resulting from smaller thickness, and (8) easier repair. Some of the disadvantages of GCLs versus CCLs include: (1) a potential for low internal shear strength [hydrated ϕ' of bentonite is 5° to 7° while unhydrated ϕ' of bentonite is 20° to 30°], (2) a possible large post-peak shear strength loss in reinforced products caused by needle punched or stitch-bonded fibers breaking and/or pulling out from the bentonite, (3) bentonite can squeeze through upper woven or non-woven geotextile at low effective stress creating a weak interface above the GCL (Byrne, 1994), (4) smaller leachate attenuation capacity, (5) shorter containment time, and (6) possibly higher long-term flux because of a reduction in bentonite thickness from uneven overburden stresses and/or uneven hydration (Koerner and Narejo, 1995; Fox et al.,1997; Anderson, 1996; Anderson and Allen, 1995). Stress concentrations are usually present around sumps, pipes and piping trenches, slope transitions, and slope benches. Stress concentrations can also be induced by a subgrade that contains stones or is uneven and/or contains ruts. Geomembrane wrinkles create zones of nonuniform stress which can allow hydrated bentonite to flow to points of lower effective stress under the wrinkle.

There is a new GCL concept being tested, and commercially available in limited quantities, of heat bonding a geonet to a geotextile or geomembrane and filling the geonet openings with bentonite. After this filling process, a geotextile is bonded to the other side of the bentonite filled geonet (Stark 1997a). The geonet/internal structure increases the internal shear strength of the GCL and protects the hydrated bentonite from migrating to a zone of low effective stress (Stark 1997b).

This section of the guidance document is intended to make regulatory concerns about GCLs known and to provide design and testing recommendations to alleviate these concerns and facilitate regulatory review. For the purposes of this document, GCLs can be grouped into two broad categories, reinforced and unreinforced. Reinforced GCLs are basically comprised of three components, bentonite sandwiched between two geotextiles, with reinforcement to provide additional strength to the bentonite.

The reinforcement consists of intermittently stitching the three components together (stitch bonding), or a ubiquitous matrix of fibers punched through the three layers (needle punching). Unreinforced GCLs consist of bentonite sandwiched between two geotextiles with no reinforcement or bentonitic clay soil adhered to a geomembrane.

The following two sections present recommendations for designing slopes that contain GCLs and for shear testing of GCLs to estimate the shear strength parameters required for stability analyses.

I. GCL SLOPE DESIGN

Little is known about the long term performance of GCLs. This issue is discussed at length in U.S. EPA's *Report of 1995 Workshop on Geosynthetic Clay Liners*, dated June 1996, and in the American Society for Testing and Materials (ASTM) Special Testing Publication No. 1308, *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, published in January of 1997. Additionally, there appears to be a growing opinion among researchers in the GCL arena that it may be prudent to evaluate post-peak strength conditions as well as peak conditions. This is due to uncertainties surrounding the processes that may initiate deformations in composite lining systems during construction, waste placement, and the waste's subsequent settlement. These processes may result in the development of post-peak or residual shear strength conditions.

Because of uncertainties and a lack of long term performance data, it is recommended that slopes be designed for post-peak conditions with a static factor of safety 1.3 for designs incorporating GCLs (Evans 1997). As mentioned previously, a permanent seismic deformation analysis is preferred to a pseudo-static stability analysis. In addition, the following conditions should be incorporated into the stability analysis (Evans 1997):

1. Potential pore-water pressure build up in the drainage layer must be taken into account when investigating the stability of the final cover system. Consideration of seepage forces should include an investigation of the maximum pore-water pressure that may build up in the drainage layer of the cover system based on the maximum fluid flux through the cover soils which could occur during saturated conditions and a major rain event. Recent observations suggest that seepage forces are important because a significant number of landfill final cover failures have occurred due to inadequate design of the drainage layer. The design inadequacies include underestimating the volume of water

that can permeate through the cover soils during a major rain event and/or inadequate controls for keeping the drainage layer from becoming partially or completely clogged throughout the life and post closure of the landfill.

2. Post-peak shear strength should be determined utilizing a shear displacement of at least 2 inches (50 mm) for the liner system and at least 1 inch (25 mm) for the cover system. It is anticipated that the shear stresses induced in the cover are less than liner and thus a smaller displacement can be used for the cover design.

3. Permanent seismic deformation in the composite cover system should not exceed 6 inches (15 cm) and deformation in the composite liner system should not exceed 4 inches (10 cm). The post-peak shear strength described above should be used in the permanent seismic deformation calculation.

II. SHEAR STRENGTH TESTING OF GCLs

The bentonite component of the GCL usually controls the shear strength characteristics of the composite liner and cover system. Hydrated bentonite has one of, if not the lowest peak and residual shear strengths of any soil (Mesri and Olson, 1970). In addition, bentonite has an extremely high affinity for moisture and it should be assumed that the GCL <u>will</u> hydrate. Bentonite also exhibits a large swell pressure (Stark 1997a) which can cause the hydrated bentonite to extrude from the GCL into the interfaces between the GCL and adjacent materials creating a weak shear surface (Byrne 1994).

Currently, no established or otherwise universally accepted test method exists for determining the internal shear strength and interface shear strength of a GCL. As a result, the following paragraphs present some recommendations for shear testing of GCLs based on Evans (1997).

A. Sample Selection

Samples should be selected from rolls that are delivered to the site. The next best alternative is to obtain identical product samples from another site. If either of the preceding options are unavailable, samples from the manufacturer may be used, if the manufacturer certifies that the samples are representative of materials that will be shipped to the field. This is important because the amount of reinforcement can vary significantly in the manufacturing process.

B. Hydration

It is recommended that project-specific GCLs and adjacent materials be allowed to fully hydrate, as a single unit, in a free swell condition until vertical expansion has essentially ceased (an inconsequential confining stress of no more than 0.5 psi to prevent sample deterioration or to provide a founding for displacement measurement is acceptable). The vertical expansion should be determined by monitoring vertical displacement until swelling has reached 100% primary as determined by ASTM 4546 and moisture samples should be taken from the hydrated GCL after the shear test to verify the degree of hydration.

C. Normal Stress
It is recommended that project-specific materials including soils and geosynthetics be tested for internal and interface shear strength (as appropriate) over the entire range of normal stresses that will be encountered in the particular design.

For cover systems, this includes the low normal stresses associated with these applications and any additional stresses which may be induced by surface water diversion benches, roads, equipment, or other structures constructed above the composite cover system.

For composite liner systems, the range of normal stresses which needs to be evaluated can be extensive, varying from low values at the perimeter of the fill to extremely high values under the deepest areas of the fill.

D. Shear Displacement Rate
Stark and Eid (1997) and Gilbert et al. (1997) show that the rate of shear displacement can greatly affect the measured shear strength of GCLs. Shear strength values from tests using a displacement rate of 1 mm/min, ASTM Standard rate, have been shown to be greater than values measured using slower displacement rates. Stark (1997) reports that rates equal to or less than 0.04 mm/min (0.0016 in/min) do not seem to have a detrimental affect on measured shear strength values of one reinforced GCL. U.S.EPA (1996) recommends ASTM D-3080 for determining the appropriate direct shear rate. For consistency, it is recommended that the ASTM D-3080 procedure for determining the appropriate direct shear rate for GCLs be used and the direct shear rate should not exceed 0.04 mm/min.

E. Test Method
The ASTM Standard Test Method (ASTM D5321) for shear testing of geosynthetics is usually used for determining internal shear strengths and interface shear strengths of GCLs. This

test method requires the use of a 12 inch by 12 inch direct shear apparatus. This is one reason for the large-scale direct shear apparatus being the most widely used apparatus for shear testing of GCLs. However, other techniques are also being utilized, e.g., peel tests (Richardson 1997), torsional ring shear tests (Eid and Stark 1997), and pullout shear machine (Fox et al. 1997).

III. AVOIDING BENTONITE MIGRATION

After GCLs have hydrated and stresses have been applied, the bentonite has been observed to migrate away from high stress concentrations, resulting in localized thinning of the GCL. This phenomenon is especially likely to occur in areas of composite lining systems where non-uniform stress concentrations typically develop. This includes areas in the immediate proximity of wrinkles, in and around sumps, and beneath leachate collection piping. Thinning of the GCL due to migration of the bentonite has been observed at two facilities in Ohio and one in Pennsylvania.

One-dimensional compression tests show that the thickness of a hydrated GCL can decrease significantly due to bentonite migration. This phenomenon has been evidenced in exhumed GCLs and has been commented on by numerous authors including Fox et al. (1997), Richardson (1997), Anderson (1996), Koerner and Narejo (1995), Stark (1998), and Anderson and Allen (1995). According to Fox et al. (1997), bentonite migration seems to be more pronounced in unreinforced GCLs than in reinforced GCLs. Anderson and Allen (1995) and Anderson (1996) also show that the thickness of a GCL can be significantly reduced in the vicinity of a wrinkle in the overlying geomembrane by the hydrated bentonite flowing up into the air space of the wrinkle, which may change shape but does not disappear according to Koerner (1996).

Thinning of the GCL has serious implications for meeting regulatory requirements and hydraulic performance. Therefore, it is recommended that the sump area and areas directly beneath leachate collection piping not incorporate GCLs, and that wrinkling of the geomembrane be kept to an absolute minimum. Since there will be design and construction difficulties associated with this recommendation, alternative approaches, including new types of GCLs will be considered.

IV. USE OF UNREINFORCED GCLs

Unreinforced GCLs lack any added reinforcement to resist shear stresses, such as needle punching or stitch bonding. As a consequence, these products have internal shear strength and bearing capacity characteristics approximately equivalent to hydrated bentonite. USEPA (1996) comments that shear strength data on unreinforced GCLs show hydrated friction angles of about 10 degrees. Richardson (1997) estimates the bearing capacity of a hydrated unreinforced GCL to be 40 kPa (825 psf) and the internal shear strength to be less than 5 kPa (100 psf) for low normal stresses such as those associated with covers. For these and other reasons, it is recommended that composite cover system designs do not incorporate unreinforced GCLs and that unreinforced GCLs be restricted to use on bottom lining slopes of less than 10%.

8.0 SEISMIC SOIL LIQUEFACTION

Liquefaction is the result of loose, saturated, cohesionless soils (gravel to silt size particles) being shaken by earthquake induced motion. In the equation $\sigma_n' = \sigma_T - u$, liquefaction occurs when σ_n' reaches zero due to an increase in u. The increase in u is caused by the loose cohesionless soil contracting under the shear stresses induced by the earthquake shaking and the increase in u caused by the contraction not being able to dissipate during the rapid earthquake shaking.

The following five criteria can be used to determine if a deposit is potentially liquefiable. These criteria are presented in the Draft Guidance, RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (Richardson et al. 1994). If a soil layer beneath a facility fails three or more of the below criteria than further investigation into the liquefaction potential of the layer is warranted.

1. **Criteria: Geologic age and origin** - The potential for liquefaction decreases with increasing age of the soil deposit. Table 2 presents the liquefaction susceptibility of soil deposits as a function of geologic age and origin (Youd and Perkins, 1978).

2. **Criteria: Fines content and plasticity** - The potential for liquefaction decreases with increasing fines content and increasing plasticity of the soil. (Fines content is defined as the percentage of material by weight passing the Number 200 sieve.) If the fines content is greater than 20%, the soil will not liquefy unless it is extremely sensitive (Seed and Idriss, 1982).

3. **Criteria: Saturation** - Full saturation is generally deemed to be a necessary condition for soil liquefaction. In performing the liquefaction analysis, the high seasonal water table should be used.

4. **Criteria: Depth below ground surface** - Liquefaction is generally not likely to occur more than 50 feet below the ground surface.

5. **Criteria: Soil penetration resistance** - Seed et al. (1985) indicate that liquefaction has not been observed in soil deposits having a normalized standard penetration test (SPT) blowcount, $(N_1)_{60}$, larger than 22. The blowcount is normalized to an effective overburden stress of one ton per square foot (tsf) and corrected to 60% of the theoretical energy. Liquefaction has not been observed in soil deposits having a corrected cone penetration test (CPT) tip resistance (q_{C1}) greater than 160 tsf or 15 mega Pascals (MPa) [Stark and Olson 1995].

Certain studies in China have indicate that clayey soils with a percent finer than 0.005 millimeter (mm) > 15%, liquid limit (w_L) $\geq$ 35%, and ω > 0.9 multiplied by the w_L may also be susceptible to liquefaction (Seed et al. 1985).

Observations of field case histories suggest that sand below a depth of 50 feet of soil (generally having a γ of 110 pounds per cubic feet) will not liquefy (i.e., when $\sigma_n' = 0$, and $\sigma_T = u$). It should be noted that this depth may be greater if the liquefiable layer is overlain by MSW because the unit weight of MSW (75 to 85 pcf) is less than soil.

Characteristics of liquefaction include:

- Flow failure,
- Lateral spreading, or retrogressive circular failure, and
- Level ground liquefaction, such as sand boils or ejection.

I. CALCUALITON OF FACTOR OF SAFETY AGAINST LIQUEFACTION

The factor of safety against liquefaction equals the shear stress or stress ratio required to cause liquefaction (CSR_l) divided by the earthquake induced shear stress or shear stress ratio

(CSR_{eq}), i.e., $CSR_1/CSR_{eq} = (\tau_{eq}/\sigma_O')_1 / [(0.65)(a_{max,z}/g)(\sigma_O/\sigma_O')]$. $\sigma_O = \sigma_T$ (total overburden stress), $\sigma_O' = \sigma_n'$ (effective overburden stress), $a_{max,z} = (A_{max}$ at $z=0)*(r_d)$ where A_{max} at z=0 is the maximum free field horizontal ground surface acceleration, and r_d is the empirical reduction factor. An empirical chart was developed by Seed and Idriss (1982) that relates r_d versus depth for level site conditions. The value of $a_{max,z}$ can also be obtained from a one dimensional wave propagation software SHAKE or equivalent. $a_{max,z}$ is projected from the ground surface to the liquefiable layer at depth z using the empirical reduction factor, r_d. $(\tau_{eq}/\sigma_O')_1$ is obtained from a chart of $(N_1)_{60}$ (Seed et al. 1985) versus $(\tau_{eq}/\sigma_O')_1$ (see Figure 18) or (q_{c1}) (Stark and Olson 1995) versus $(\tau_{eq}/\sigma_O')_1$ (see Figure 19) for Richter earthquake magnitude of 7.5.

To correct for earthquakes of other magnitude, apply the equation $CSR_1 = (CSR_1$ at $M=7.5)*(C_M)$, where C_M is a correction factor obtained from a table of M versus C_M (from Seed and Idriss, 1982 or Arango, 1996). Table 3 presents the magnitude scaling factors proposed by Arango (1996). The earthquake magnitude for a potential site can be estimated from the U.S.G.S. map (Algermissen et al., 1982) which identifies seismic source zones.

Table 3. Magnitude scaling factors proposed by Arango (1996)

EARTHQUAKE MAGNITUDE	EQUIVALENT UNIFORM NUMBER OF CYCLES	MAGNITUDE SCALING FACTORS
8.5	26	0.76
7.5	15	1.0
6.75	10	1.22
6	5-6	1.65
5.25	2-3	2.45

A correction of CSR_1 for an effective confining pressure other than 1 tsf is expressed by $CSR_1 = (CSR_1$ for $\sigma_O' = 1$ tsf$)*(K_\sigma)$, where K_σ is a correction factor obtained from a chart (see Figure 20) of σ_O' versus K_σ (from Seed and Harder, 1990).

The static shear stress is expressed using α which equals the initial driving stress on a horizontal plane, τ_{hv}, divided by initial effective overburden stress, or $\alpha = \tau_{hv} / \sigma_O'$. A correction of CSR_1 for static shear stress other than $\alpha = 0$ is expressed by CSR_1 = (CSR_1 for α = 0 tsf)*(K_α), where K_α is a correction factor obtained from a chart (see Figure 21) of α versus K_α (from Seed and Harder, 1990).

The field measured SPT blow count (N) values must be corrected for the hammer type release system, sampler configuration, short rod lengths, overburden stresses (σ_O' in tsf), and percent (60%) of theoretical energy. The hammer type release system typically used in the United States is a 140 pound safety hammer dropped 30 inches (generally two raps of rope around the cathead of the drill rig) to deliver an estimated rod energy (energy ratio, ER) of 60% for N blow counts ($N_{60\%}$) over a depth range of 6 to 18 inches. $N_{60\%}$ = N x ER/60% is used for correction to the hammer type release system. $N_{60\%SAMPLER}$ (corrected for sampler configuration) = $N_{60\%}$ x $C_{SAMPLER}$, where $C_{SAMPLER}$ varies from 1.1 for N $\leq$ 5 blows per foot to $C_{SAMPLER}$ = 1.25 for N $\geq$ 30 blows per foot. N values measured at shallow depths, using drilling rods less than about 10 feet in length, should be multiplied by 0.75 (decreased) to correct for dynamic inefficiencies inherent in "short rod" drilling systems. Correction for overburden stress σ_O', can be expressed as $(N_1)_{60\%} = N_{60\%} \times C_N$, where $C_N = 1/(\sigma_O')^{0.5}$ (Liao and Whitman 1985) where σ_O' is in units of tons/square feet. There are also charts for the overburden correction factor, C_N, which are reproduced in Seed and Harder (1990).

The cone penetration test (CPT) can be used to evaluate liquefaction potential by either converting the tip resistance, q_c, to an $(N_1)_{60}$ value or using the tip resistance normalized to an effective stress of 1 tsf.

There are charts available to convert cone tip resistance q_c to N_{60}, e.g., Stark and Olson (1995). CPT q_c is corrected for effective overburden stress by multiplying q_c by a cone resistance modification factor C_q , where C_q can be obtained from charts of C_q versus effective overburden pressure or an equation. For example, Kayen et al. (1992) proposed the following equation to describe the effective overburden stress correction factor proposed by Seed et al. (1983):

$$C_q = \frac{1.8}{0.8 + (\sigma'_{vo}/\sigma'_{ref})}$$

where σ'_{ref} is a reference stress equal to one atmosphere (approximately 100 kPa). q_{c1} is the designation of the corrected or normalized CPT tip resistance q_c.

As described previously F.S. for liquefaction = CSR_1/CSR_{eq} $=(\tau_{eq}/\sigma_O')_1 /[(0.65)(a_{max,z}/g)(\sigma_O/\sigma_O')]$. CSR_1 can be determined from relationships between q_{c1} and CSR_1 (e.g., Figure 19) for materials of different fines content and gradation, i.e., grain diameter at 50% passing.

II. POST LIQUEFACTION STABILITY ANLAYSES

If the soil layer is liquefiable one can conduct a post-liquefaction stability analysis, liquefaction settlement analysis, and liquefaction lateral deformation analysis.

Post-liquefaction stability analysis can be performed as follows:

- o Conduct a two-dimensional limit equilibrium analysis to identify the critical slip surface as described in Section 1.0.
- o Assign an undrained residual shear strength to the liquefiable layer that is encountered by the critical slip surface using Seed (1987) or Stark and Mesri (1992).
- o Assign a pseudo-static seismic coefficient of zero.
- o Conduct a two-dimensional limit equilibrium analysis to calculate the post liquefaction F.S. for the critical slip surface through the liquefied layer. The resulting F.S. should be greater than 1.2 or 1.3 depending on the amount of drainage that occurs after earthquake shaking stops.

An equation for estimating permanent horizontal liquefaction-induced displacement of gentle slopes above a liquefiable layer has been determined from Japanese data (Hamada et al. - 1987) for very loose sand deposits and ground slopes less than 10%. This equation is D (permanent horizontal liquefaction-induced displacement in meters) = $0.75 \times (H)^{0.5} \times (\theta)^{0.33}$ where H = liquefiable loose sand layer thickness in meters, θ = greater value of surface slope and gradient of the bottom of the liquefiable layer in percent.

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Appendix A

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3.0 PSEUDO-STATIC LIMIT EQUILIBRIUM ANALYSIS

Historically earthquake analysis has been accomplished by determining a pseudo-static (Terzaghi 1950) F.S. for the critical slip surface determined from the static limit equilibrium analysis described above. This would be accomplished by using a seismic coefficient multiplied by the acceleration due to gravity ($g = 32.2$ ft./sec.^{2}) that represents the maximum (or peak) horizontal earthquake acceleration from the regulatory maximum earthquake event (A_{max}). A typical A_{max} is a multiple of the peak horizontal bedrock acceleration, which comes from the Algermissen Chart or U.S.G.S. Map (1990) to estimate A_{max} for 10% exceedence in 250 years earthquake event. The seismic coefficient is the maximum horizontal earthquake acceleration acting at the center of gravity of the slide mass divided by gravity. The seismic coefficient can then be used to calculate a resultant dynamic driving shear force, shear moment or shear stress which can then be added to the static driving forces of the critical slip surface. The pseudo-static F.S. can then be calculated for the static critical slip surface as before except that the resultant resistant shear mechanism is divided by the resultant dynamic shear mechanism, which includes a horizontal force that equals weight of the slide mass multiplied by the seismic coefficient. This process for determining pseudo-static F.S. should be repeated for only the static critical slip surface. A search should not be conducted for the critical failure surface with a seismic coefficient.

Pseudo-static slope stability analysis shall not be used as the only method of seismic slope stability analysis. Pseudo-static slope stability analysis may be used as a supplement to a permanent deformation analysis. A deformation analysis is preferred because it provides a direct estimate of field performance while the pseudo-static F.S. gives an indirect estimate. In addition, estimation of the seismic coefficient is difficult. However, for completeness a simplified pseudo-static analysis procedure is as follows:

- o Pseudo-static F.S. is determined from the sum of the static resisting forces, stresses, or moments divided by the sum of the following:

1. Opposing/driving static stresses, forces, or moments plus;

2. The opposing/driving dynamic stress, force, or moment at the centroid of the slide mass calculated by multiplying the seismic coefficient at the centroid by the weight of the slide mass above the critical failure surface.

- Determine a seismic coefficient (n_g)which is an approximation of the maximum equivalent horizontal earthquake acceleration at the centroid of the static critical slide mass divided by g. An approximation of n_g is 0.5 multiplied by the appropriate maximum horizontal bedrock acceleration (A_{max}) divided by g.

- Determine the pseudo-static F.S. Use the same C2DLE software or equivalent software to determine the critical cross section and the critical circular and non-circular failure surface. Then estimate a pseudo-static F.S. for each critical slip surface. This pseudo-static F.S. is found by the computer software program using a seismic coefficient. (Again, to ensure convergence, a search should not be conducted for the critical failure surface with a seismic coefficient.)

- If the pseudo-static F.S. is greater than unity and 12 inches of permanent deformation is acceptable the slope is stable. If the pseudo-static F.S. is less than unity or 12 inches of permanent deformation is not acceptable a permanent deformation analysis should be conducted. Permanent deformation analyses are discussed in the following section.

The limitations of pseudo-static stability analysis method are as follows:

- Assumes horizontal force, derived from the seismic coefficient is permanent and acts in one direction.

- Pseudo-static F.S. is not a single value as calculated above, but varies with time over one earthquake event; movement only theoretically occurs when pseudo-static F.S. < one.

- Pseudo-static F.S. does not indicate displacements in slope after shaking. As a result, a pseudo-static analysis cannot be used to determine whether the sum of all displacements cause structural damage or a post-peak strength loss. Pseudo-static F.S. is for one point in time (one earthquake pulse).

- Sometimes with geocomposite/textured geomembrane interfaces and increasing σ_n', smoothing of the textured geomembrane surface at post peak τ_{max} shear failure can occur. This is caused by the texturing being removed from the geomembrane. This was observed for geomembranes manufactured using the lamination and impingement procedures. [*Note that the geocomposite in this discussion consists of non-woven geotextile over geonet over non-woven geotextile.*]

- For geocomposite/textured geomembrane interfaces as discussed here, as σ_n' is increased the stress/displacement plot increases.

- For geocomposite/textured geomembrane interfaces as discussed here, the geonet can facilitate removal of the texturing especially at high σ_n'. This causes the peak shear strength τ_{max} envelope to decrease to a $\tau_{residual}$ envelope after the texturing is removed.

Reinforced GCL’s can be used in cover systems because the normal stress is significantly less than in a liner system which should reduce the amount of bentonite migration. However, stress concentrations can occur in a cover due to an uneven subgrade. The internal or interface shear strength of the GCL will control the stability of the cover. Additionally, the low bearing capacity of reinforced GCL’s may result in thinning of the GCL from bentonite migration due to high stress concentrations from wheel loads that may be applied to a coverduring closure and post-closure. The GCL will probably hydrate under a confining stress significantly less than the swell pressure of the bentonite. Reinforced GCL’s may also undergo significant creep due to time-dependent deformational characteristics of hydrated bentonite resulting in extremely low post peak or residual strength conditions. Unreinforced GCL’s should not be used in the cover or liner system due to the low shear strength of bentonite.

For design purposes the reinforced GCL should be hydrated and tested at the low normal stresses associated with a cover. The GCL should be allowed to completely hydrate. Shearing should be allowed to occur through the bentonite or at the GCL/geomembrane interface to determine the critical failure mechanism. The resulting shear strength values should be used to evaluate the stability of the cover.

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