CHAPTER 8

DEEP-SEATED FAILURE ANALYSIS

This chapter provides information to use when analyzing the potential for deep-seated translational failures and deep-seated rotational failures under static and seismic conditions at an Ohio *waste containment facility*.

> Ohio EPA considers any failure that occurs through a material or along an interface that is loaded with more than 1,440 psf to be a deep-seated failure.

Deep-seated translational failures occur along the weakest interfaces or through the weakest foundation layers, especially if a foundation layer is relatively thin and underlain by stronger materials. Translational failures are more prevalent at facilities containing geosynthetics. This is because translational failures involve a planar failure surface that parallels the weak layer and exits through the overlying stronger material. Rotational failures occur through relatively weak layers of a foundation and possibly a relatively weak waste layer or engineered component of a *waste containment facility*. Rotational failures are more prevalent at facilities that are made of or filled with weak materials or are supported by relatively weak foundation soils. Rotational failures tend to occur through a relatively uniform material, where translational failures tend to occur when dissimilar materials are involved.

> Ohio has experienced at least 13 felt earthquakes since 1986. At least four of those exceeded magnitude 5.0 on the Richter scale. Ohio has experienced at least two earthquakes with ground accelerations exceeding 0.2 g since 1995. Ohio can also be strongly affected by earthquakes from outside the state, as occurred during 1811 and 1812, when large earthquakes estimated to be near 8.0 on the Richter scale occurred in New Madrid, Missouri damaging buildings in Ohio (from various publications from ODNR, Division of Geological Survey Web site).

The potential for a slope to have a deep-seated translational or rotational failure is dependent on many factors including, but not limited to, the angle and height of the slope, the angle and extent of underlying materials, the geometry of the toe of the slope, the soil pore water pressure developed within the materials, seismic or blasting effects, and the internal and interface shear strengths of the slope components. Failures of this type can be catastrophic in nature, detrimental to human health and the environment, and costly to repair. They can and must be avoided through state of the practice design, material testing, construction, and operations.

Ohio EPA requires that *waste containment facilities* be designed to withstand a plausible earthquake, because they are intended to isolate the public and environment from contaminants for a long time. The maximum magnitude of a plausible earthquake in Ohio, as of the writing of this policy, is expected to be 6.1 or higher on the Richter scale.

REPORTING

This section describes the information that should be submitted to demonstrate that a facility is not susceptible to deep-seated rotational and translational failures. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report:

> Any drawings or cross sections referred to in this policy that are already present in another part of the geotechnical and stability analyses report can be referenced rather than duplicated in each section. It is helpful if the *responsible* party ensures the referenced items are easy to locate and marked to show the appropriate information.

- A narrative summary of the results of the deep-seated failure analysis.
- One or more tables summarizing the internal and interface shear strengths of the various components of the *internal*, *interim*, and *final slopes* (e.g., see Table 6 starting on page 8-21);
- Graphical depictions of any individual and compound non-linear shear strength envelopes being proposed for each interface, material, or composite system (see Chapter 4, starting on page 4-15 for more information).
- One or more tables summarizing the results of the deep-seated failure analysis on all the analyzed cross sections (e.g., see Table 6 starting on page 8-23);
- The scope, extent, and findings of the subsurface investigation as they pertain to the analyses of potential deep-seated failures at the *waste containment facility*.
- A narrative description of the logic and rationale used for selecting the critical cross sections for the *internal*, *interim*, and *final slopes*.
- A narrative justifying the assumptions made in the calculations and describing the methods and rationale used to search for the worst-case failure surface in each cross section. This should include:
    - a description of the *internal*, *interim*, and *final slopes* that were evaluated,
    - the assessed failure modes, such as deep-seated rotational and deep-seated translational failures,
    - the site conditions that were considered, including, at a minimum, static and seismic conditions (blasting, if applicable) and temporal high *phreatic* and *piezometric surfaces*, and
    - the rational for selecting the strength conditions analyzed, including *drained shear strength*, *undrained shear strength*, *peak shear strength*, and *residual shear strength*.
- Plan views of the *internal*, *interim*, and *final slope* grading plans, clearly showing the locations of the analyzed cross sections, northings and eastings (e.g., see **Figure 8-12** on page 8-18 and **Figure 8-13** on page 8-19), and the limits of the *waste containment unit(s)*;

- Drawings of the analyzed cross sections, showing the slope components including:
    - soil material and waste boundaries,
    - temporal high *phreatic* and *piezometric surfaces*, if any,
    - soil, synthetic, and waste material types,
    - moist field densities and, where applicable, the *saturated* field densities,
    - material interface shear strengths (peak and residual, as applicable),
    - material internal shear strengths (*drained* and *undrained*, as applicable),
    - a depiction of each critical failure surface and its factor of safety, and
    - the engineered components of the facility.

Figure 8-1 A sliding mass of waste is capable of producing enormous force as is demonstrated in this picture of mining and earthmoving equipment that were crushed by a large waste failure at an Ohio landfill. Photo courtesy of CEC, Inc.

- Static stability calculations (both inputs and outputs) for *internal*, *interim*, and *final slopes* assuming *drained conditions* beneath the facility,

- As appropriate, static stability calculations for *internal*, *interim,* and *final slopes* assuming *undrained conditions* in the *soil units* beneath the facility. When a slope is underlain by a material that may develop excess pore water pressure during loading, the static factor of safety must be determined using the *undrained shear strength* of the foundation materials. The *undrained shear strengths* must be determined by shear strength testing of site-specific, undisturbed *samples* of all critical layers that may develop excess pore water pressure,

- Seismic stability calculations for *internal*, *interim*, and *final slopes* assuming *drained conditions*, or if applicable, *undrained conditions* beneath the facility,

- Any other calculations used for the analyses, and

The effective shear strength of a *soil unit* should be used when modeling conditions where excess pore water pressures have completely dissipated, or when the soil layers at the site will not become *saturated* during construction and filling of a facility.

The *unconsolidated-undrained shear strength* of a soil (as determined by shearing fully *saturated specimens* in a manner that does not allow for drainage from the *specimen* to occur) should be used whenever one or more fine-grained *soil units* exist at a site that are, or may become, *saturated* during construction and operations. This will produce a worst-case failure scenario, since it is unlikely that in the field any given *soil unit* will exhibit less shear strength than this.

- All figures, drawings, or references relied upon during the analysis, including at least a map of Ohio showing the peak acceleration (%g) with 2% probability of exceedance in 50 years that denotes the facility's location (e.g., see **Figure 8-9** on page 8-16).

FACTORS OF SAFETY

The following factors of safety should be used, unless superseded by rule, when demonstrating that a facility will resist deep-seated failures:

Static analysis: FS $\geq$ 1.50
Seismic analysis: FS $\geq$ 1.00

> The number of digits after the decimal point indicates that rounding can only occur to establish the last digit. For example, 1.579 can be rounded to 1.58, but not 1.6.

The use of higher factors of safety may be warranted whenever:

- A failure would have a catastrophic effect upon human health or the environment,

- Uncertainty exists regarding the accuracy, consistency, or validity of data, and no opportunity exists to conduct additional testing to improve or verify the quality of the data,

- Large uncertainty exists about the effects that changes to the site conditions over time may have on the stability of the facility, and no engineered controls can be implemented that will significantly reduce the uncertainty.

> Designers may want to consider increasing the required factor of safety if repairing a facility after a failure would create a hardship for the *responsible parties* or the waste disposal customers.

A facility must be designed to prevent deep-seated failures. Because of the uncertainties involved when calculating the factors of safety, and because any failure of the *waste containment facility* due to a deep-seated failure is likely to increase the potential for harm to human health and the environment, if a facility has a static factor of safety against deep-seated failure less than 1.5, elimination of the soil layers susceptible to a deep-seated failure, redesigning the facility to provide the required factor of safety, or using another site not at risk of a deep-seated failure will be necessary in most cases.

> The factors of safety specified in this policy are based on the assumptions contained in this policy. Those assumptions include, but are not limited to, the use of conservative, site-specific, *higher quality data*; proper selection of worst-case geometry; and the use of calculation methods that are demonstrated to be valid and appropriate for the facility. If different assumptions are used, these factors of safety may not be appropriately protective of human health and the environment.

However, if unusual circumstances exist at a facility, such as the critical failure surface occurs at interfaces with geosynthetics or internal to a GCL or RSL, and internal and interface *residual shear strengths* will be used for all construction materials and interfaces; or the geometry of a worst-case *internal slope* or *interim slope* is unique to one phase, and it will be constructed, buttressed and/or buried by sufficient waste or fill material during the same construction season so that it achieves the required factor of safety, then the *responsible party* may propose (this does not imply approval will be granted) to use a lower static factor of safety against deep-seated failures in the range of 1.5 to 1.25. The proposal should include any pertinent information necessary for demonstrating the appropriateness of the lower factor of safety to the facility.

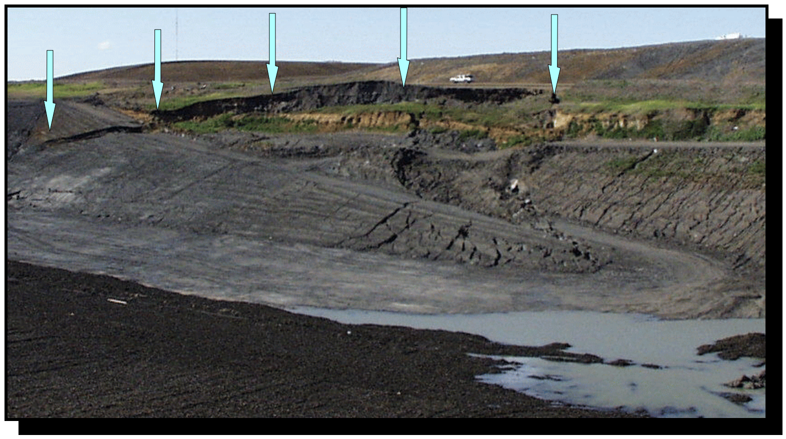


Figure 8-2 A complex rotational failure at a Texas landfill. White arrows identify the failure escarpment. For scale, note the pickup truck above the failure escarpment. Photograph courtesy of Dr. Timothy D. Stark, PE, University of Illinois, Urbana.

A design with a seismic factor of safety less than 1.00 against deep-seated failure indicates a failure may occur if the design earthquake occurs. Designing a *waste containment facility* in this manner is not considered a sound engineering practice. Furthermore, performing a deformation analysis to quantify the risks and the damage expected to a *waste containment facility* that includes geosynthetics is not considered justification for using a seismic factor of safety less than 1.00 for deep-seated failures. This is because geosynthetics are susceptible to damage at small deformations, and any failure to the *waste containment facility* due to a deep-seated failure is likely to increase the potential for harm to human health and the environment. If a facility has a seismic factor of safety against deep-seated failure less than 1.00, elimination of the soil layers susceptible to the deep-seated failure, redesigning the facility to provide the required seismic factor of safety, or using another site not at risk of a deep-seated failure will be necessary.

However, if unusual circumstances exist at a facility, such as an *internal slope* or *interim slope* represents a geometry that will not be present in additional phases during the life of the facility, the static factor of safety is greater than 1.5, and the slope will be constructed and buttressed or buried by sufficient waste or fill material during the same construction season so that it achieves the required factors of safety, then the *responsible party* may propose (this does not imply approval will be granted) to omit a seismic analysis of deep-seated failures for the slope. The proposal should include any pertinent information necessary for demonstrating the appropriateness of omitting the seismic analysis for the slope.

The *responsible party* should ensure that the design and specifications in all authorizing documents and the QA/QC plans clearly require that the assumptions and specifications used in the deep-seated failure analysis for the facility will be followed during construction, operations, and closure. If the *responsible party* does not do this, it is likely that Ohio EPA will require the assumptions and specifications from the deep-seated failure analysis to be used during construction, operations, and closure of a facility through such means as are appropriate (e.g., regulatory compliance requirements, approval conditions, orders, settlement agreements).

From time to time, changes to the facility design may be needed that will alter the assumptions and specifications used in the deep-seated failure analysis. If this occurs, a request to change the facility design is required to be submitted for Ohio EPA approval in accordance with applicable rules. The request to change the facility design must include a new deep-seated failure analysis that uses assumptions and specifications appropriate for the change.

ASSIGNING SHEAR STRENGTHS

When assigning shear strength values to materials and interfaces for modeling purposes, the following will usually apply:

- For foundation materials; values that are the lowest representative values for each *soil unit* should be used. These values will be available because the subsurface investigation should be completed before conducting stability analyses. Nonlinear shear strength envelopes that start at the origin should be used for each type of in situ material unless *unconsolidated-undrained shear strength* is being used for a *saturated* in situ soil layer (see Conformance Testing in Chapter 4 starting on page 4-15 for more information about nonlinear shear strength envelopes).

- For structural fill and recompacted soil components; soil materials may have been compacted in the laboratory using the lowest density and highest moisture content specified for construction and then tested for internal shear strength during the subsurface investigation (this is recommended). If this occurred, then values based on the field and laboratory testing conducted during the subsurface investigation will be available. Strength values for each engineered component made of structural fill or RSL should be modeled using the lowest representative values obtained from the testing of the weakest materials that will be used during construction. Nonlinear shear strength envelopes that start at the origin should be used for each material (see Conformance Testing in Chapter 4 starting on page 4-15 for more information about developing nonlinear shear strength envelopes).

  If testing of soils that will be used for structural fill and recompacted layers did not occur before the stability modeling because the source of the soils was not known, then the stability analysis can be used to determine the minimum shear strengths needed for these materials. As an alternative, conservative, assumed shear strengths for structural fill and RSL can be used. The assumed shear strengths should be low enough to ensure that the likelihood is very high that the strength exhibited by the structural fill and the recompacted materials during conformance testing prior to construction will always exceed the assumed values when constructed. However, the assumed shear strength values should not be so low that they cause the modeling software to relocate the worst-case failure surface inappropriately. The assumed values for internal *drained shear strength*s should be defined using shear strength envelopes that pass through the origin.

> Typically, cyclic loads will generate excess pore water pressures in loose *saturated* cohesionless materials (gravels, sands, non-plastic silts), which may liquefy with a considerable loss of pre-earthquake strength. However, cohesive soils and dry cohesionless materials are not generally affected by cyclic loads to the same extent. If the cohesive soil is not sensitive, in most cases, it appears that at least 80 percent of the static shear strength will be retained during and after the cyclic loading. (attributed to Makdisi and Seed in Abramson, et al, 1996, pp. 408).

- For interfaces with geosynthetics and for internal shear strengths of GCLs; it is recommended that the deep-seated failure analysis be used to determine the minimum interface shear strengths (and internal shear strengths of GCL) that are necessary to provide the required factors of safety. This will provide the maximum flexibility for choosing materials during construction. The resultant values determined by the stability modeling for peak and residual interface shear strengths should assume cohesion (c) is equal to zero. The actual internal and interface shear strengths of construction materials must be verified before construction (see Conformance Testing in Chapter 4 starting on page 4-15).

For deep-seated failure analysis of *internal*, *interim*, or *final slopes*, the following types of shear strengths should be specified in the authorizing documents and the QA/QC plan for the listed components:

- *Peak shear strengths* may be used for interfaces with a geosynthetic on slopes of 5 percent or less or slopes that will never be loaded with more than 1,440 psf. This allows the use of *peak shear strength*, if appropriate, for most *facility bottoms* during deep-seated failure analyses.

- *Residual shear strengths* are required for interfaces with a geosynthetic on slopes greater than 5 percent that will be loaded with more than 1,440 psf. This requires the use of *residual shear strengths* during deep-seated failure analysis for all interfaces that are on *internal slopes*.

MSW is difficult to test for shear strength. MSW has been shown to require so much displacement to mobilize its *peak shear strength*, and has a *peak shear strength* that is so much stronger than most other waste and soil materials, that using realistic shear strength values of the waste can cause *strain incompatibility* problems with computer modeling software. This could lead to the computer software overlooking the critical failure surface. In order to avoid this problem, the maximum allowable shear strength parameters to use when modeling MSW are: c = 500 psf and ϕ = 35°. It is appropriate to use lower shear strength values for MSW as long as they still force the failure surface into the liner system and foundation materials during modeling (adapted from Benson, 1998).

- Internal *peak shear strengths* may be used for reinforced GCL, if the internal shear strength of the GCL exceeds the *peak shear strength* of at least one of the interfaces with the GCL.

- Internal and interface *residual shear strengths* are required for unreinforced GCL, and

- *Drained* or *undrained shear strengths*, as appropriate, are required to be used for foundation and construction soil materials. When an *interim slope* or *final slope* is underlain by a material that may develop excess pore water pressure during loading, the static factor of safety must be determined using the *undrained shear strength* of the foundation materials. The *undrained shear strengths* must be determined by shear strength testing of site-specific, undisturbed *saturated specimens* of all materials that may develop excess pore water pressure. Using an *unconsolidated-undrained shear strength* for these types of soil layers allows for a worst-case analysis. This is because it is unlikely that soils in the field will exhibit less shear strength than the *unconsolidated-undrained shear strength* obtained from shearing fully *saturated specimens* while allowing no drainage from the *specimen*.

Residual shear strengths should be substituted for *peak shear strengths*, especially for interfaces, whenever reason exists to believe that the design, installation, or operation of a facility is likely to cause enough shear displacement within a material or interface that a post-peak shear strength will be mobilized (see **Figure f-2** on page xiv).

ACCOUNTING FOR THE EFFECTS OF WATER

Water is one of the most important factors to take into consideration when conducting a stability analysis. The presence or absence of water can have a dramatic effect upon the shear strength of soil materials, waste, and interfaces. It is essential that forces created by *phreatic* and *piezometric surfaces* are applied properly to an analysis.

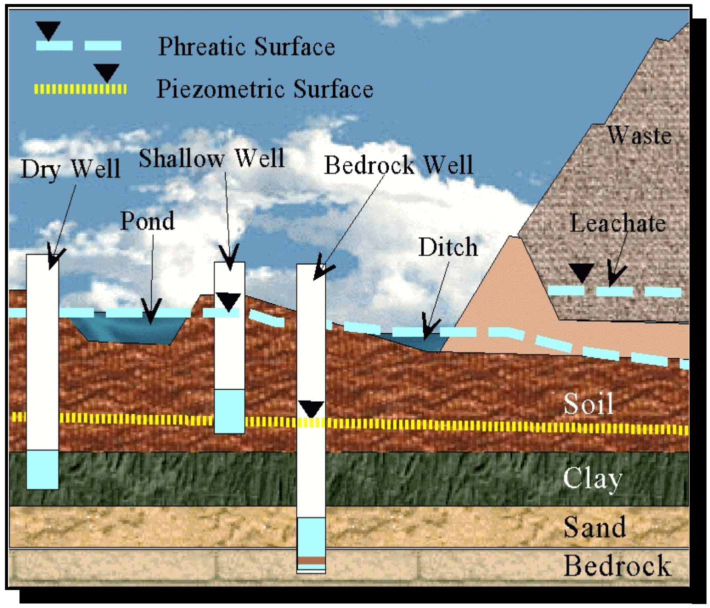


Figure 8-3 Examples of *phreatic* and *piezometric surfaces*.

Phreatic Surfaces

Phreatic surfaces (see **Figure 8-3**) that were identified during the subsurface investigation or that can be anticipated to occur must be included as part of all modeling. *Phreatic surfaces* include, but are not limited to:

- Leachate levels above liner systems caused by normal operations, leachate recirculation, or precipitation, among others,
- Surface water levels in ditches, streams, rivers, lakes, ponds, or lagoons that are part of the cross section that is being analyzed,
- The ground water tables associated with *soil units saturated* for only part of their thickness, and
- Anticipated levels of water to be found in engineered components such as berms.

Figure 8-4 Looking through the failed containment berm of a storm water retention basin that was located in Cuyahoga County. The outlet was plugged, causing the *phreatic surface* in the basin to become unexpectedly high. As a result, it overwhelmed the shear strength of the soil materials used to construct the berm and caused it to collapse.

Most modeling software will allow one or more *phreatic surfaces* to be modeled. It is important that the plausible worst-case *phreatic surfaces* (i.e., the highest temporal elevation of each *phreatic surface*) be modeled. For example, if a *waste containment facility* has an exterior berm that intrudes into a flood plain, an appropriate flood elevation (e.g., 100-year or 500-year flood elevation) should be used as the elevation of the *phreatic surface* in the berm. For this type of scenario, to model the worst-case, the *phreatic surface* should be drawn to show where it would be located immediately after the flood waters have subsided. This is the time that the *phreatic surface* will be at the highest elevation in the berm, but the berm will not have any confining pressure from the flood waters to help stabilize it, making it more vulnerable to failure (see **Figure 8-5**).

Other *phreatic surfaces* such as leachate on the liner, water levels in wastewater lagoons, and water tables in *soil units* should be modeled at the highest levels expected. Ohio EPA recommends conducting a sensitivity analysis on the worst-case *interim slope* and *final slope* by varying each *phreatic surface*, especially leachate head on a liner, water levels in lagoons and ponds, and any *phreatic surfaces* that occur within engineered components. By performing the sensitivity analysis, estimating the ability of the *waste containment facility* to resist failure will be possible if some unanticipated condition causes the *phreatic surfaces* to be increased above the maximum expected.

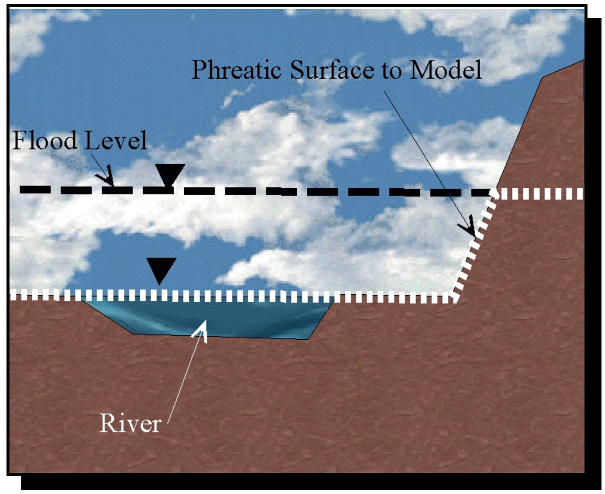


Figure 8-5 Example *phreatic surface* to model to account for pore water pressure created by flooding and then flood subsidence.

For example, modeling is often performed with one foot of leachate head on the liner of a solid waste facility because, by rule, that is the maximum amount of head allowed. However, if the pumps are not able to operate for a few days to a few weeks, the head could easily exceed the maximum and potentially threaten the stability of the facility. Another example would be modeling the normal water levels in a waste water lagoon. However, a heavy rain event may cause the water level in the lagoon to increase by several feet. The *phreatic surface*, in this case, should be modeled at the elevation of the water when it is discharging through the emergency spillway, in addition to an analysis when water is discharging at the elevation of the primary spillway.

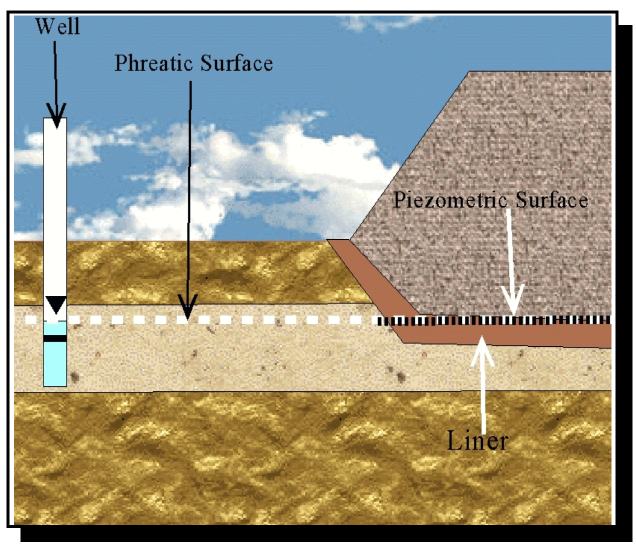


Figure 8-6 Example of a *piezometric surface* created by engineered components of a *waste containment facility*.

Piezometric Surfaces

Piezometric surfaces (see **Figure 8-3** on page 8-8) identified during the subsurface investigation or that can be anticipated to occur must be included as part of all modeling when the failure surfaces being analyzed pass through the unit associated with the *piezometric surface*. *Piezometric surfaces* include, but are not limited to:

! Surfaces that identify the pressure head found in a confined *saturated* layer,

! Surfaces that identify the pressure head found beneath an engineered component of a *waste containment facility* that acts as an aquaclude to an underlying *saturated soil unit* (see **Figure 8-6** on page 8-9).

Piezometric surfaces should only be used when examining stability in relation to the single material or interface subjected to head pressure created by the water confined within the unit. For example, in **Figure 8-3** on page 8-8, the sand layer below the clay unit should be associated with the piezometric surface (the short-dashed line) in the modeling software. The clay unit would have no *phreatic* or *piezometric surface* associated with it because wells screened exclusively in the clay unit were dry. The *soil unit* should be associated with the *phreatic surface* (the long-dashed line). The *piezometric surface* of the sand unit would be ignored for all units except the sand because the piezometric head has its effect only on failure surfaces that pass through the sand.

ANALYSIS

Three types of slopes will be the focus of this section: *internal slopes* (e.g., the interior side slope liner of a landfill or lagoon), *interim slopes* (e.g., a temporary slope), and *final slopes* (e.g., the cap system of a landfill, or exterior berm of a lagoon). See **Figure f-1** on page xii for a graphical representation of each of these types of slopes. Most *internal slopes* and *interim slopes* need to remain stable until they are buttressed with waste or fill. Some *internal slopes* (e.g., at a waste water impoundment) and all *final slopes* need to remain stable indefinitely.

> Numerous case histories of failures demonstrate that *interim slopes* are often more critical than *final slopes*. This is because they often have inherently less stable geometry and are often left in-place due to construction delays or changes in waste placement. Inadvertent over-filling, toe excavation, and over-steepening have also triggered failures of *interim* and *internal slopes*.

Static Analysis

After the *drained shear strengths* and *undrained shear strengths* for soil materials have been assigned, the *peak shear strengths* and *residual shear strengths* for interfaces have been assigned, and it has been determined how to model the *phreatic surfaces* and *piezometric surfaces* for the facility, the deep-seated failure analysis for *internal slopes*, *interim slopes*, and *final slopes* should be performed using the conservative assumption that the entire mass of the facility was placed all at once. If the facility design does not meet the required 1.50 factor of safety for *drained conditions*, the facility should be redesigned. If a facility has fine-grained soil units, and they are saturated or may become saturated for any reason during the life of the facility, then a stability analysis should use the undrained shear strength of these soil units. If using the undrained shear strength in the analysis is appropriate, and the facility design does not meet the required 1.50 factor of safety for *undrained conditions* when assuming the mass of the facility was placed all at once, then an analysis of staged loading may be performed, or the facility can be redesigned.

A staged loading analysis will determine how much of the mass of the facility can be constructed at one time and still provide the required factor of safety. When conducting a staged loading analysis, CU triaxial compression test data with pore water measurements representing future loading are used in combination with UU triaxial test data representing the conditions before receiving the first loading. These data are used to determine the maximum load that can be added without exceeding the *undrained shear strength* of the underlying materials. Settlement calculations are then used to determine the time it will take to dissipate excess pore water pressure. The information is used to maintain stability during filling by developing a plan for the maximum rate of loading.

The *responsible party* should ensure that the design and specifications in all authorizing documents and the QA/QC plans clearly require that the assumptions and specifications used in a staged loading analysis for the facility will be followed during construction, operations, and closure. If the *responsible party* does not do this, it is likely that Ohio EPA will require the assumptions and specifications from the staged loading analysis to be used during construction, operations, and closure of a facility through such means as are appropriate (e.g., regulatory compliance requirements, approval conditions, orders, settlement agreements).

Figure 8-7 Expansion crack (marked by white arrows) that developed at the top of a slope of an Ohio landfill that had experienced a deep-seated translational failure involving RSL and unreinforced GCL. Contrast this with the damage at the toe of the same slope shown in **Figure 8-8**.

When calculating the static factor of safety for *internal*, *interim*, and *final slopes*, multiple cross sections of the facility should be analyzed. Cross sections should be selected based on the angle and height of the slopes; the relationship of the length and slope of the *facility bottom* to the adjoining *internal slope*; the grade, extent, and shear strength of underlying materials; and the internal and interface shear strengths of structural fill and other engineered components. The location of toe excavations, temporal high *phreatic* and *piezometric surfaces*, and construction timing should also be taken into account when selecting the cross sections. The intent of the static analysis is to find all cross sections with factors of safety less than what is required anytime during construction, operations, closure, or the post-closure period of the facility.

Figure 8-8 Damage to FML of an Ohio landfill at the bottom of a slope from a deep-seated translational failure involving RSL and unreinforced GCL. Contrast this with the tension crack near the top of the same slope shown **Figure 8-7**.

Most commonly, each cross section is entered into a computer program that calculates the factor of safety using two-dimensional limit equilibrium methods. These cross sections should be entered so that the computer program is allowed to generate failure surfaces through the foundation of the facility well beyond the toe and well beyond the peaks of slopes. The cross sections should be analyzed for translational and rotational failures. When analyzing cross sections containing geosynthetics for translational failures, the search for the failure surface should focus on the layer(s) representing the geosynthetics. This is because layers that include geosynthetics tend to be the most prone to translational failures (see **Figure 8-14** on page 8-20). If the slope or foundation materials contain relatively thin *critical layers*, they should also be examined for translational failures.

Circular failure surfaces having relatively short radii should be analyzed for the lower portions of each slope (see **Figure 8-15** on page 8-20). This part of the analysis is performed to ensure that potential failures at the toe are not overlooked. A failure at the toe could result in a complete regressive failure of the *waste containment facility*.

When using programs that allow a variable number of randomly generated failure surfaces, a sufficient number of failure surfaces should be used to assure that the worst-case failure surface has been located. This may require from 1,000 to 5,000 or more searches depending on the size of the search boxes, search areas, and the length of the cross sections. Once an area within a cross section has been identified as the probable location of the failure surface, subsequent searches should be conducted to refine the location of the failure surface and ensure that the surface with the lowest factor of safety has been found.

Seismic Analysis

> The worst-case failure surface found during the static analysis is used for pseudostatic modeling because the search engines of most modeling software are not designed for use when a seismic load has been applied. Therefore, a new search for a critical failure surface should not be conducted in a pseudostatic analysis.

When calculating the seismic factor of safety for *internal*, *interim*, and *final slopes*, the worst-case static translational failure surface and the worst-case static rotational failure surface associated with each selected cross section should be analyzed for stability using the appropriate horizontal ground acceleration to represent a seismic force.

> Ohio EPA is unlikely to allow a deformation analysis at facilities with geosynthetics because even small deformations can cause geosynthetics to be damaged to a degree that they cannot perform their design functions.

If the facility design does not meet the required 1.00 seismic factor of safety, the facility should be redesigned or different materials should be specified to obtain the required factor of safety.

However, if unusual circumstances exist at a facility, such as no geosynthetics are included in the design, the ratio of site-specific yield acceleration (k_y) to site-specific horizontal ground acceleration (n_g) at the base of the sliding mass is greater than 0.60, and the cross section has a static factor of safety of at least 1.25 against deep-seated failures using the post-peak strength of the materials measured at the largest displacement expected from deformation caused by the design seismic event, then the *responsible party* may propose (this does not imply approval will be granted) to use deformation analysis when the seismic factor of safety for a cross section is lower than 1.00. The proposal should include any pertinent information necessary for demonstrating the appropriateness of allowing the lower factor of safety and relying upon deformation analysis to verify the stability of the facility.

Example Method - Brief Procedure for the Newmark Permanent Deformation Analysis

1. "Calculate the yield acceleration, k_y. The yield acceleration is usually calculated in pseudo-static analyses using a trial and error procedure in which the seismic coefficient is varied until a factor of safety = 1.0 is obtained." (U.S. EPA, 1995).

2. Divide the yield acceleration by the peak horizontal ground acceleration (n_g) expected at the facility, adjusted to account for amplification and/or dampening effects of the waste and soil fill materials.

3. If the resulting ration is greater than 0.60, then no deformation would be expected.

Selecting a Horizontal Ground Acceleration for Seismic Analysis

Selecting an appropriate horizontal ground acceleration to use during seismic analysis is highly facility-specific. The location of the facility, the types of soils under the facility, if any, and the type, density, and height of the engineered components and the waste, all affect the horizontal ground acceleration experienced at a facility from any given seismic event. The base of facilities founded on *bedrock* or medium soft to stiff *soil units* will likely experience the same horizontal ground acceleration as the *bedrock*. Facilities founded on soft *soil units* or deep cohesionless *soil units* will need a more detailed analysis and possibly field testing to determine the effects the soils will have on the horizontal ground acceleration as it reaches the base of the facility.

The seismic hazard maps produced by USGS show predicted peak ground accelerations at the ground surface, not at the top of *bedrock*. USGS creates the maps based on the assumption that the top 30 m of material below the ground surface has a shear wave velocity of 760 m/sec. If a facility design calls for the excavation or addition of a significant amount of material, or if the foundation materials under the facility have a significantly different shear wave velocity, then the designer may want to calculate a site-specific horizontal ground acceleration to prevent using a seismic coefficient for the facility that is excessively conservative or excessively unconservative. At the time of writing this policy, USGS was proposing creation of peak *bedrock* acceleration maps. If they become available, they could be used as a basis for deriving a site-specific seismic coefficient. See the USGS earthquake Web site at http://eqhazmaps.usgs.gov/ for more information.

Waste and structural fill can cause the horizontal ground acceleration experienced at the base of a facility to be transmitted unchanged, dampened, or amplified by the time it reaches the surface of the facility. The expected effects of the waste and structural fill on the horizontal ground acceleration will need to be determined for each facility to estimate the proper horizontal ground acceleration to use for stability modeling purposes. MSW is typically a relatively low density, somewhat elastic material. It is expected that a horizontal ground acceleration with a shear wave velocity of 700 feet/sec (fps)[b] at the base of a MSW facility having 200 feet or more waste may dampen as it reaches the surface of the facility (see **Figure 8-10** on page 8-17). It is also expected that the same horizontal ground acceleration at the base of a MSW facility having 100 feet of waste or less will be amplified as it travels to the surface of the facility (see **Figure 8-11** on page 8-17).

The amplification caused by any depth of municipal waste is not expected to exceed the upper bound of amplification observed for motions in earth dams as attributed to Harder, 1991, in Singh and Sun, 1995 (see **Figure 8-11** on page 8-17). To determine the effects of industrial wastes, such as flue gas desulfurization dust, cement kiln dust, lime kiln dust, foundry sands, slags, and dewatered sludges on the horizontal ground acceleration, the characteristics of the waste will need to be determined. This is done by either measuring actual shear wave velocity through the materials or applying a method for estimating the effect of the waste on the horizontal ground acceleration, such as demonstrating the similarity of the waste to compacted earth dam material, very stiff natural soil deposits, or deep cohesionless soil deposits and applying the above noted figures.

Selecting a value for the horizontal ground acceleration to use during seismic analysis is also dependant upon the methodology being used and the conservatism deemed appropriate for the design. If the

[b] Singh and Sun, 1995, report that shear wave velocities recorded at MSW landfills have generally ranged from 400 fps to 1000 fps, and sometimes higher. 700 fps is the average shear wave velocity used by Singh and Sun.

methodology for seismic analysis applies the horizontal force at the center of gravity of the sliding mass (e.g., a pseudostatic stability analysis), then an average of the horizontal ground acceleration experienced by the facility at its base and at its surface is currently thought to be appropriate. This means that for facilities where amplification of the horizontal ground acceleration is expected as it approaches the surface of the facility, an acceleration greater than the horizontal ground acceleration will be used. Also, for facilities expected to dampen the horizontal ground acceleration as it approaches the surface of the facility, an acceleration less than the horizontal ground acceleration may be used. However, to be conservative, designers may want to consider using the actual horizontal ground acceleration for facilities expected to dampen accelerations.

> Designers may chose to use other methods for deriving the seismic coefficient that are more accurate than using the arithmetic mean of the horizontal ground acceleration expected at the top and bottom of the facility. For example, a mass average value of the horizontal ground acceleration may be used, or the WESHAKE program can be used to propagate the predicted horizontal ground acceleration through the structural fill and waste.

If the methodology for seismic analysis applies the horizontal force at the failure surface (e.g., an infinite slope analysis), then the horizontal ground acceleration expected at the failure surface should be used rather than the average mentioned in the previous paragraph.

Alternative methods for determining site-specific adjustments to expected horizontal ground accelerations may also be used. These typically involve conducting seismic testing to determine site-specific shear wave velocities and amplification/dampening characteristics. A software package such as WESHAKE produced by the U.S. Army Corps of Engineers (USACOE), Engineer Research and Development Center, Vicksburg, MS, is then used to calculate the accelerations at different points in the facility. Because of the differences between earthquakes that occur in the western and the eastern United States, using earthquake characteristics from Ohio and the eastern United States is necessary when using software, such as WESHAKE, to estimate induced shear stress and accelerations.

> Seismic events may be naturally occurring or manmade. Examples of events that may create significant seismic force at a *waste containment facility* include earthquakes, landslides on adjacent areas, avalanches, explosions (intended or unintended) such as blasting, and low frequency vibrations created by long trains.

Ohio EPA requires that the seismic coefficient (n_g), used in numerous stability modeling software packages, be based on the value of the peak ground acceleration from a final version of the most recent USGS “National Seismic Hazard Map” (e.g., see **Figure 8-9** on page 8-16) showing the peak acceleration (%g) with 2% probability of exceedance in 50 years. As of the writing of this policy, the seismic hazard maps are available www.usgs.gov on the USGS Web site. Once the facility location on the map has been determined, then the peak horizontal ground acceleration indicated on the map must be adjusted to account for amplification effects and may be adjusted to account for dampening effects of the soils, engineered components, and waste at the facility, as discussed above. If instrumented historical records indicate that a facility has experienced horizontal ground accelerations that are higher than those shown on the USGS map, then the higher accelerations should be used as the basis for determining the seismic coefficient for the facility.

> FEMA document 369 contains additional information for using the USGS seismic hazard maps for estimating site-specific horizontal ground accelerations, as well as additional information about designing earthquake resistant buildings and non-building structures.

Deep Failure - Example Calculation

A 100-acre landfill is proposed to be located in south-central Ohio. The existing contours slope gently to the south. The northern portion of the landfill will be excavated approximately 40 feet. A 40-foot berm will be constructed to the south of the unit (see **Figure 8-12** on page 8-18). SPTs performed at a frequency of one per four acres, found that the facility is underlain by approximately 65 feet of very stiff silts and clays with some intermittent sand seams, transitioning down into about 10 feet of wet, stiff clay, over 5 feet of *saturated* sand that is lying on top of the sandstone *bedrock*. Multiple *samples* of each layer were analyzed. The lowest representative internal *drained shear strength*s of each *soil unit* and construction material were used to create nonlinear *drained shear strength* envelopes specific to each *soil unit* and construction material. The lower clay unit had a lowest representative *undrained shear strength* of 0^{o} and a cohesion of 2,000 psf. The facility has 3(h):1(v) *internal slopes, interim slopes*, and *final slopes*. The liner system comprises 5 feet of RSL, a 60-mil textured FML, a geotextile cushion layer, a 1-foot granular drainage layer, and a geotextile filter layer.

The deep-seated analysis was used to challenge the in situ foundation materials under the waste mass to ensure that they provide a static factor of safety of 1.50 and a seismic factor of safety of 1.00 for circular failures. The deep-seated analysis was also used to determine the minimum shear strength necessary to provide a static factor of safety of 1.50 and a seismic factor of safety of 1.00 against translational failure surfaces propagating through the liner/leachate collection system.

This example examines multiple *internal*, *interim*, and *final slopes* to find the factor of safety for the worst-case deep-seated rotational and translational failure surfaces assuming *drained conditions* and, where appropriate, *undrained conditions*. Next, it examines the worst-case rotational and translational failure surfaces with *drained conditions* for each *interim slope* and *final slope* during seismic conditions.

See **Figure 8-12** on page 8-18 and **Figure 8-13** on page 8-19 for plan views of the facility. See **Figure 8-14** and **Figure 8-15** on page 8-20 for examples of the cross sections. A summary of the shear strengths and the results of the stability analysis are found in Table 6 starting on page 8-21. The input data and results of a seismic analysis of one cross section are found at the end of this chapter starting on page 8-25.

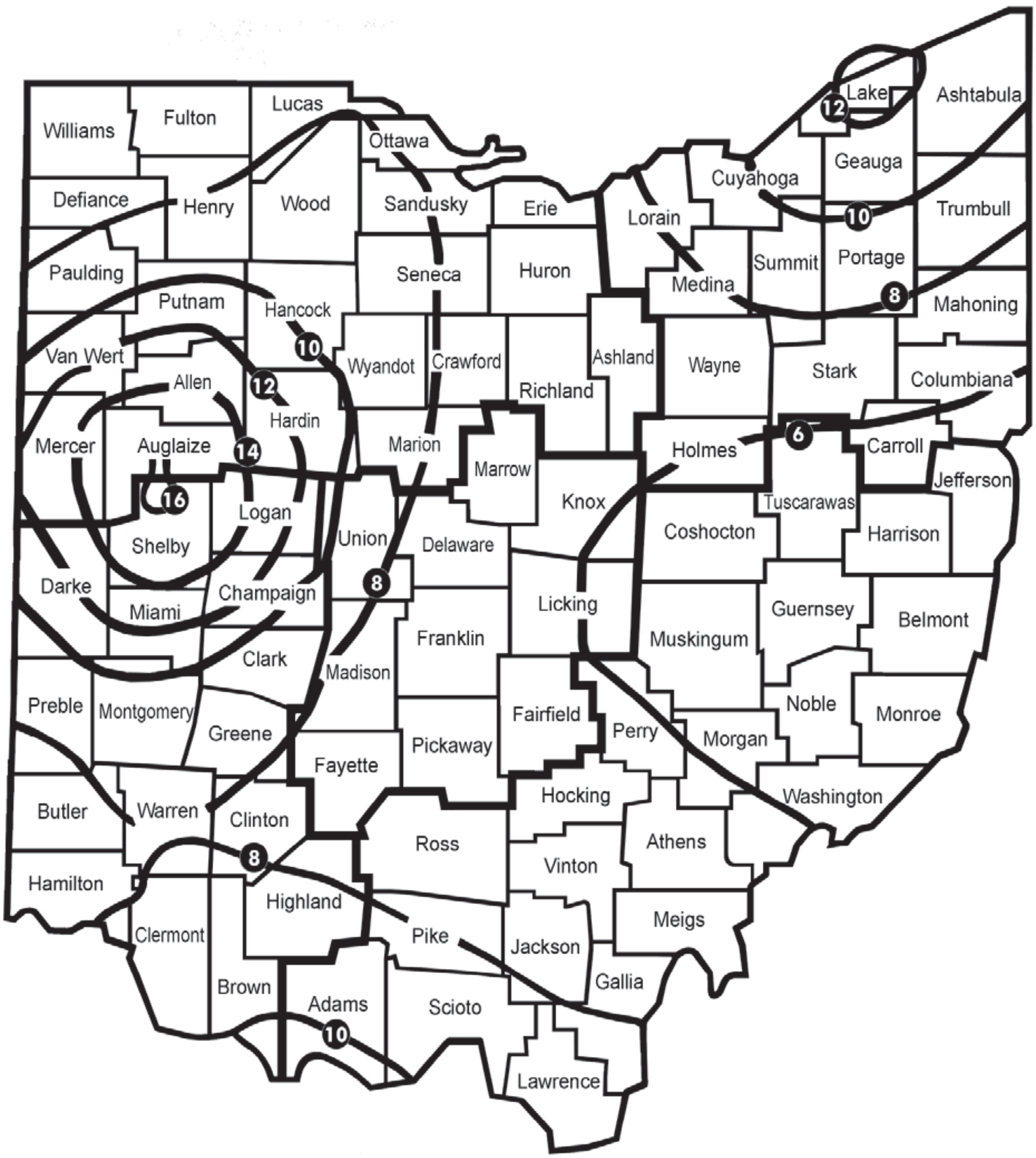


Figure 8-9 The peak horizontal ground acceleration (%g) with 2% probability of exceedance in 50 years. U.S. Geological Survey, October 2002, National Seismic Hazard Mapping Project, “Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary).”

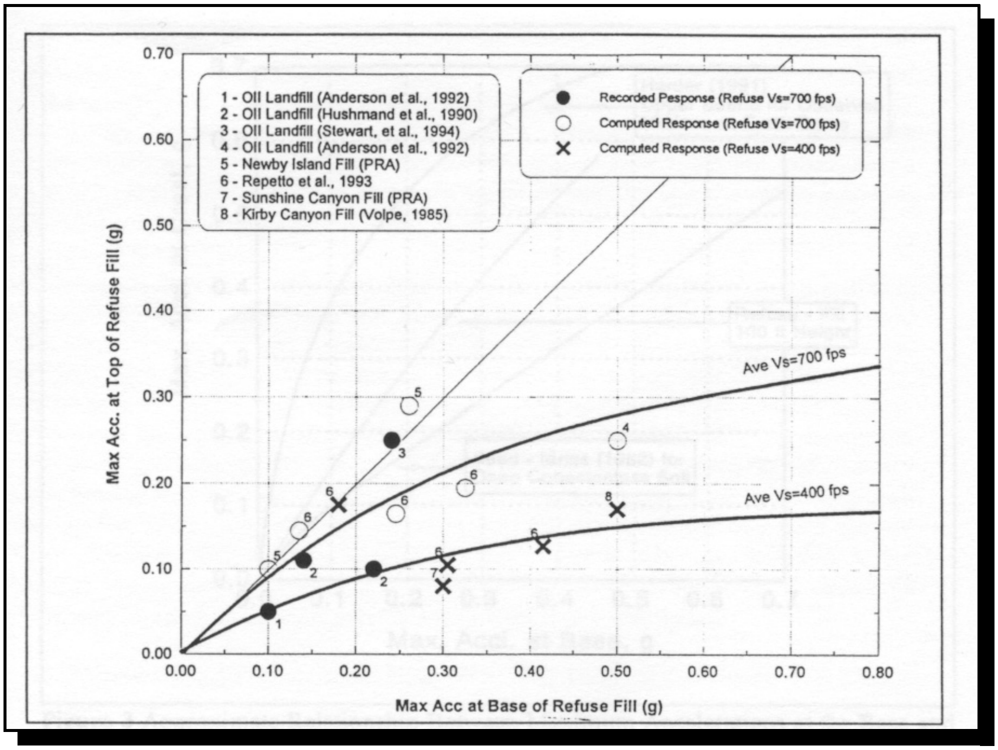


Figure 8-10 Approximate relationship between maximum accelerations at the base and crest of 200 feet of refuse. Singh and Sun, 1995, Figure 1.

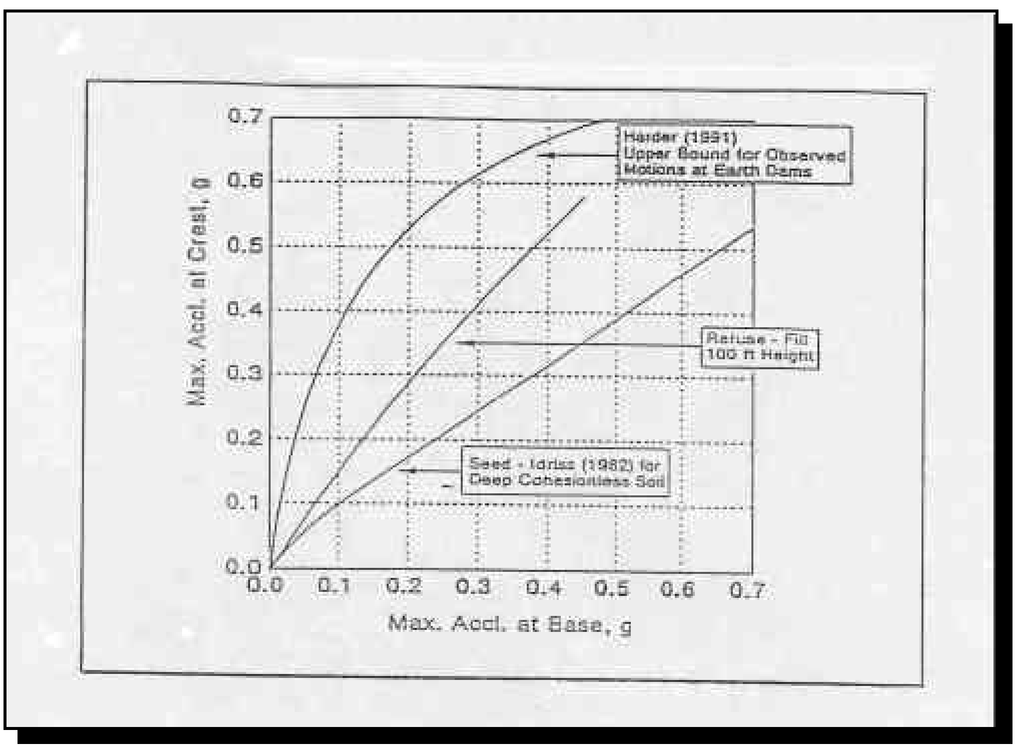


Figure 8-11 Approximate relationship between maximum accelerations at the base and crest for various ground conditions. Singh and Sun, 1995, Figure 3.

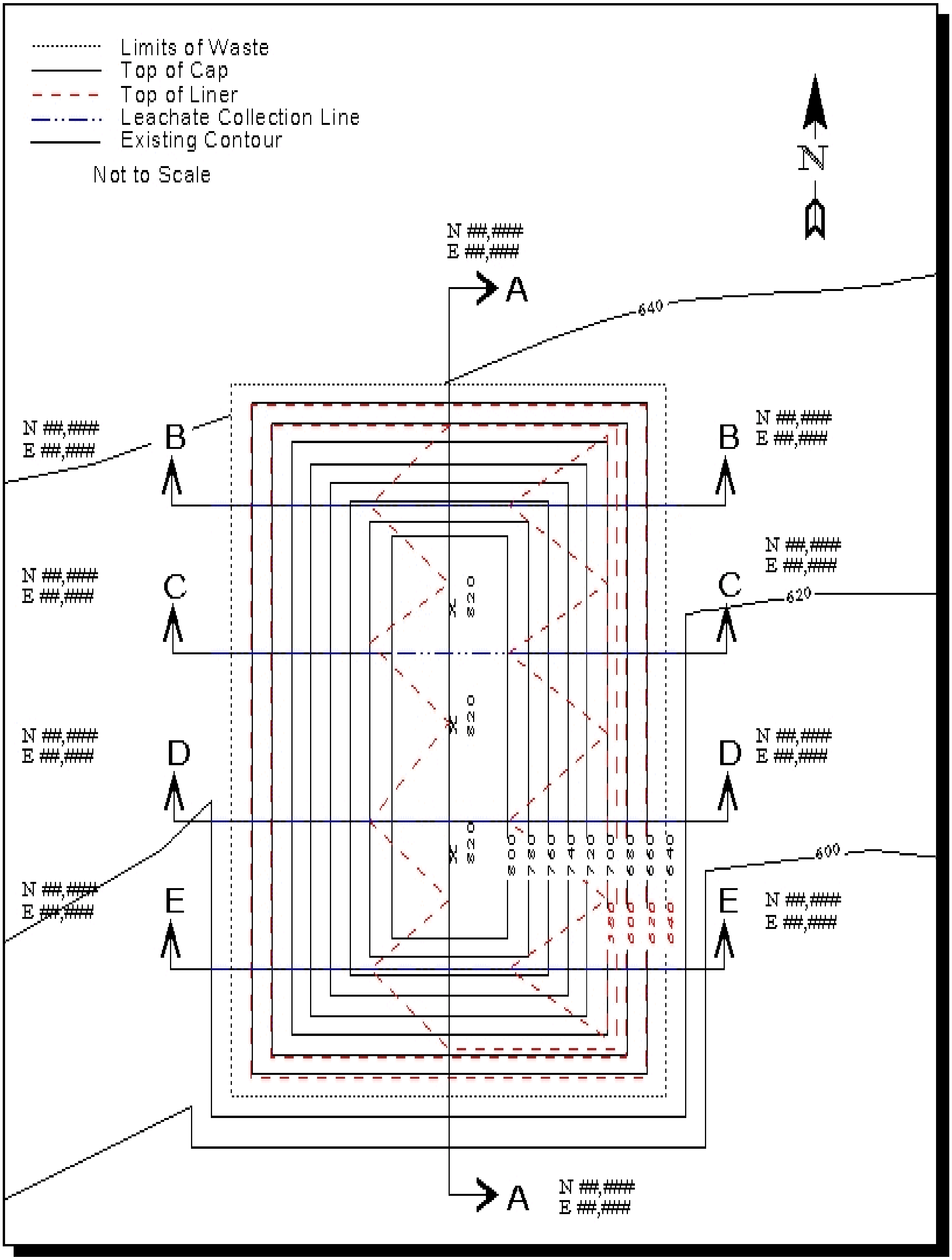


Figure 8-12 Example plan view showing top and bottom elevations and the location of cross sections that were analyzed for stability.

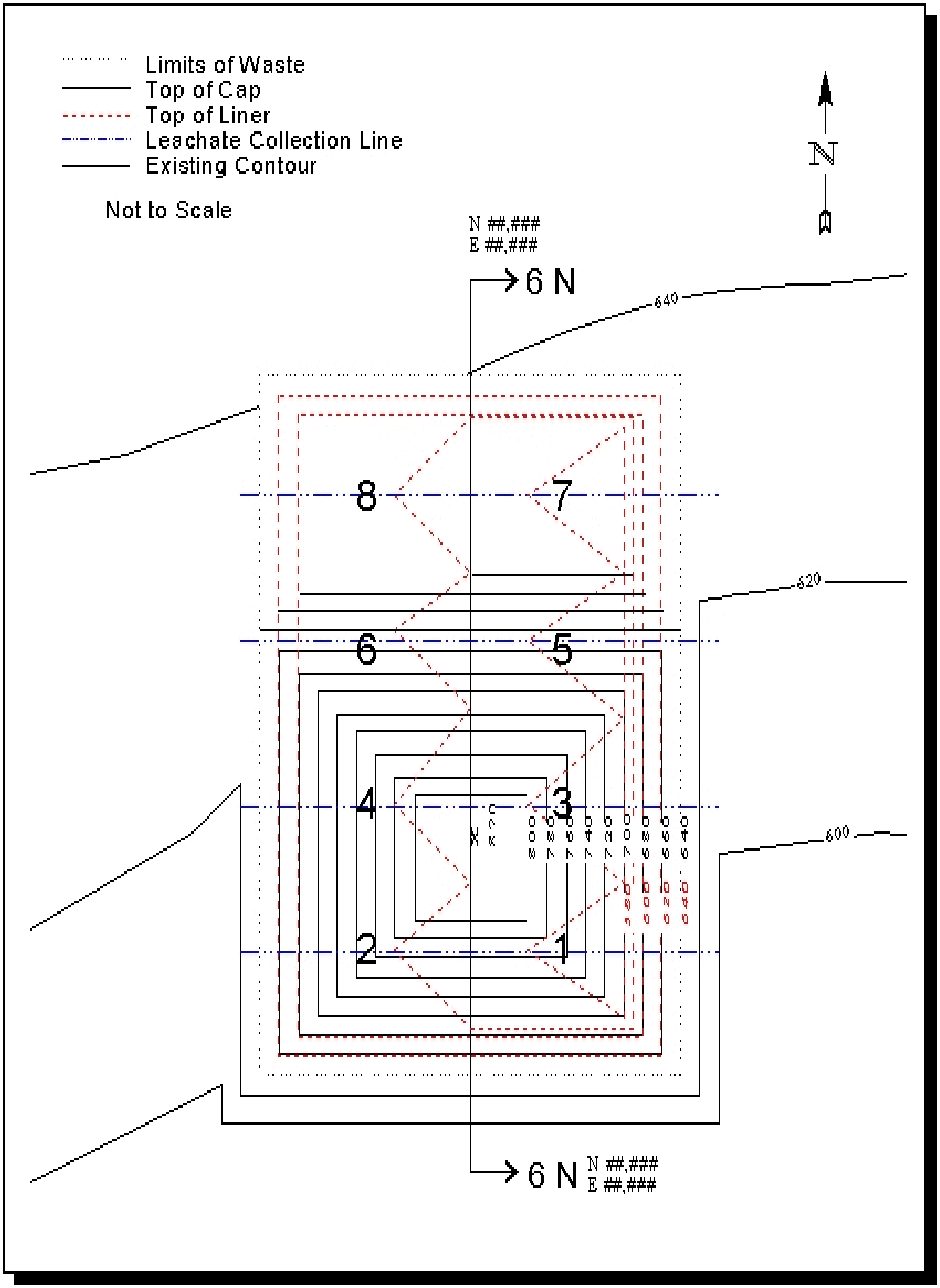


Figure 8-13 Example plan view showing the location of one of the *interim slope* cross sections that were analyzed for stability.

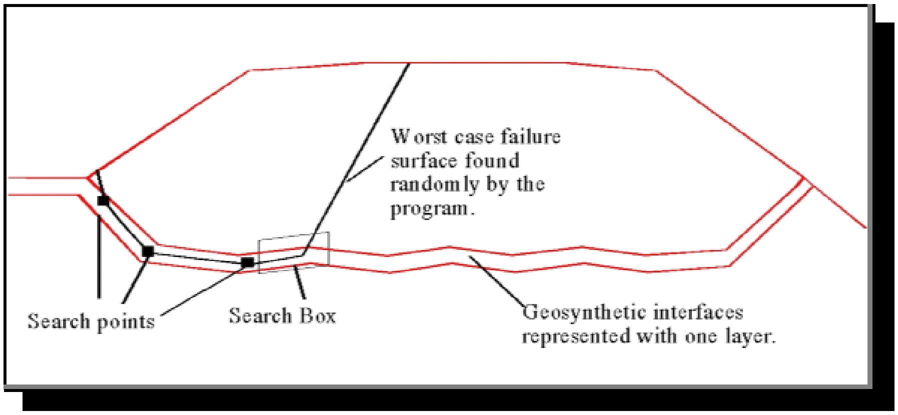


Figure 8-14 Cross Section A-A'. Example translational failure surface found by directing modeling software to a specific interface.

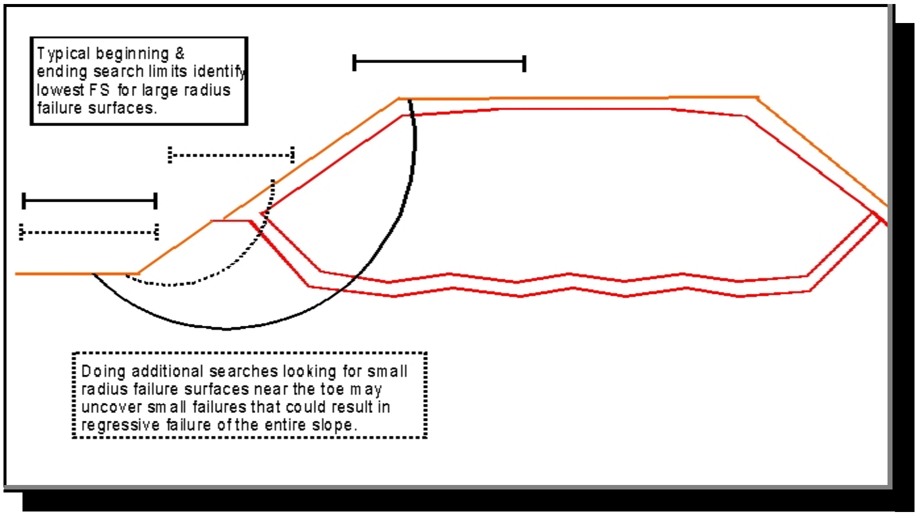


Figure 8-15 Example of using different search limits to look for different size failure surfaces.

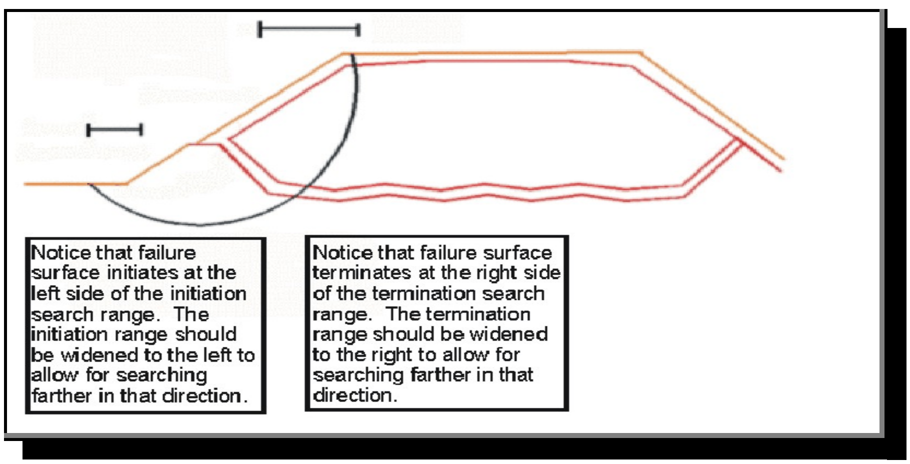


Figure 8-16 Example of search limits inappropriately restricting the search engine in both directions. Even if the search limits inappropriately restrict searching in only one direction, the search range should be adjusted and the analysis run again.

Table 6. An example summary table of internal and interface shear strengths and stability analysis results.

Deep-Seated Failure Analysis				
Inputs		C (psf)	ϕ	Moist field density, (, (psf)
1	Waste	480[A]	33°	70
2[B]	Drainage layer sand	0	35°	
3[B]	This shear strength applies to all geosynthetic interfaces placed on *internal slopes* or the *facility bottom* with a grade of 5% or greater. The *residual shear strength* of all such interfaces would be required to exceed these values during *conformance testing*. If soil unit #3 had been omitted from the model, the shear strength envelope for soil unit #5 would also apply to the geosynthetic interfaces on *internal slopes* or the *facility bottom* with a grade of 5% or greater. The interface *peak shear strength* of the geosynthetic interfaces would be required to exceed the soil unit #5 values during *conformance testing*. For modeling purposes a nonlinear shear strength envelope was adjusted until the minimum factor of safety of 1.50 was obtained. However, a linear envelope with c = 0 could have been used instead.	Shear Strength Envelope		62.4
		Normal Stress	Shear Stress	
		(psf)	(psf)	
		0	0	
		288	200	
		720	300	
		1440	550	
		7200	1500	
		12960	1900	
		35000	1900[C]	
4[B]	This shear strength applies to all geosynthetic interfaces placed on the *facility bottom* with a grade of 5% or less. The *peak shear strength* of all such interfaces would be required to exceed these values during *conformance testing*. If *soil unit* #4 had been omitted from the model, the shear strength envelope for soil unit #5 would also apply to the geosynthetic interfaces on the *facility bottom* with a grade of 5% or less. The interface *peak shear strength* of the geosynthetic interfaces would be required to exceed the soil unit #5 values during *conformance testing*. For modeling purposes a nonlinear shear strength envelope was adjusted until the minimum factor of safety of 1.50 was obtained. However, a linear envelope with c = 0 could have been used instead.	Shear Strength Envelope		62.4
		Normal Stress	Shear Stress	
		(psf)	(psf)	
		0	0	
		288	210	
		720	320	
		1440	560	
		7200	1580	
		12960	2330	
		35000	2330[C]	
5[B]	The nonlinear shear strength envelope used for the RSL was chosen in order to ensure that it was low enough that the internal *peak shear strength* of the RSL during *conformance testing* would exceed these values without making it so low that the modeling software incorrectly placed the worst-case failure surface. A linear envelope with c = 0 and an assumed ϕ could have been used instead for modeling purposes. If that was done, then the internal *peak shear strength* of the RSL from *conformance testing* would need to exceed the assumed linear shear strength value used.	Shear Strength Envelope		110
		Normal Stress	Shear Stress	
		(psf)	(psf)	
		0	0	
		288	110	
		720	276	
		1440	552	
		7200	2763	
		12960	4974	
		35000	4974[C]	

[A] If MSW is modeled with c = 0 psf, it is likely that negative stress errors will be eliminated during modeling. This is especially appropriate when analyzing translational failures surfaces.

[B] For modeling purposes, Units # 2, #3, #4, and #5, which represent the composite liner/leachate collection system, could have been modeled as one unit equal to the thickness of the liner/leachate collection system. A nonlinear or linear shear strength envelope could have been used and adjusted in the modeling software until the required factor of safety was obtained. The resulting shear strength envelope would then become the required minimum for all components of the liner/leachate collection system for the types of shear strength applicable to the materials on each type of slope.

[C] It was assumed that available testing apparatuses would not be able to test at a normal stress of 35,000 psf. Therefore, this shear stress was conservatively estimated by using the same shear stress as the highest normal load expected to be tested.

Table 6. An example summary table of internal and interface shear strengths and stability analysis results. (Cont.)

Deep-Seated Failure Analysis				
Inputs		C (psf)	ϕ	Dry density γ (psf)
6	The nonlinear shear strength envelope used for the structural fill was chosen in order to ensure that it was low enough that the internal *peak shear strength* of the structural fill during *conformance testing* would exceed these values, without making it so low that the modeling software incorrectly placed the worst-case failure surface. A linear envelope with c = 0 and an assumed ϕ could have been used instead for modeling purposes. If that was done, then the internal *peak shear strength* of the structural fill from *conformance testing* would need to exceed the assumed value used here.	Normal Stress	Shear Stress	110
		(psf)	(psf)	
		0	0	
		1440	752	
		7200	2963	
		12960	5174	
		35000	5174[A]	
7	Upper clay/silt[C]	Shear Strength Envelope		110
		Normal Stress	Shear Stress	
		(psf)	(psf)	
		0	0	
		1440	781	
		7200	3108	
		12960	5436	
		35000	5436[A]	
8	Lower clay undrained condition[B]	2000	0°	100
9	Lower clay *drained condition*[C]	Shear Strength Envelope		100
		Normal Stress	Shear Stress	
		(psf)	(psf)	
		0	0	
		1440	674	
		7200	2770	
		12960	4867	
		35000	4867[A]	
10	Lower sand	0	35°	130
11	Sandstone *bedrock*	15000	0°	140

[A] It was assumed that available testing apparatuses would not be able to test at a normal stress of 35,000 psf. Therefore, this shear stress was conservatively estimated by using the same shear stress as the highest normal load expected to be tested.

[B] This is the lowest representative *undrained shear strength* measured during testing of this in situ foundation material.

[C] The normal stresses chosen for soil units #7 and #9 are from multiple laboratory tests conducted during the subsurface investigation. The shear stresses represent the lowest shear stresses measured for each foundation material during testing.

Table 6. An example summary table of internal and interface shear strengths and stability analysis results. (Cont.)

Deep-Seated Failure Analysis						
STATIC						
Cross Section	Translational		Rotational			
	Along geosynthetic interface	Through lower clay *undrained condition*	*Drained conditions*		*Undrained conditions*	
			Large radius	Short radius	Large radius	Short radius
Internal AA North	NA	NA	1.72 / 1.68[2]	2.00 / 1.98[2]	Not Analyzed[3]	Not Analyzed[3]
Internal AA South	NA	1.80	1.76 / 1.76[2]	2.04 / 2.02[2]	Not Analyzed[3]	Not Analyzed[3]
AA North	1.84 / 2.03[1]	1.89	2.62 / 2.58[2]	3.03 / 3.03[2]	2.27	Not Analyzed[3]
AA South	1.84 / 2.03[1]	1.54	2.28 / 2.27[2]	1.87 / 1.87[2]	1.90	Not Analyzed[3]
BB East	2.11 / 2.37[1]	5.3	2.65 / 2.65[2]	2.80 / 2.78[2]	2.70	Not Analyzed[3]
BB West	1.93 / 2.12[1]	Not Analyzed[4]	2.74 / 2.72[2]	3.02 /3.00[2]	2.54	Not Analyzed[3]
CC East	1.86 / 2.03[1]	Not Analyzed[4]	2.24 / 2.53[2]	1.82 / 1.82[2]	2.28	Not Analyzed[3]
CC West	1.80 / 1.96	Not Analyzed[4]	2.48 / 2.45[2]	2.06 / 2.06[2]	2.25	Not Analyzed[3]
DD East	1.93 / 2.08[1]	Not Analyzed[4]	2.385 / 2.37[2]	2.00 / 1.98[2]	2.21	Not Analyzed[3]
DD West	1.79 / 1.96[1]	Not Analyzed[4]	2.44 / 2.41[2]	2.02 / 2.03[2]	2.23	Not Analyzed[3]
EE East	2.13 / 2.26[1]	4.5	2.30 / 2.28[2]	1.96 / 1.96[2]	2.14	Not Analyzed[3]
EE West	1.91 / 2.09[1]	Not Analyzed[4]	2.48 / 2.46[2]	2.07 / 2.07[2]	2.14	Not Analyzed[3]
Interim End of Phase 1	1.71 / 1.75[1]	1.78	2.21 / 2.18[2]	2.27 / 2.25[2]	1.83	2.15
Interim End of Phase 2	1.68 / 1.73[1]	1.62	2.26 / 2.23[2]	2.57 / 2.56[2]	1.94	Not Analyzed[3]
Interim End of Phase 4	2.04 / 2.22[1]	1.63	2.14 / 2.11[2]	2.51 / 2.50[2]	1.85	Not Analyzed[3]
Interim End of Phase 5	1.71 / 1.81[1]	1.94	2.18 / 2.16[2]	2.48 / 2.48[2]	2.10	Not Analyzed[3]
Interim End of Phase 6	1.52 / 1.50[1]	1.50	2.09 / 2.06[2]	2.40 / 2.38[2]	1.84	2.30

1 Factor of safety calculated with Simplified Janbu method/Spencer's method.

2 Factor of safety calculated with Simplified Bishop method/Spencer's method.

3 The worst-case failure surface found by XSTABL remained within the berm and did not extend through the undrained layer.

4 This cross section has a similar geometry and the same shear strengths as the BB East and EE East cross sections that have very high factors of safety. It is reasonable to assume that this cross section will also have a similarly high factor of safety. Therefore, analysis of this cross section was not needed.

Table 6. An example summary table of internal and interface shear strengths and stability analysis results (Cont.).

Deep-Seated Failure Analysis				
SEISMIC				
Cross Section	Seismic coefficient (n_g)	Translational	Rotational *Drained conditions*	
		Along geosynthetic interface	Large radius	Short radius
Internal AA North	0.10	NA	1.3	1.42
Internal AA South	0.10	NA	1.32	1.43
AA North	0.10[1]	1.37	1.88	2.46
AA South	0.10[1]	1.37	1.64	1.38
BB East	0.10[1]	1.62	1.93	2.03
BB West	0.10[1]	1.44	2	2.22
CC East	0.10[1]	1.38	1.87	1.39
CC West	0.10[1]	1.32	1.78	1.44
DD East	0.10[1]	1.43	1.69	1.41
DD West	0.10[1]	1.33	1.74	1.43
EE East	0.10[1]	1.53	1.65	1.45
EE West	0.10[1]	1.41	1.78	1.57
Interim End of Phase 1	0.125[2]	1.19	1.54	1.46
Interim End of Phase 2	0.125[2]	1.18	1.61	1.7
Interim End of Phase 4	0.10[1]	1.58	1.51	1.8
Interim End of Phase 5	0.10[1]	1.58	1.55	1.78
Interim End of Phase 6	0.10[1]	1.04	1.49	1.73

1 The seismic coefficient (n_g) was calculated using the average of the values for the top and bottom of facility obtained from **Figure 8-9** on page 8-16 and adjusted using **Figure 8-10** on page 8-17 [(0.10 + 0.09) / 2 = 0.095, use 0.10].

2 The seismic coefficient (n_g) was calculated using the average of the values for the top of the phase and bottom of the facility obtained from **Figure 8-9** on page 8-16 and adjusted using **Figure 8-11** on page 8-17 [(0.10 + 0.15) / 2 = 0.125, use 0.125]. The maximum height of waste of phases 1 and 2 is less than 200 feet and more than 100 feet at the point in time when filling operations move into adjacent phases.

Example Computer Modeling Output

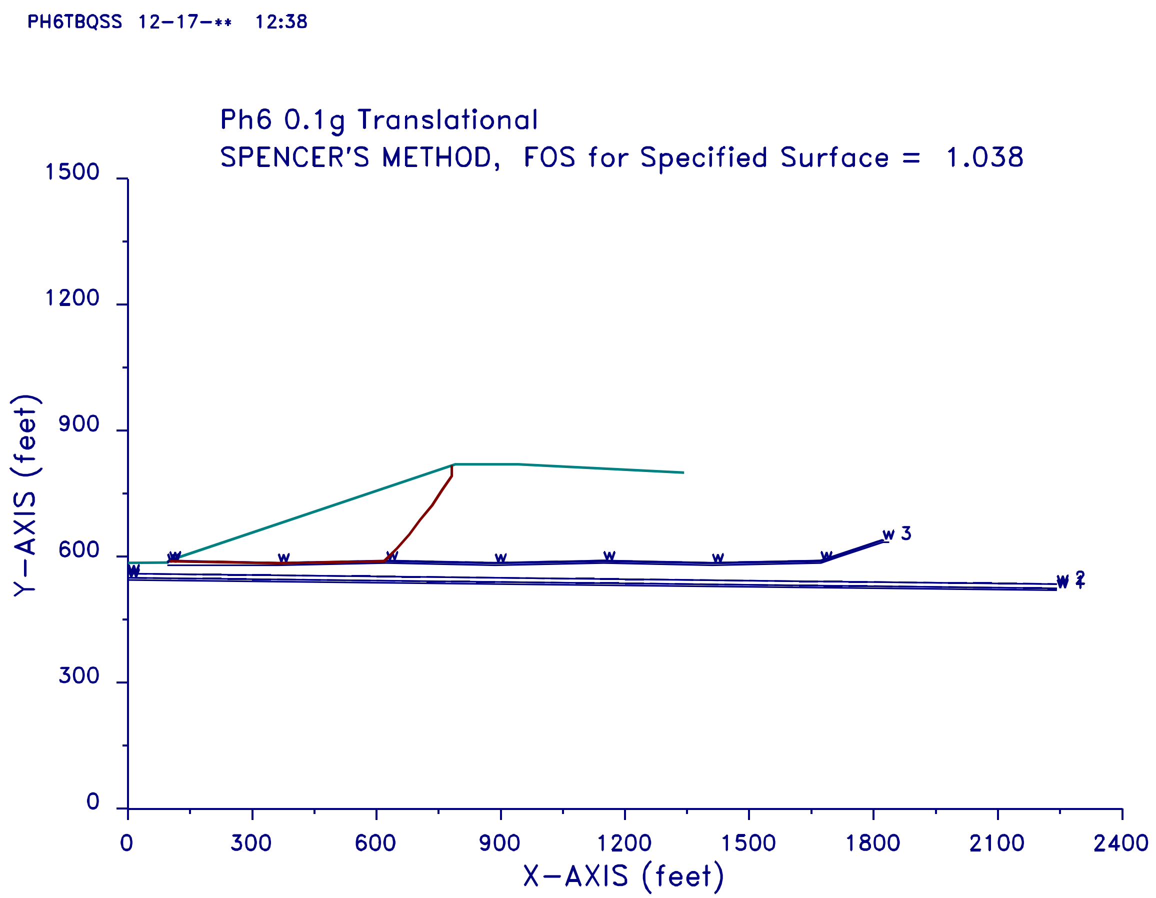
XSTABL File: PH6TBQSS12-17-02 12:44

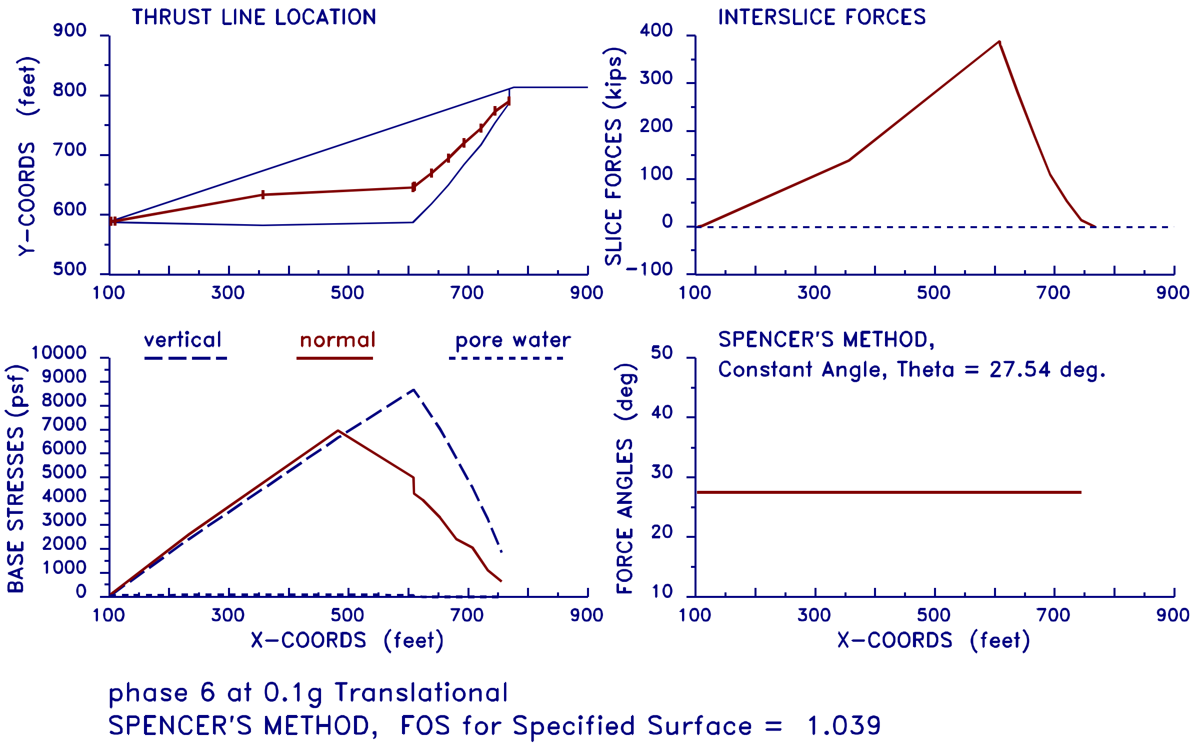
X S T A B L

Slope Stability Analysis
using the
Method of Slices

Ver.5.202 96) 1697

Problem Description : Ph6 0.1g Translational





According to XSTABL Reference Manual, copyrighted 1995, Interactive Software Designs, Inc., the four graphs presented in this figure are:

(A) Thrust Line Location (upper left) – shows the location of the thrust line computed using Spencer's method or the GLE method. The location of the assumed line is shown for the Janbu GPS procedure. For a reasonable solution, the thrust line should be located within the failure slide mass.

(B) Stress plots (lower left) – these show the variation of the total vertical and normal stress along the failure surface. The lines shown connect the calculated average value of the vertical and normal stress at the center of the slice base. If pore water pressure exists along the failure surface, it is also plotted on this graph. For a reliable solution, the calculated normal stresses should be very near or below the reported vertical stresses.

(C) Interslice Forces (upper right) – this plot shows the variation of the calculated interslice forces within the slide mass. For a reasonable solution, the distribution should be relatively smooth and indicate only compressive forces (i.e., positive) throughout the failure surface. Sometimes, tensile forces reported very close to the crest of a failure surface may be tolerated, or alternatively, a cracked zone should be implemented into the slope geometry. The insertion of such a cracked zone will often relieve the tensile forces and improve the location of the thrust line. For such cases, the user should also seriously consider the inclusion of a hydrostatic force that may be attributed to a water-filled crack.

(D) Interslice Force Inclination (lower right) – this plot shows the computed values of the interslice force angles and the overall distribution of their range, as assumed by the GLE methods. For the Janbu GPS procedure, this plot gives the values of the interslice force angles calculated from the assumed location of the thrust line. For a reasonable solution, the magnitude of the interslice force angle should typically be less than the angle of internal friction of the soils within the failure mass. For cases where different soils are present within a typical slice, an average ϕ-value will be selected to check for compliance with this condition.

SEGMENT BOUNDARY COORDINATES
----

5 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	.0	585.0	95.0	586.0	6
2	95.0	586.0	100.0	591.5	2
3	100.0	591.5	790.0	820.0	1
4	790.0	820.0	942.0	820.0	1
5	942.0	820.0	1342.0	800.0	1

When modeling a waste containment facility's global stability, it is not always necessary to model the entire cross section in detail. For example, final cap layers do not need to be included when looking for deep-seated translational and circular failures through foundation materials, liner/leachate collection systems can be modeled as one layer, and for cross sections that are much wider than is the depth to *bedrock* only the portion of the cross section being evaluated needs to be included in the cross section that is modeled.

37 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	100.0	591.5	102.0	591.0	2
2	102.0	591.0	362.0	586.0	2
3	362.0	586.0	624.0	591.0	2
4	624.0	591.0	886.0	586.0	2
5	886.0	586.0	1148.0	591.0	2
6	1148.0	591.0	1410.0	586.0	2
7	1410.0	586.0	1672.0	591.0	2
8	1672.0	591.0	1822.0	641.0	2
9	95.0	585.0	100.0	590.0	4
10	100.0	590.0	362.0	585.0	4
11	362.0	585.0	624.0	590.0	4
12	624.0	590.0	886.0	585.0	4
13	886.0	585.0	1148.0	590.0	4
14	1148.0	590.0	1410.0	585.0	4
15	1410.0	585.0	1672.0	590.0	4
16	1672.0	590.0	1822.0	640.0	3
17	95.0	584.0	100.0	589.0	5
18	100.0	589.0	362.0	584.0	5
19	362.0	584.0	624.0	589.0	5
20	624.0	589.0	886.0	584.0	5
21	886.0	584.0	1148.0	589.0	5
22	1148.0	589.0	1410.0	584.0	5
23	1410.0	584.0	1672.0	589.0	5
24	1672.0	589.0	1822.0	639.0	5
25	1822.0	639.0	1825.0	639.0	5

The geosynthetic interfaces (highlighted) have been modeled one-foot thick so it is easier to force the failure surfaces through the geosynthetic. To simplify modeling further, the entire composite liner/leachate collection system could have been modeled as one layer four (4) to six (6) ft thick, depending on the design of the facility. The shear strength necessary to provide the required factor of safety would then apply to all interfaces and materials in the composite liner/leachate collection system.

26	95.0	580.0	362.0	580.0	7
27	362.0	580.0	624.0	585.0	7
28	624.0	585.0	886.0	580.0	7
29	886.0	580.0	1148.0	585.0	7
30	1148.0	585.0	1410.0	580.0	7
31	1410.0	580.0	1672.0	585.0	7
32	1672.0	585.0	1717.0	600.0	7
33	1717.0	600.0	1822.0	635.0	6
34	1822.0	635.0	1837.0	635.0	6
35	.0	560.0	2242.0	535.0	8
36	.0	550.0	2242.0	525.0	10
37	.0	545.0	2242.0	520.0	11

A CRACKED ZONE HAS BEEN SPECIFIED

Depth of crack below ground surface	=	24.00 (feet)
Maximum depth of water in crack	=	0.00 (feet)
Unit weight of water in crack	=	62.40 (pcf)

Failure surfaces will have a vertical side equal to the specified depth of crack and be affected by a hydrostatic force according to the specified depth of water in the crack.

After the first Spencer's analysis was completed, a cracked zone was added to relieve negative (tensile) interslice forces and to improve the location of the thrust line. A crack depth of 24 feet was the shallowest depth that was found that improved the analysis results. However, it should be noted that the addition of this crack did not affect the final factor of safety, but only proved to better predict the failure surface.

ISOTROPIC Soil Parameters
----

11 *Soil unit*(s) specified

Soil Unit No.	Soil Type	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pressure Constant (psf)	Water Surface No.
1	Waste	70.0	75.0	480.0[A]	33.00	.000	.0	3
2	Drainage layer sand	130.0	135.0	.0	35.00	.000	.0	3
3	All geosynthetic interfaces <5% slope at *residual shear strength*	62.4	62.4	.0	.00	.000	.0	3
4	All geosynthetic interfaces >5% slope at *peak shear strength*	62.4	62.4	.0	.00	.000	.0	3
5	RSL	110.0	120.0	.0	.00	.000	.0	0

6	Structural fill	110.0	120.0	.0	.00	.000	.0	0
7	Upper clay/silt	110.0	120.0	.0	.00	.000	.0	0
8	Lower clay *unconsolidated-undrained conditions*	110.0	120.0	2000.0	.00	.000	.0	2
9	lower clay *drained conditions*	110.0	120.0	.0	.00	.000	.0	0
10	lower sand	135.0	135.0	.0	35.00	.000	.0	1
11	rock	100.0	100.0	15000.0	.00	.000	.0	0

[A] If MSW is modeled with c = 0 psf, it is likely that negative stress errors will be eliminated during modeling. This is especially appropriate when analyzing translational failures surfaces.

UNDRAINED STRENGTHS as a function of effective vertical stress
have been specified for 1 Soil Unit(s)

Soil Unit #	Parameter a	Parameter Psi
8.	2000.0	.00

This is the lowest representative *undrained shear strength* measured during testing of this in situ foundation material.

NON-LINEAR MOHR-COULOMB envelope has been specified for 6 soil(s)

The normal stresses chosen for *soil units* #3 through #6 bracket the normal stresses expected at the facility. They are for materials that will be tested in the laboratory before construction of the waste containment facility. The shear stresses used here represent the shear strengths that created the minimum acceptable factor of safety. When construction materials are tested before construction of the *waste containment facility*, it is expected that the shear stresses associated with the normal stress of 35,000 psf will not be able to be tested with the available testing apparatus. Therefore, this shear stress was conservatively estimated by using the same shear stress as the highest normal load that can be tested.

Soil Unit # 3

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	288.0	200.0
3	720.0	300.0
4	1440.0	550.0
5	7200.0	1500.0
6	12960.0	1900.0
7	35000.0	1900.0

Soil Unit # 4

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	288.0	210.0
3	720.0	320.0
4	1440.0	560.0
5	7200.0	1580.0
6	12960.0	2330.0
7	35000.0	2330.0

Soil Unit # 5

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	288.0	110.0
3	720.0	276.0
4	1440.0	552.0
5	7200.0	2763.0
6	12960.0	4974.0
7	35000.0	4974.0

Soil Unit # 6

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	1440.0	752.0
3	7200.0	2963.0
4	12960.0	5174.0
5	35000.0	5174.0

Soil Unit # 7

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	1440.0	781.0
3	7200.0	3108.0
4	12960.0	5436.0
5	35000.0	5436.0

The normal stresses chosen for *soil units* #7 and #9 are those that bracket the expected normal stresses at the facility. They were tested in the laboratory during the subsurface investigation. The shear stresses are the lowest representative stresses measured for each in situ foundation material that will be under the *waste containment facility*, except the shear stresses associated with the normal stress of 35,000 psf, which could not be tested with the available testing apparatus. Therefore, this shear stress was conservatively estimated by using the same shear stress as the highest normal load tested.

Soil Unit # 9

Point No.	Normal Stress (psf)	Shear Stress (psf)
1	.0	.0
2	1440.0	674.0
3	7200.0	2770.0
4	12960.0	4867.0
5	35000.0	4867.0

3 Water surface(s) have been specified

Unit weight of water = 62.40 (pcf)

Water Surface No. 1 specified by 2 coordinate points

****

PHREATIC SURFACE,

****

Point No.	x-water (ft)	y-water (ft)
1	.00	550.00
2	2242.00	525.00

Water Surface No. 2 specified by 2 coordinate points

****

PIEZOMETRIC SURFACE,

****

Point No.	x-water (ft)	y-water (ft)
1	.00	560.00
2	2242.00	535.00

Water Surface No. 3 specified by 9 coordinate points

****

PHREATIC SURFACE,

****

Point No.	x-water (ft)	y-water (ft)
1	95.00	586.00
2	100.00	591.00
3	362.00	586.00
4	624.00	591.00
5	886.00	586.00
6	1148.00	591.00
7	1410.00	586.00
8	1672.00	591.00
9	1822.00	641.00

A horizontal earthquake loading coefficient of 0.100 has been assigned

A vertical earthquake loading coefficient of 0.000 has been assigned

Some computer programs only support *phreatic* or *piezometric surfaces* and some recommend not using random searching techniques when incorporating piezometric surface. Please refer to your user manual for instructions for modeling water surfaces.

A *phreatic surface* has been placed at the top of the sand since *borings* showed that the water table was located there.

A *piezometric surface* has been placed at the top of the lower clay since the *borings* indicated that this clay was wet and had the potential of exhibiting *undrained shear strength* if loaded rapidly, due to the creation of excess pore water pressure.

A *phreatic surface* has been placed one-foot above the bottom of the layer representing the interfaces with the geosynthetics to represent the leachate head on the liner.

The seismic coefficient was calculated by averaging the peak horizontal ground acceleration expected at the base of the facility with the peak horizontal ground acceleration expected at the surface of the facility. These numbers were obtained from the USGS National Seismic Hazard Map and adjusted based on the characteristics of the *waste containment facility*. See Table 6 on page 8-24 for more details.

A SINGLE FAILURE SURFACE
HAS BEEN SPECIFIED FOR
ANALYSIS

Trial failure surface specified by
the following 12 coordinate points :

Point No.	x-surf (ft)	y-surf (ft)
1	100.00	591.50
2	105.00	589.38
3	362.00	584.36
4	618.50	589.04
5	649.15	620.61
6	678.48	653.40
7	705.19	688.37
8	733.89	721.72
9	757.54	758.82
10	781.77	793.27
11	781.77	793.27
12	781.77	817.27

This cross section was first modeled with a critical failure surface search method using a random technique for generating sliding block surfaces. The active and passive portions of the sliding surfaces were generated according to the Simplified Janbu method. This was done by running 1000 random trial surfaces with the passive and active portions of the failure surface being generated at fixed angles using the Rankine method (passive 45+ ϕ/2, active = 45-ϕ/2), defined using the following boxes:

Box no.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Width (ft)
1	105.0	589.4	105.0	589.4	0.4
2	362.0	584.5	362.0	584.5	0.5
3	362.1	584.5	624.0	589.5	1.0

This resulted in a failure surface that terminated about fifty feet away from the crest of the slope. This distance from the crest indicated that a more critical failure surface may exist, so the analysis was re-run using the same boxes and the Simplified Janbu method, but a different technique (called block in XSTABL) that generates “irregularly oriented segments” for the passive and active portions of the block surface. This technique tends to require more random trial surfaces, so 5000 were used. This resulted in a failure surface that appears to conservatively represent the worst-case failure surface for this cross section.

After the first Spencer’s analysis was run on the worst-case failure surface, the following was preformed to improve the graphical outputs provided by XSTABL:

1. A cracked zone was added to relieve negative (tensile) interslice forces and to improve the location of the thrust line. Then, a new worst-case failure surface was found. The depth of 24 feet was the shallowest depth that improved the analysis results.

2. The first coordinate point was moved to the toe of the slope to improve the location of the thrust line.

However, it should be noted that the addition of this crack and moving the initiation point changed the final factor of safety by 0.004 and took a lot of time. Adding the crack to relieve negative (tensile) interslice forces is considered optional, unless the thrust line is excessively erratic or misplaced.

****
SELECTED METHOD OF ANALYSIS: Spencer (1973)
****

****
SUMMARY OF INDIVIDUAL SLICE INFORMATION
****

Slice	x-base (ft)	y-base (ft)	height (ft)	width (ft)	alpha	beta	weight (lb)
1	100.62	591.24	.47	1.23	-22.98	18.32	48.
2	101.62	590.81	1.22	.77	-22.98	18.32	79.
3	102.85	590.29	2.15	1.70	-22.98	18.32	333.
4	104.35	589.65	3.29	1.30	-22.98	18.32	383.
5	233.50	586.87	48.84	257.00	-1.12	18.32	894486.
6	490.25	586.70	134.04	256.50	1.05	18.32	2421814.
7	618.92	589.48	173.87	.85	45.85	18.32	10347.
8	619.84	590.42	173.23	.99	45.85	18.32	12027.
9	634.74	605.77	162.81	28.82	45.85	18.32	328406.
10	663.82	637.01	141.21	29.33	48.19	18.32	289913.
11	691.83	670.88	116.61	26.71	52.63	18.32	218020.
12	719.54	705.05	91.62	28.70	49.29	18.32	184068.
13	745.71	740.27	65.06	23.65	57.48	18.32	107714.
14	769.66	776.05	37.22	24.23	54.88	18.32	63125.

Nonlinear —C Iteration Number - 1

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	25.4680	1.0407	1.0209
3	24.7137	-----	1.0407
3	25.0908	1.0395	-----
4	24.7640	1.0386	1.0395
5	24.7837	1.0386	1.0386

Nonlinear —C Iteration Number - 2

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	24.8846	1.0380	1.0378

Nonlinear —C Iteration Number - 3

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
2	24.8725	1.0380	1.0380

ITERATIONS FOR SPENCER'S METHOD

Iter #	Theta	FOS_force	FOS_moment
1	24.8725	1.0380	1.0380

SLICE INFORMATION ... continued :

Slice	Sigma (psf)	c-value (psf)	phi	U-base (lb)	U-top (lb)	P-top (lb)	Delta
1	182.1	.0	35.00	0.	0.	0.	.00
2	442.7	.0	35.00	8.	0.	0.	.00
3	751.1	.0	35.00	76.	0.	0.	.00
4	572.4	136.7	14.29	111.	0.	0.	.00
5	3702.0	305.0	10.04	25370.	0.	0.	.00
6	9626.2	642.5	7.42	27965.	0.	0.	.00
7	7221.8	642.5	7.42	108.	0.	0.	.00
8	6210.3	.0	35.00	44.	0.	0.	.00
9	5790.0	480.0	33.00	0.	0.	0.	.00
10	4732.9	480.0	33.00	0.	0.	0.	.00
11	3461.1	480.0	33.00	0.	0.	0.	.00
12	2932.3	480.0	33.00	0.	0.	0.	.00
13	1584.5	480.0	33.00	0.	0.	0.	.00
14	903.8	480.0	33.00	0.	0.	0.	.00

SPENCER'S (1973) - TOTAL Stresses at center of slice base

Slice #	Base x-coord (ft)	Normal Stress (psf)	Vertical Stress (psf)	Pore Water Pressure (psf)	Shear Stress (psf)
1	100.62	182.1	39.1	.0	122.8
2	101.62	452.4	103.2	9.7	298.7
3	102.85	792.0	195.6	40.9	506.7
4	104.35	651.1	295.3	78.7	272.1
5	233.50	3800.7	3480.5	98.7	925.4
6	490.25	9735.2	9441.8	109.0	1826.5
7	618.92	7310.8	12232.7	89.0	1524.9
8	619.84	6241.5	12158.6	31.2	4189.4
9	634.74	5790.0	11397.0	.0	4085.0
10	663.82	4732.9	9884.5	.0	3423.6
11	691.83	3461.1	8162.5	.0	2627.9
12	719.54	2932.3	6413.5	.0	2297.0
13	745.71	1584.5	4554.5	.0	1453.8
14	769.66	903.8	2605.2	.0	1027.9

SPENCER'S (1973) - Magnitude & Location of Interslice Forces

Slice #	Right x-coord (ft)	Force Angle (degrees)	Interslice Force (lb)	Force Height (ft)	Boundary Height (ft)	Height Ratio
1	101.23	24.87	267.	.54	.93	.583
2	102.00	24.87	672.	.68	1.51	.453
3	103.70	24.87	2218.	1.18	2.80	.421
4	105.00	24.87	2959.	1.87	3.78	.495
5	362.00	24.87	187541.	50.47	93.90	.537
6	618.50	24.87	386789.	63.32	174.17	.364
7	619.35	24.87	380050.	63.70	173.58	.367
8	620.34	24.87	376282.	63.47	172.88	.367
9	649.15	24.87	280410.	55.55	152.75	.364
10	678.48	24.87	188075.	46.93	129.67	.362
11	705.19	24.87	108002.	37.79	103.54	.365
12	733.89	24.87	52589.	29.35	79.70	.368
13	757.54	24.87	13819.	20.95	50.43	.416
14	781.77	.00	-6.	-.26	24.00	-.011

AVERAGE VALUES ALONG FAILURE SURFACE

Total Normal Stress	=	5614.52 (psf)
Pore Water Pressure	=	68.72 (psf)
Shear Stress	=	1750.68 (psf)

Total Length of failure surface = 781.13 feet

For the single specified surface and the assumed angle of the interslice forces, the SPENCER'S (1973) procedure gives a

FACTOR OF SAFETY = 1.038

Total shear strength available
along specified failure surface =141.12E+04 lb

This factor of safety is greater than 1.00, which is the minimum necessary to demonstrate seismic stability.

REFERENCES

Abramson, L. W., Lee, T. S., Sharma, S., and Boyce, G. M., 1996, Slope Stability and Stabilization Methods. John Wiley and Sons, Inc. New York.

Benson, C. H., 1998, "Properties of Municipal Solid Waste," Static and Seismic Slope Stability for Waste Containment Facilities, April 27 - 29, University of Wisconsin-Madison.

Gilbert, R. B., 2001, "Peak versus Residual Strength for Waste Containment Systems," Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.

Interactive Software Designs, Inc., 1998, XSTABL: An Integrated Slope stability Analysis Program for Personal Computers, Reference Manual, Version 5, Moscow, Idaho.

Koerner, R. M., 1997, "Designing with Geosynthetics," 4th Edition, Prentice Hall Publ. Co., New York.

Singh, S., and Sun, J. I., 1995, "Seismic Evaluation of Municipal Solid Waste Landfills," Geoenvrionment 2000, New Orleans, pp. 1081 - 1096.

Stark, T. D. and Poeppel, A. R., 1994, "Landfill Liner Interface Strengths from Torsional Ring Shear Tests," Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 3, pp. 597 - 615.

Eid, H. T., Stark, T. D., Evans, W. D., and Sherry, P., 2000, "Municipal Solid Waste Landfill Slope Failure I: Foundation and Waste Properties," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 5, pp. 397 - 407.

FEMA, 2001, NEHRP Recommended Provisions (National Earthquake Hazards Reduction Program) for Seismic Regulations for New Buildings and Other Structures, Part 2: Commentary (FEMA 369) Building Seismic Safety Council, Washington, D.C.

Stark, T.D., Eid, H.T., Evans, W.D., and Sherry, P., 2000, "Municipal Solid Waste Landfill Slope Failure II: Stability Analyses," Journal of Geotechnical and Geoenvironmental Engineering., ASCE, Vol. 126, No. 5, pp. 408-419.

Thiel, R., 2001 , "Peak vs Residual Shear Strength for Landfill Bottom Liner Stability Analyses," Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.

United States Environmental Protection Agency, Solid Waste and Emergency Response (5305), November 1993, revised April 1998, EPA530-R-93-017, "Solid Waste Disposal Facility Criteria, Technical Manual."

United States Environmental Protection Agency, Office of Research and Development, April 1995, EPA/600/R-95/051, RCRA Subtitle D (258) "Seismic Design Guidance for Municipal Solid Waste

Landfill Facilities." Available as of the writing of this policy at www.epa.gov/clhtml/pubtitle.html on the U.S. EPA Web site.

United States Geological Survey, 1996, National Seismic Hazard Mapping Project, "Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary)."