# CHAPTER 9

# SHALLOW FAILURE ANALYSIS

This chapter provides information to use when analyzing the potential for shallow translational failures or shallow rotational failures of *internal slopes* and *final slopes* (see **Figure f-1** on page xii) of an Ohio *waste containment facility*. Most *internal slopes* will need to remain stable until buttressed with waste or fill. However, some *internal slopes*, such as those at waste water lagoons, and all *final slopes* need to remain stable indefinitely.

> Ohio EPA considers any failure that occurs through a material or along an interface on a slope that is greater than five percent and that is loaded with 1,440 psf or less above a geosynthetic to be a shallow failure. This load was designated because it is reasonable to expect that most cap systems will have less than 1,440 psf permanent loading, and under those conditions, it is generally accepted practice to use peak interface shear strengths during stability analyses. Whereas, slopes loaded with more than 1,440 psf above a geosynthetic will generally be more deeply buried and necessitate the use of residual interface shear strengths during stability analyses.

Shallow translational failures occur along the weakest interfaces, and shallow rotational failures occur through the weakest layers of a slope. Translational failures are more prevalent in slopes containing geosynthetics, and rotational failures are more prevalent in slopes that do not contain geosynthetics. While these types of failures tend not to be catastrophic in nature, they can be detrimental to human health and the environment and costly to repair.

Shallow rotational failures of roads, benches, and berms built on top of a cap system (with or without geosynthetics in the cap) must be analyzed to ensure that the structures will remain stable. In most cases, shallow rotational failure surfaces of these types of structures can be successfully analyzed using the same types of computer modeling software as those used for deep-seated failure analysis. However, when using the computer modeling software for shallow rotational failure analysis, the search parameters need to be set to force the software to search for failure surfaces through the shallow surfaces of the cap, including roads, berms, and benches.

## REPORTING

Ohio EPA recommends that the results of the shallow failure surface analysis be included in their own section of the geotechnical and stability analyses report. At a minimum, the following information about the shallow failure analysis should be reported to Ohio EPA:

> 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 describing the results of the shallow failure analysis,

- One or more tables summarizing the results of the shallow failure analysis for each cross section analyzed,

- One or more tables summarizing the internal and interface shear strengths of the various components of the *internal slopes* and *final slopes*,


**Figure 9-1** An example of a shallow rotational failure of soil.

- Graphical depictions of any non-linear failure envelopes being proposed for each interface, material, and composite system (e.g., see **Figure 4-5** on page 4-23),

- A narrative justifying the assumptions used in the calculations,

- The scope, extent, and findings of the subsurface investigation as they pertain to the analyses of potential shallow failures at the *waste containment facility*,

- Plan views of the *internal slope* and *final slope* grading plans, clearly showing the location of the worst-case cross sections, northings and eastings, and the limits of the *waste containment unit(s)*,

- Drawings of the worst-case cross sections, including the slope components (e.g., geosynthetics, soil cover material, drainage layers, RSL, waste, drainage pipes, temporal high *phreatic* and *piezometric surfaces*),

- Stability calculations for *unsaturated internal slopes* and *final slopes* assuming static conditions,

- Stability calculations for *saturated internal slopes* and *final slopes* assuming static conditions,

- Stability calculations for *unsaturated final slopes* assuming seismic conditions,

- Any other necessary calculations, and

- Any figures, drawings, or references relied upon during the analysis. This includes copies of the most recent final version of the following figures showing the facility's location on each.

  - **Figure 9-6** on page 9-18: The 50-year 1-hour storm map of Ohio,
  - **Figure 9-7** on page 9-18: The 100-year 1-hour storm map of Ohio,
  - **Figure 9-8** on page 9-19: A map of Ohio showing the peak acceleration (%g) with 2% probability of exceedance in 50 years, and
  - Any other charts, graphs, data, and calculations used, marked to show how they apply to the facility.

## FACTORS OF SAFETY

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

Static analysis assuming *unsaturated* conditions: FS $\geq$1.50
Static analysis assuming *saturated* conditions: FS $\geq$1.10
Seismic analysis assuming *unsaturated* conditions: 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 against shallow failures 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 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 shallow failures. Because of the uncertainties involved when calculating the factors of safety, and because shallow failures may cause damage to other engineered components, if a facility has a static factor of safety against shallow failure lower than those listed above for *saturated* or *unsaturated* conditions, then different materials will need to be specified or different geometries will need to be used to design the slopes such that the required factors of safety are provided.

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.

If unusual circumstances exist at a facility, such as an *internal slope* with a leachate collection system that has a very high hydraulic conductivity drainage material, appropriate piping and pump settings that will quickly carry liquids away from the toe of the slope, a drainage layer that is protected from intrusion, freezing, and clogging, and appropriate calculations that demonstrate that little or no probability exists of any head building up on the slope during the worst-case weather scenario, then the *responsible party* may propose (this does not imply approval will be granted) to omit a shallow translational failure analysis assuming *saturated* conditions. The proposal should include any pertinent information necessary for demonstrating the appropriateness of omitting the shallow failure analysis assuming *saturated* conditions for the slope.

A design with a seismic factor of safety less than 1.00 against shallow failure indicates a failure may occur if a 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 against shallow failure. This is because geosynthetics are susceptible to damage at small deformations. Failure to the *waste containment facility* due to a shallow failure may damage other engineered components and is likely to increase harm to human health and the environment. If a facility has a seismic factor of safety against shallow failure less than 1.00, then different materials will need to be specified or different geometries will need to be used to design the slopes such that the required factor of safety is provided.

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 shallow 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 shallow 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 shallow 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 shallow 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 shallow failures, the following will usually apply:

- For foundation soils of *internal slopes*; use the lowest representative shear strength values for the *soil unit* immediately under the RSL. If multiple *soil units* intersect the *internal slope*, use the shear strength from the weakest *soil unit* that intersects the RSL. These values will usually be available because the subsurface investigation must be completed before conducting stability analyses. Linear shear strength envelopes for foundation materials should be developed from nonlinear shear strength envelopes that start at the origin (see Conformance Testing in Chapter 4 starting on page 4-15 for more information about nonlinear shear strength envelopes). To develop a linear shear strength envelope for the purposes of determining cohesion and $\phi$, for foundation materials, use the portion of the nonlinear envelope that extends entirely across the normal stresses expected above the top of the foundation material surface on the *internal slope* after the composite liner system is in place, and before it is loaded with waste or waste water.

- When the foundation material of a *final slope* is waste; assume the waste and the interface of the waste with the RSL will be at least as strong as the internal strength of the RSL, unless reason exists to believe otherwise.

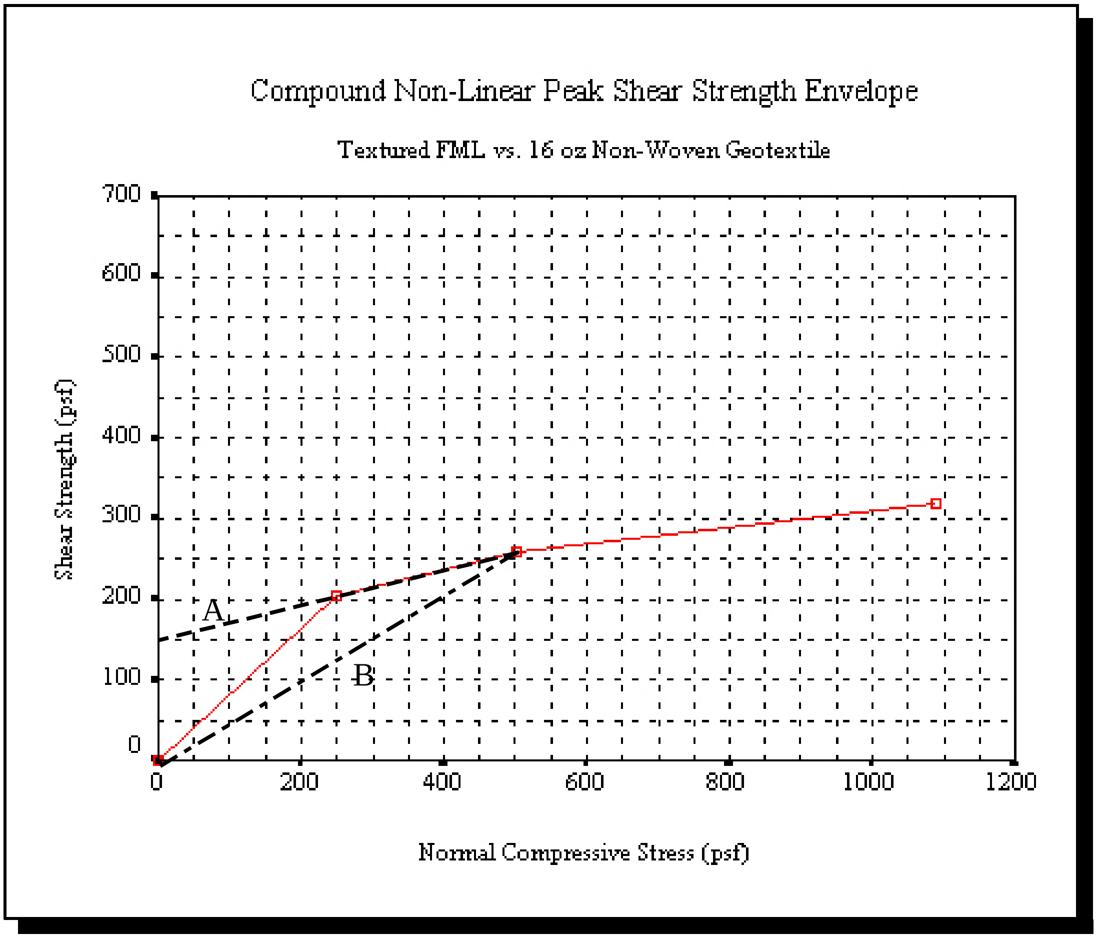



**Figure 9-2** An example of a compound nonlinear *peak shear strength* envelope from test results of textured FML/GT interfaces that will be used on a final slope with no tack-on benches or roads, having a 1-ft leachate drainage layer covered by a 2-ft thick protective layer. For this facility, the sand drainage layer and soil protective layer produce approximately 365 psf normal stress on the interface [(1 x 125 pcf) + (2 x 120 pcf)]. For modeling purposes, either A or B could be used to represent the shear strength of this interface in an infinite slope calculation, or the shear stress corresponding to 365 psf normal stress (230 psf) could be used with a $\phi = 0$. As an alternative, the non-linear envelope could be used in modeling software such as XSTABL.

- For structural fill and recompacted soil components; soil materials may have been compacted in the laboratory using the minimum 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, strength values for each engineered component made of structural fill or recompacted layers should be modeled using the values obtained from testing of the materials that represent the weakest materials that will be used during construction. Linear shear strength envelopes for structural fill and RSL materials should be developed from compound nonlinear shear strength envelopes that start at the origin. To develop a linear shear strength envelope for the purposes of determining cohesion and $\phi$, for RSL or structural fill, use the portion of the nonlinear envelope that extends entirely across the normal stresses expected above the RSL or structural fill component. For a composite liner system on an *internal slope*, this is the range of normal stresses caused by the composite liner system before any waste or waste water is in place. For a composite cap system on a *final slope*, this is the range of the normal stresses caused by the composite cap system drainage layer and the *protective layer*, tack-on benches and roads, and deployment equipment.

For example, if the RSL of a composite cap system with a 3-foot thick *protective layer* on top (including a drainage layer) with no benches or roads exhibits a compound nonlinear peak shear stress envelope such as shown in **Figure 9-2** on page 9-5, then the expected range of normal stress in the field would be less than 500 psf [1.0 ft x 125 pcf) + (2 x 120) = 365.0 psf]. As a result, from **Figure 9-2**, it can be seen that a c = 230 psf and a $\phi = 0^{o}$, a c and $\phi$ derived from line A, or a $\phi$ derived from line B could be used in an infinite slope analysis of the RSL of this composite cap system. As an alternative, the entire non-linear shear strength envelope could be used in a computer modeling software such as XSTABL. See Conformance Testing in Chapter 4 starting on page 4-15 for more information about developing nonlinear shear strength envelopes. This example does not take into account the stress created by deployment equipment. A designer should consider evaluating the slope in light of the deployment equipment weight to avoid mobilizing post-peak shear strength in the materials or creating an unexpected failure during construction as has happened at some facilities in Ohio.

- For interface shear strengths with geosynthetics, it is recommended that the shallow failure analysis be used to determine the minimum interface shear strengths that are necessary to provide the required factors of safety. This will provide the maximum flexibility for choosing materials during construction.

- For internal shear strengths of GCLs and RSLs, it is recommended that the shallow failure analysis be used to determine the minimum internal shear strengths of GCLs and RSLs that are necessary to provide the required factors of safety. This will provide the maximum flexibility when using these materials during construction.

- The resultant values determined by the shallow failure analysis calculations for interface and internal *peak shear strengths* and *residual shear strengths* should assume cohesion (c) is 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).

The design phase should include a determination of the weakest internal and interface shear strengths that the materials in each component need to exhibit to provide stability for the *waste containment facility*. These minimum shear strengths must then become part of the project design specifications. *Conformance testing* of the internal and interface shear strengths of construction materials must be conducted prior to use to verify that they will provide the shear strength necessary to meet the stability requirements of the design.

For shallow failure analysis of *internal slopes* and *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 geosynthetic interfaces,

- Internal *peak shear strengths* may be used for reinforced GCL,

- Internal and interface *residual shear strengths* must be used for unreinforced GCL,

- Internal *peak shear strengths* may be used for soil materials.

*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 the facility is likely to cause enough displacement within an interface that a post-peak shear strength will be mobilized (see **Figure f-2** on page xiv).

Sometimes, Ohio EPA may require composite systems comprising multiple geosynthetic interfaces to be tested to determine which interface or material will be the locus of the failure surface throughout the range of normal stresses expected in the field. This may entail using a direct shear device or other appropriate device to test *specimens* containing all the layers of a composite system. For example, if *residual shear strengths* were appropriate for an analysis, and all of the *peak shear strengths* for each interface and material are near each other, but a wide range of *residual shear strengths* exist, either the lowest *residual shear strength* measured will need to be used, or *specimens* comprising all the layers in a composite system will need to be tested.

The site conditions existing during construction, operations, and closure should be taken into account. For example:

- During static conditions, the soil portion of an RSL / FML interface may increase in moisture content due to leachate seeps, migration of ground water, or condensation. This can reduce the shear strength of the interface and cause slope failure.

- After a period of wet weather that has caused the *protective layer* to reach field capacity, a large rain event may occur and cause pore water pressure in a drainage layer of a cap or bottom liner to increase until a failure occurs at the FML/drainage layer interface.

- During the construction of an *internal slope* of a *waste containment facility*, a granular drainage layer being placed from the top of the slope to the bottom may create a driving force on the slope that exceeds the assumptions of the stability analysis, causing a failure.


**Figure 9-3** A translational failure through RSL at an Ohio landfill triggered by filling granular drainage material downslope.

## ANALYSIS

Two types of slopes will be focused on in this section: *internal slopes* (e.g., the interior side slope liner of a landfill or lagoon) 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* 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.

## Static Analysis

When performing a shallow failure analysis of an *internal slope* or *final slope*, the worst-case cross sections should be determined, taking into account known shear strengths of the materials, the steepest slope angle, and longest slope length. In cases where the worst-case slopes do not meet the required factors of safety, it must be ensured that no other slopes fail to meet the required factors of safety. Once all the slopes that do not meet the required factors of safety are identified, adjustments to the material specifications and/or facility design can be made to ensure that the required factors of safety are achieved for all slopes.

Shallow rotational failures of roads, benches, and berms built on top of a cap system (with or without geosynthetics in the cap) must be analyzed to ensure that the structures will remain stable. In most cases, shallow rotational failure surfaces of these types of structures can be successfully analyzed using the same types of computer modeling software as those used for deep-seated failure analysis. However, when using the computer modeling software for shallow rotational failure analysis, the search parameters need to be set to force the software to search for failure surfaces through the shallow surfaces of the cap, including roads, berms, and benches.

### Static Saturated Analysis

When calculating the static factor of safety against shallow failure for *saturated* conditions, the worst-case cross sections should be based on the following:

*Internal slopes*

- For *internal slopes* with a *protective layer* over the drainage layer (e.g., a granular layer over a geocomposite), use the steepest slope angle, use the longest slope length between slope drainage structures, assume the moisture content of the *protective layer* is at field capacity, and use the calculated head on the weakest interface affected by the pore water pressure that develops in the drainage layer during the design storm. Ohio EPA recommends using a fifty-year one hour storm (see **Figure 9-6** on page 9-18),

- For *internal slopes* with a drainage layer having no *protective layer* on top (e.g., a granular leachate collection layer), use the steepest slope angle, use the longest slope length between slope drainage structures, and use the calculated head that will develop on the weakest interface affected by the pore water pressure that develops in the drainage layer during the design storm. Ohio EPA recommends using a fifty-year one hour storm (see **Figure 9-6** on page 9-18),

> Based on observations of performance at Ohio landfills, it appears that a granular drainage layer on *internal slopes* should have a hydraulic conductivity of 0.5 to 1.0 cm/sec. Granular drainage layers with hydraulic conductivities less than this may cause failure of the frost protection layer, leachate collection system, cushion layer, and geomembrane. Even if the geomembrane is not damaged from this type of failure, it may be exposed to UV degradation for several months before repairs can be conducted. If this type of failure occurs during winter, the RSL under the geomembrane may be damaged by freeze/thaw cycles, which would require it to be rebuilt.

*Final slopes*

- Use the steepest slope and the longest slope length between slope drainage structures, assume the moisture content of the *protective layer* is at field capacity, and use the calculated head on the weakest interface affected by the pore water pressure that develops in the drainage layer during the one hundred-year one hour storm (see **Figure 9-7** on page 9-18).

Two of the scenarios above include *protective layers*. They represent field conditions where a storm occurs after a period of wet weather that has caused the *protective layer* to reach field capacity. Therefore, "there is no additional storage capacity, and the infiltrating water all passes through the system as percolation in accordance with Darcy's formula" (Soong and Koerner, 1997). This means that correctly estimating the hydraulic conductivity of the protective layer ($k_c$) is critical to properly estimating the inflow of water to the cap drainage layer. The value used should be representative of the hydraulic conductivity of the protective layer after it has been in place long enough to have experienced freeze/thaw cycles, wet/dry cycles, root penetration, insect and animal burrowing, and other physical weathering. A typical value of $1 \times 10^{-4}$ cm/sec has been offered by Richardson. However, USDA soil surveys, and on-site testing of the hydraulic conductivity of long-time undisturbed vegetated areas could also be used for determining $k_c$. If another method of calculating the head on the weakest interface ($h_{avg}$) is used, the alternative method should also assume that the cover soil has reached field capacity.

## Seismic Analysis

When calculating the seismic factor of safety for *final slopes* that include geosynthetic interfaces, the worst-case cross sections should be determined using the steepest slope angle and slope geometry, using *unsaturated* conditions, and assuming typical head conditions in the drainage layer, if a drainage layer is part of the design.

For shallow failure analysis, the methodology for seismic analysis applies the horizontal force at the failure surface. As a result, the highest peak horizontal ground acceleration expected at any point along the failure surface should be used.

### Determining a Horizontal Ground Acceleration to Use for Seismic Analysis

Selecting an appropriate horizontal 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 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 acceleration as the *bedrock*. Facilities founded on soft 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 acceleration as it reaches the base of the facility.

Waste and structural fill can cause the horizontal acceleration experienced at the base of the 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 acceleration will need to be determined for each facility so that the appropriate horizontal acceleration at the expected shallow failure surface can be estimated for stability modeling purposes. MSW is typically a relatively low density, somewhat elastic material. It is expected that the horizontal acceleration at the base of a MSW facility will be amplified as it progresses towards surfaces 100 feet or less above the ground surface (see **Figure 9-9** on page 9-20). 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 9-9**). To determine the effects of structural fill and industrial wastes, such as flue gas desulfurization dust, cement kiln dust, lime kiln dust, foundry sands, slags, and dewatered sludges on the horizontal acceleration, the characteristics of the materials will need to be determined either by measuring shear wave velocities or by demonstrating the similarity of the materials to compacted earth dam material, *bedrock*, or deep cohesionless soils and applying the above noted figures.

Alternative methods for determining site-specific adjustments to expected horizontal accelerations may be also 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 USACOE, Engineer Research and Development Center, Vicksburg, MS, is then used to calculate the accelerations at different elevations in the facility. Because of the differences between earthquakes that occur in the western United States and earthquakes that occur in 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.

Ohio EPA requires that the seismic coefficient ($n_g$) used in numerous stability modeling calculations be based on the horizontal acceleration of peak ground acceleration from a final version of the most recent USGS “National Seismic Hazard Map” (e.g., see **Figure 9-8** on page 9-19) 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 at www.usgs.gov on the USGS Web site. Once the facility location on the map has been determined, then the peak horizontal acceleration indicated on the map may be adjusted for dampening effects and must be adjusted for the amplification effects of the soils, engineered components, and waste at the facility as discussed above. If instrumented historical records show 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.

### Anchoring Geosynthetics on Internal Slopes

An anchor runout is a portion of geosynthetic that extends beyond the crest of a slope and is weighted with soil or other material to hold the geosynthetic in place (see **Figure 9-4**, **A**). An anchor trench usually occurs at the end of a runout. A trench is dug beyond the crest of a slope, and the end of the runout material drops into the trench that is then back filled with soil or other material to hold the geosynthetic in place (see **Figure 9-4**, **B**).

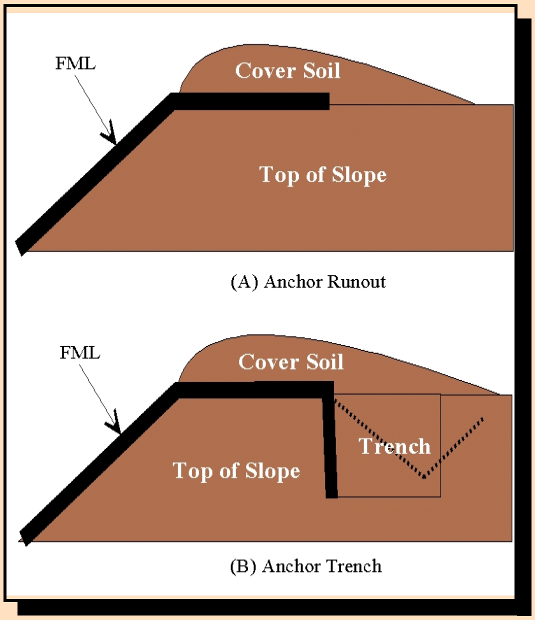



**Figure 9-4** Example detail of (A) anchor runout and (B) anchor trench. Anchor trenches can also be "V" shaped (dashed line).

Anchorages are used with geosynthetics for the following reasons:

- To hold the geosynthetics in place during installation of subsequent layers,
- To prevent surface water from flowing beneath the geosynthetics anytime during or after installation. This is necessary because flowing water damages the underlying soil layers and decreases the interface shear strength of the liner system, and
- To prevent surface water from entering any leak detection layers or drainage layers. This is necessary because suspended soils may enter those layers and lead to clogging. That in turn, can cause an increase in water pressure and a decrease in interface shear strength of the layers. Surface water infiltration into a leak detection layer of a *waste containment facility* can increase the cost of leachate treatment and unwarranted concern that the primary liner is leaking.

Although the tensile strength of geosynthetics must not be taken into account when evaluating stability, it is appropriate when analyzing the performance of anchorages. This is because it is necessary to determine if geosynthetics will pull out of their anchorages or rip.

In 1997, Ohio EPA issued a stop work order at a stabilized hazardous waste closure unit. More than two dozen tears and ripped seams occurred in the geotextile filter layer between the granular *protective layer* and the geonet drainage layer that was part of the primary composite liner/leachate collection system. Long tears developed at the crest of the *internal slope* at the beginning of the anchor runout and other areas. Work was stopped until the granular drainage layer could be removed and the geonet and geotextile inspected, repaired, or replaced as needed.

It is generally accepted that most anchorages are over-designed and are likely to result in tearing of geosynthetics should unexpected tensile stresses occur. Designers should consider using a less robust design for anchorages to reduce the likelihood that geosynthetics will tear if unexpected tensile stresses occur.

Some designers recommend attempting to direct a failure to a specific interface, often called a "slip layer," when concern exists about the ability of an essential geosynthetic component (e.g., a geomembrane liner) to withstand unanticipated tensile strain.

The slip layer is placed above the essential geosynthetic it is protecting. The slip layer material is chosen so that its interface shear strength will be lower than the interface shear strength of the essential geosynthetic with its underlying material. The anchorage for the slip layer is designed to release before the essential geosynthetic will pull out of its anchorage. This increases the probability that the slip layer interface will fail first and leave the essential geosynthetic in place and intact, hopefully preserving containment. Even if a facility incorporates a slip layer in the design, it must be stable without relying on the tensile strength of the geosynthetics including the slip layer if one is used.

**Factor of Safety Against Shallow Failure - Example Method**

Many alternatives exist to analyze *internal slopes* and *final slopes* for susceptibility to shallow translational and rotational failures, ranging from computer modeling to hand calculations.  For shallow translational failures, a typical method used is a limit equilibrium method calculated using a spreadsheet.  Some examples of these equations can be found in the following references;

Giroud, J. P., Bachus, R. C.  and Bonaparte, R., 1995, "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes," Geosynthetics International, Vol. 2, No. 6, pp. 1149 - 1180.

Matasovic, N., 1991, "Selection of Method for Seismic Slope Stability Analysis," Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper 7.20, March 11 - 15, pp. 1057 - 1062.  St. Louis, Missouri.

Soong, T. Y. and Koerner, R. M., 1997, "The Design of Drainage Systems Over Geosynthetically Lined Slopes," GRI Report #19.

Of these, Matasovic, 1991, is the simplest to use, involves an infinite slope analysis, uses a seismic coefficient, and tends to be more conservative.  It also provides results comparable to computer modeling software such as XSTABL.

$$FS = \frac{\dfrac{c}{g_c z_c \cos^2 b} + \tan f \left[1 - \dfrac{g_w (z_c - d_w)}{g_c z_c}\right] - n_g (\tan b)(\tan f)}{n_g + \tan b} \qquad (9.1)$$

$$f = \tan^{-1}\left[\frac{FS(n_g + \tan b) - \left(\dfrac{c}{g_c z_c \cos^2 b}\right)}{\left[1 - \dfrac{g_w (z_c - d_w)}{g_c z_c}\right] - n_g \tan b}\right] \qquad (9.1.1)$$

where $FS$ = factor of safety against shallow failure,

$n_g$ = peak horizontal acceleration at the failure surface (%g),

$\gamma_c$ = field density of cover materials,

$\gamma_w$ = density of water,

$c$ = cohesion of failure surface,

$\phi$ = internal angle of friction,

$\beta$ = angle of slope,

$z_c$ = depth of cover soils, and

$d_w$ = depth to water table that is assumed parallel to slope ($d_w = z - h_{avg}$), (see Equation 9.2, 9.3, or 9.4 for $h_{avg}$).

**Calculating Head on the Weakest Interface - Example Method**

The expected head on the weakest interface ($h_{avg}$) may be estimated by hand or spreadsheet calculations using the equations such as those based on work performed by Koerner, Soong, Daniel, Thiel, Stewart, or Giroud (see references at end of this chapter). This equation assumes that a storm occurs after a period of wet weather that has caused the cover soil to reach field capacity. Therefore, "there is no additional storage capacity and the infiltrating water all passes through the system as percolation in accordance with Darcy's formula" (Soong and Koerner, 1997). If another method of calculating the head on the weakest interface is used, then that method should also assume that the cover soil has reached field capacity.

$$h_{avg} = \frac{P(1 - RC) \cdot L(\cos\beta)}{k_d(\sin\beta)} \qquad (9.2)$$

or if $P(1\text{-}RC) > k_c$ use:

$$h_{avg} = \frac{k_c \cdot L(\cos\beta)}{k_d(\sin\beta)} \qquad (9.3)$$

or if $h_{avg}$ from the above calculation is > $T_d$ then use:

$$h_{avg} = T_d + T_c \qquad (9.4)$$

- $h_{avg}$ = average head,
- $P$ = precipitation,
- $\beta$ = angle of slope,
- $L$ = slope length,
- $T_c$ = thickness of cover soil,
- $RC$ = runoff coefficient (SCS Runoff Curve Number/100),
- $k_d$ = permeability of drainage layer. Apply reduction factors if geocomposite (see Richardson and Zhao, 1999; or Koerner, 1997),
- $T_d$ = thickness of drainage layer, and
- $k_c$ = permeability of cover material. Use a $k_c$ that represents long term field conditions (assume $1\times10^{-4}$ cm/sec, use USDA Soil Survey estimates, or do in-field testing of a long-term vegetated area adjacent to the facility).

**Figure 9-5** A shallow rotational failure in a containment berm at an ash settling pond in Ohio.

**Shallow Failure - Example Calculations**

A 200-ft high landfill in Ohio has 3(h):1(v) (18.43º) *internal slopes* and *final slopes*. The *final slopes* comprise 1.5 feet of RSL; a 40-mil textured FML; a 0.20-inch (0.508 cm) thick geocomposite drainage layer (GDL) with a transmissivity of $2.0\text{x}10^{-3}$ $m^2$/sec (k = 39.4 cm/sec). The GDL was tested with RSL/FML below it and protective layer above it, using a normal load of 500 psf between at a 0.32 gradient. Outlets are spaced at 130-foot (3,962.4 cm) intervals along the *final slopes*; and there is a 2.5-foot thick *protective layer* with a long-term permeability of $1.0\ \text{x}10^{-4}$ cm/sec. A good stand of grass (SCS Runoff Curve Number = 90) exists on the slope.

The *internal slopes* comprise 5-foot RSL, a 60-mil textured FML, and a 1-foot granular drainage layer (DL) with a permeability of 1 cm/sec along the slopes that rise 50 feet. A leachate collection pipe at 0.5 percent grade transects the slope so that the maximum distance of flow is 75 feet. This example assumes that the liner components will be chosen after the facility design has been approved. Therefore, the shear strengths determined by the following calculations will be used as the minimum requirements in the permit.

**Shallow Failure, Unsaturated Static Conditions - Example Calculation 1**

Determine the friction angle required for a 1.50 static factor of safety for the *internal slopes* and *final slopes* using the worst-case cross sections for the facility and Equation 9.1.1

*Internal slope*
$$f_{required} = \tan^{-1}\left[\frac{1.5\left(0g + \tan 18.43\right) - \left(\frac{0\,psf}{120\,pcf \bullet 1\,ft \bullet \cos^2 18.43}\right)}{\left[1 - \frac{62.4\,psf\,(1\,ft - 1\,ft)}{120\,pcf \bullet 1\,ft}\right] - 0g\left(\tan 18.43\right)}\right] = 26.56^{\circ}$$

*Final slope*
$$f_{required} = \tan^{-1}\left[\frac{1.5\left(0g + \tan 18.43\right) - \left(\frac{0\,psf}{120\,pcf \cdot 2.5\,ft \cdot \cos^2 18.43}\right)}{\left[1 - \frac{62.4\,psf\,(2.5\,ft - 2.5\,ft)}{120\,pcf \cdot 2.5\,ft}\right] - 0g\left(\tan 18.43\right)}\right] = 26.56^{\circ}$$

The minimum interface and internal *peak shear strength* of all materials required for both *internal slopes* and *final slopes* at this facility is 26.56º to obtain a 1.50 static factor of safety against shallow failure.

**Shallow Failure, Unsaturated Seismic Conditions - Example Calculation 2**

Determine the shear strength required for a 1.00 seismic factor of safety for the *final slopes* using the worst-case cross sections for the facility. **Figure 9-8** on page 9-19 shows an expected peak ground acceleration for the facility of 0.10 g. For shallow failure analysis of final cap, the highest horizontal acceleration expected on any surface should be used. Therefore, it is recommended that the peak horizontal acceleration for cap be estimated from **Figure 9-9** on page 9-20. This is because it is expected that the surfaces of slopes less than 100 feet high in a facility will experience the highest horizontal accelerations. Using an $n_g$ at the base of waste of 0.10g results in an estimated amplification to approximately 0.14g for cap at or below 100 ft from the ground surface. Calculate the shear strength required using Equation 9.1.1

*Final slope*
$$f = \tan^{-1}\left[\frac{1.0\left(0.14g + \tan 18.43\right) - \left(\frac{0\,psf}{120\,pcf \cdot 2.5\,ft \cdot \cos^2 18.43}\right)}{\left[1 - \frac{62.4\,psf\,(2.5\,ft - 2.5\,ft)}{120\,pcf \cdot 2.5\,ft}\right] - 0.14g\left(\tan 18.43\right)}\right] = 26.40^{\circ}$$

The minimum interface and internal *peak shear strength* for all materials in the composite cap system at this facility is 26.40º to obtain a 1.00 seismic factor of safety.

**Shallow Failure, Saturated Static Conditions - Example Calculation 3**

Determine the required shear strength to have a 1.10 static factor of safety for *internal slopes* and *final slopes* assuming *saturated* conditions. The RSL/FML interfaces for the *internal slopes* and *final slopes* are not affected by the pore water pressure developed in the drainage layer because the RSL/FML interface is separated from the drainage layer by the FML. However, the interfaces above the FML are affected by pore water on both slopes.

For *internal slopes*: The interface of interest is the FML/DL. Therefore, calculate the head on the interface during the 50-year, 1-hour storm using **Figure 9-6** on page 9-18 (2.75 in/hr) and Equation 9.2 because the DL is the *protective layer*. Calculate the required minimum shear strength using Equation 9.1.1.

From Equation 9.2 $$h_{avg} = \frac{1.94 \times 10^{-3}\, cm/\sec(1 - 0.0) \cdot 2286\, cm(\cos 18.43)}{1\, cm/\sec(\sin 18.43)} = 13.3\, cm = 0.436\ ft$$

From Equation 9.4 $d_w$ = 1 ft - 0.436 ft = 0.564 ft

From Equation 9.1.1 $$f_{required} = \tan^{-1}\left[\frac{1.1(0g + \tan 18.43) - \dfrac{0\ psf}{120\ pcf \cdot 1\ ft \cdot \cos^2 18.43}}{\left[1 - \dfrac{62.4\ psf\,(1\ ft - 0.564\ ft)}{120\ pcf \cdot 1\ ft}\right] - 0g(\tan 18.43)}\right] = 25.37^\circ$$

The minimum *peak shear strength* required to have a 1.10 static factor of safety for *internal slopes* before waste is placed in this facility is 25.37º for the interfaces above the FML during the design storm.

For *final slopes*: The interface of interest is the FML/GDL. Therefore, calculate the head on the interface during the 100-year, 1-hour storm using **Figure 9-7** on page 9-18, which is 3.0 in/hr ($2.12x10^{-3}$ cm/sec) and Equation 9.3 because P(1-RC) > $k_c$ (e.g., $2.12x10^{-3}$ cm/sec (1 - 0.9) > 1.0 $x10^{-4}$ cm/sec). Calculate the shear strength required using Equation 9.1.1.

Calculate the permeability of the geocomposite using the reduction factors recommended in Richardson and Zhao, 1999; Giroud, Zhao, and Richardson 2000; or Koerner, 1997.

$$\mathrm{Tr_L} = \frac{\mathrm{Tr_T}}{\mathrm{FS_I \cdot FS_{Cr} \cdot FS_{CC} \cdot FS_B \cdot FS_S}} \qquad (9.5)$$

where $Tr_L$ = long term transmissivity,
$Tr_T$ = tested transmissivity,
$FS_I$ = factor of safety to account for intrusion,
$FS_{Cr}$ = factor of safety to account for creep,
$FS_{CC}$ = factor of safety to account for chemical clogging,
$FS_B$ = factor of safety to account for biological clogging, and
$FS_S$ = factor of safety to account for clogging due to infiltration of fines.

From Equation 9.5: $$Tr_L = \frac{2.0x10^{-3}\, m^2\ /\sec}{1.5 \cdot 4 \cdot 1.0 \cdot 1.5 \cdot 4.0} = 5.56\, x\ 10^{-5}\, m^2/\sec$$

**Shallow Failure - Example Calculation 3 (contd.)**

Convert the transmissivity to hydraulic conductivity:

$$K_d = \frac{Tr_L}{Td} = \frac{5.56\,x10^{-5}\,m^2/\sec}{5.08\,x10^{-3}\,m} = 1.094x10^{-2}\ m/\sec = 1.094\,cm/\sec$$

From Equation 9.3 $$h_{avg} = \frac{1.0 \times 10^{-4}\,cm/\sec \cdot 3962.4\,cm(\cos 18.43)}{1.094\,cm/\sec(\sin 18.43)} = 1.0869\ \ cm = 0.03565 ft$$

$h_{avg}$ is thicker than the GDL, therefore: $d_w = 0$ ft

From Equation 9.1.1 $$f_{required} = \tan^{-1}\left[\frac{1.1(0g + \tan 18.43) - \dfrac{0\,psf}{120\,pcf \cdot 2.5\,ft \cdot \cos^2 18.43}}{\left[1 - \dfrac{62.4\ psf(2.5\ ft - 0.ft\ \ )}{120\,pcf \cdot 2.5\,ft}\right] - 0g(\tan 18.43)}\right] = 37.37^\circ$$

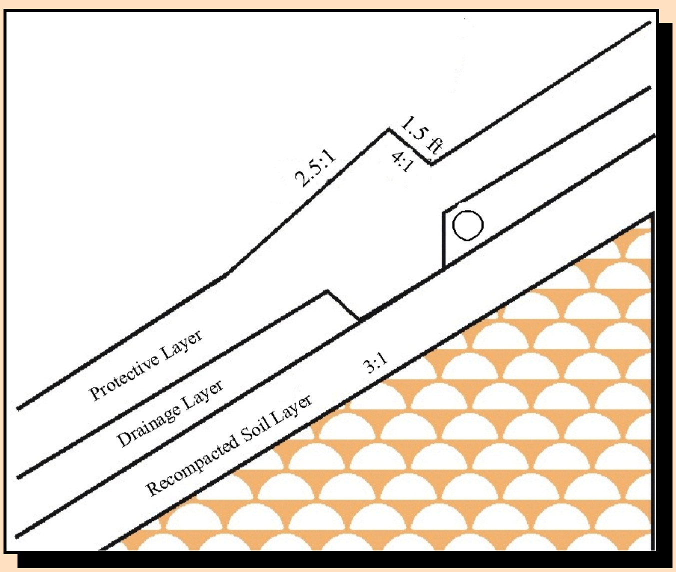
The minimum *peak shear strength* required for all interfaces and materials to have a static factor of safety equal to 1.10 for *final slopes* under *saturated* conditions at this facility is 37.37$^o$.

When multiple scenarios are analyzed to determine the minimum shear strength necessary to provide the required factors of safety, the scenario that produces the highest minimum factor of safety will be used to establish the minimum internal and interface shear strengths that the materials must exhibit to provide stability. For these examples, the minimum internal and interface peak shear strength that will provide stability in all analyzed scenarios is 37.37E.

**Shallow Failure Analysis of Final Cap with Tack-on Benches - Example Calculation**

OBJECTIVE
To evaluate the stability of the cap on a 200-ft high landfill, which has a 3 (h):1(v) *final slope*, comprised of 1.5-foot RSL; a 40-mil textured FML; a 1-foot thick DL with a permeability of 1 x10$^{-2}$ cm/sec; and a 1.5-foot thick *protective layer*. Outlets for the drainage layer are spaced at 130-foot intervals along the *final slope* at 1.5-foot high tack-on benches.

METHODOLOGY
Back calculate the necessary shear strengths of the RSL, FML, DL, and the protective layer and the permeability of the protective layer in order to maintain an acceptable FS.



**Shallow Failure with Tack-on Benches - Example Calculations (cont.)**

| Shallow Failure Analysis of Final Cap with Tack-on Benches Summary Table of Typical Worst-Case Conditions | | | |
|---|---|---|---|
| Component Being Evaluated | Method Used | Back Calculated Result | |
| Protective layer permeability | Formula 9.3 | $2.54 \times 10^{-5}$cm/sec[1] | |
| Protective layer shear strength | Shallow rotational XSTABL modeling[2] | c = 0 ϕ = 31E[1] | |
| RSL shear strength to provide an FS$1.50 under drained static conditions | Shallow rotational XSTABL modeling | Shear Strength[1] Envelope | |
| | | Normal Stress | Shear Stress |
| | | 0 | 0 |
| | | 288 | 275 |
| | | 576 | 300 |
| | | 1440 | 350 |
| FML vs. DL or RSL shear strength to provide an FS$1.50 under drained static conditions | Shallow translational XSTABL[2] modeling | Shear Strength[1] Envelope | |
| | | Normal Stress | Shear Stress |
| | | 0 | 0 |
| | | 288 | 215 |
| | | 576 | 275 |
| | | 1440 | 350 |

[1] This shear strength should be the required minimum specification for this component in the quality assurance quality control plan.

[2] see attached example outputs

| Shallow Failure Analysis of Final Cap with Tack-on Benches Summary Table of Typical Non-Worst-Case Conditions | | |
|---|---|---|
| Component Being Evaluated[1] | Method Used | Calculated FS |
| RSL shear strength saturated static | Shallow rotational XSTABL modeling | 1.459 |
| RSL shear strength drained seismic | Shallow rotational XSTABL modeling | 1.125 |
| FML vs. DL or RSL shear strength saturated static | Shallow translational XSTABL modeling | 1.398 |
| FML vs. DL or RSL shear strength drained seismic | Shallow translational XSTABL modeling | 1.250 |

[1] these cross section were evaluated using the input values determined by the typical worst-case conditions.

For more detailed information, see the XSTABL output at the end of this chapter.

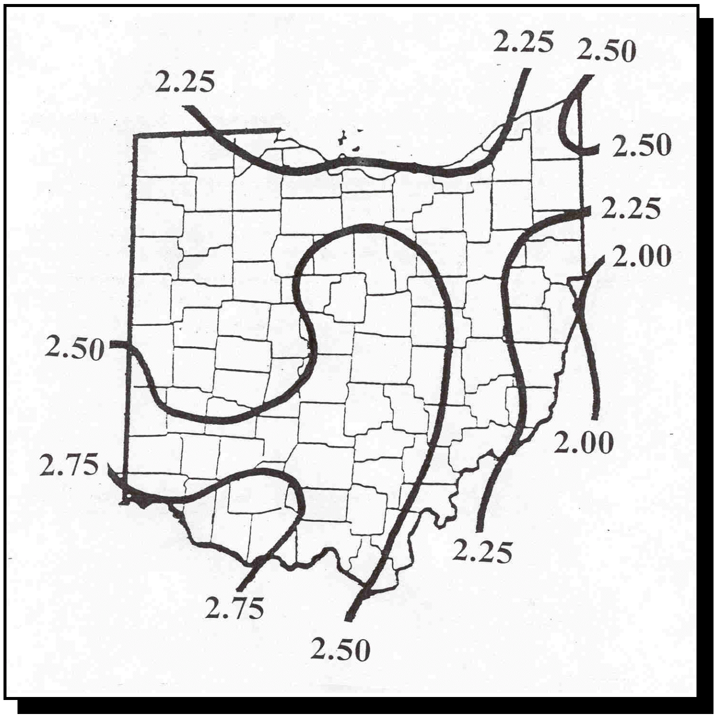



**Figure 9-6** The 50-year 1-hour storm. Spatial distribution of 1-hour rainfall (inches/hour). Huff, Floyd A., and Angle, James R., "Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

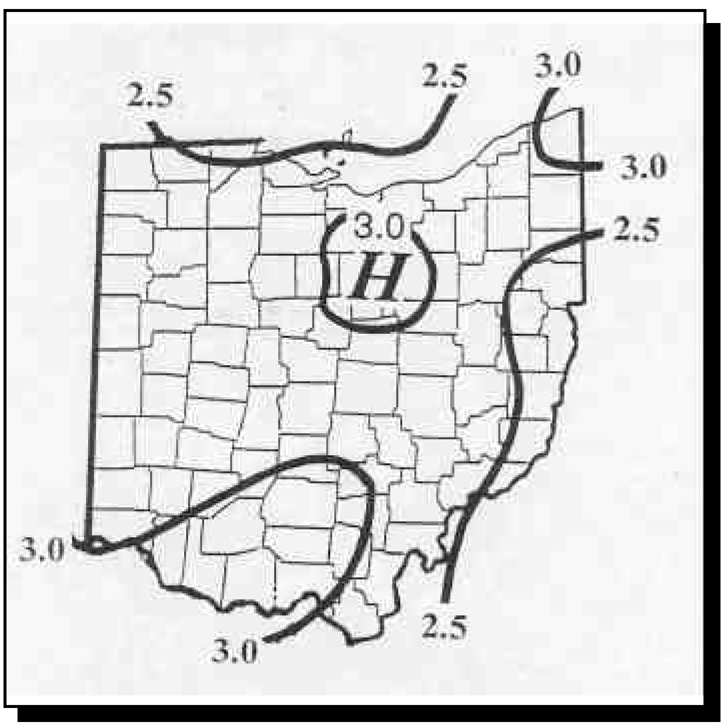



**Figure 9-7** The 100-year 1-hour storm. Spatial distribution of 1-hour rainfall (inches/hour). Huff, Floyd A., and Angle, James R,. "Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

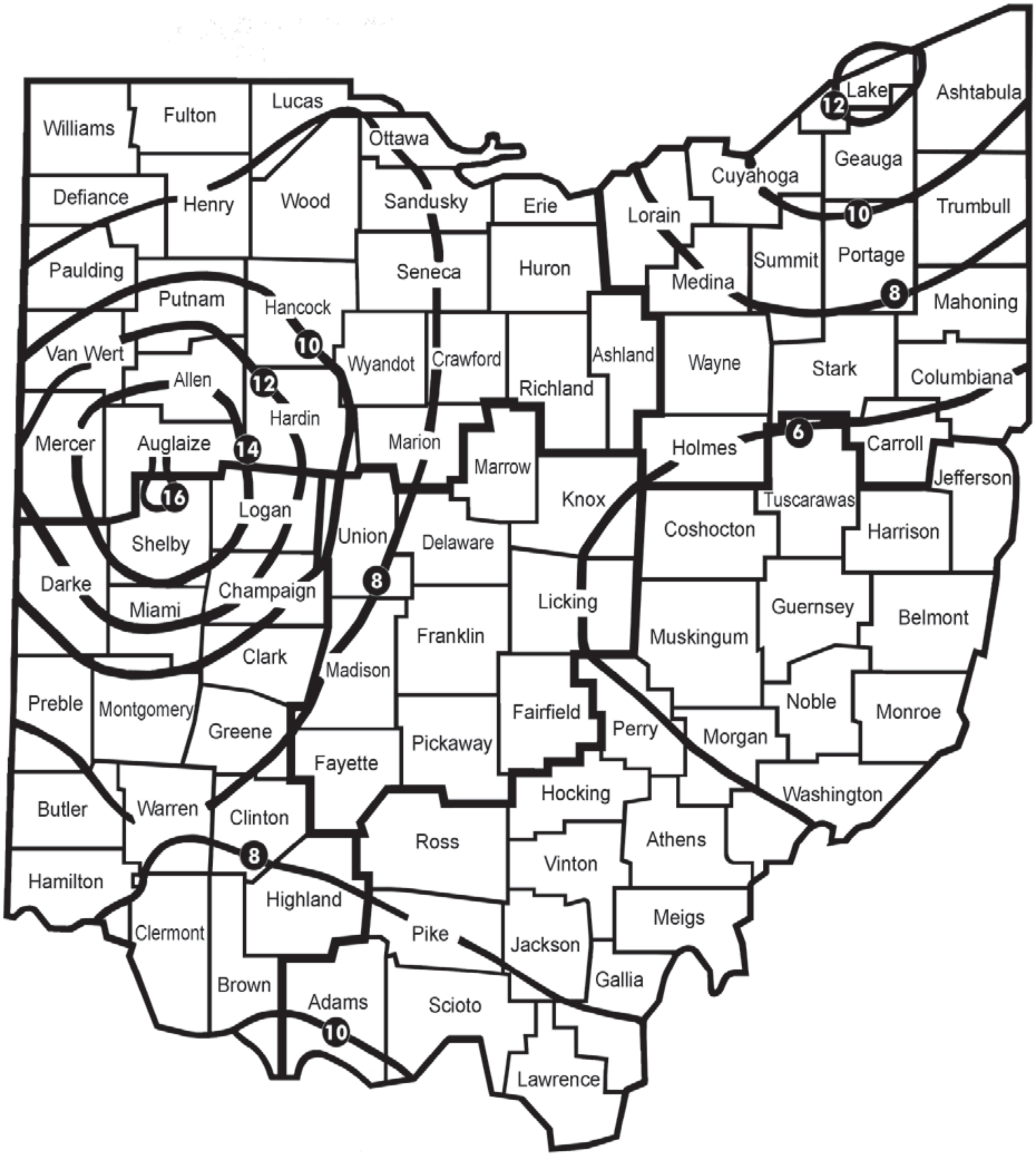



**Figure 9-8** The peak acceleration (%g) with 2% probability of exceedance in 50 years. U.S. Geological Survey (October 2002) National Seismic Hazard Mapping Project, "Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary)."

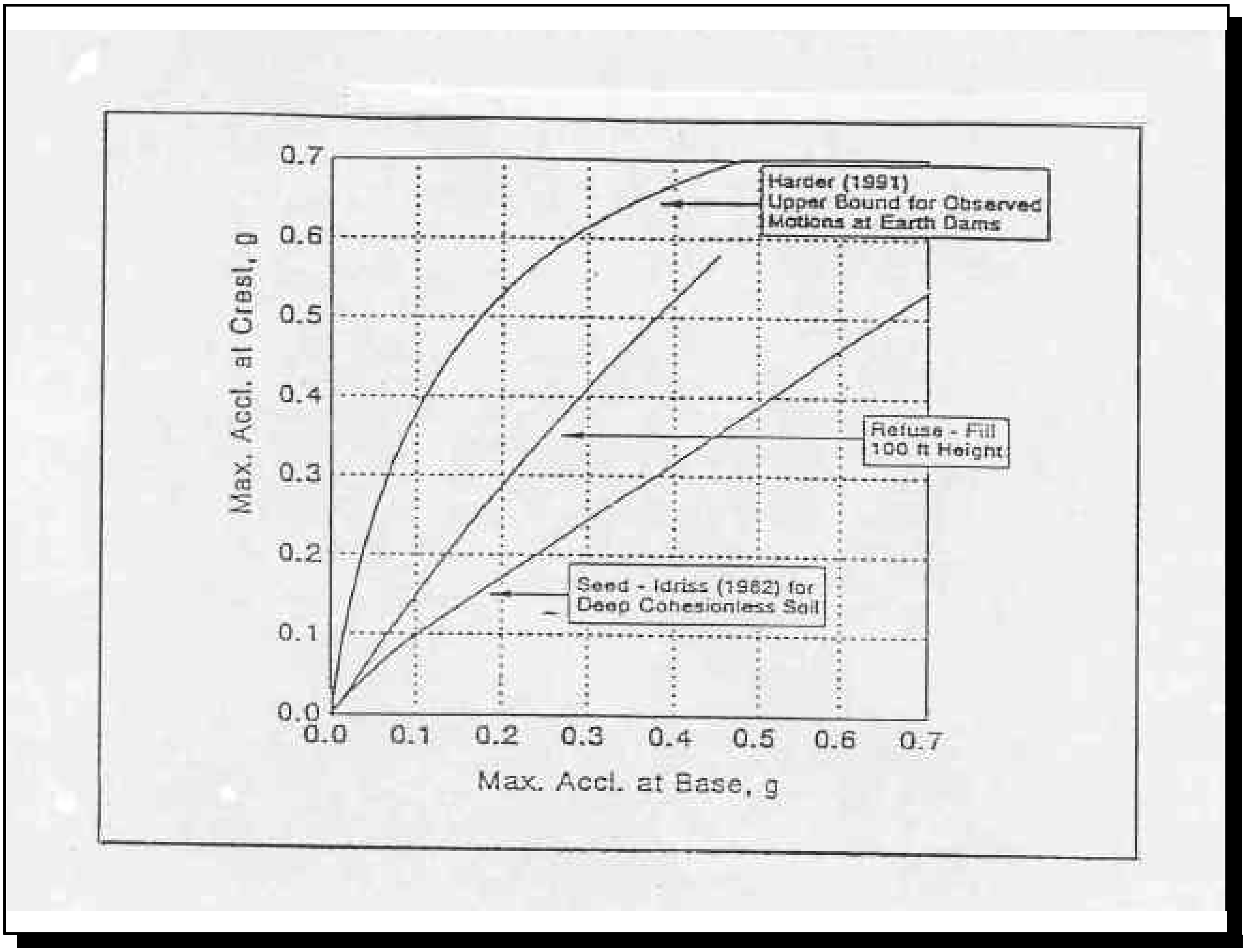


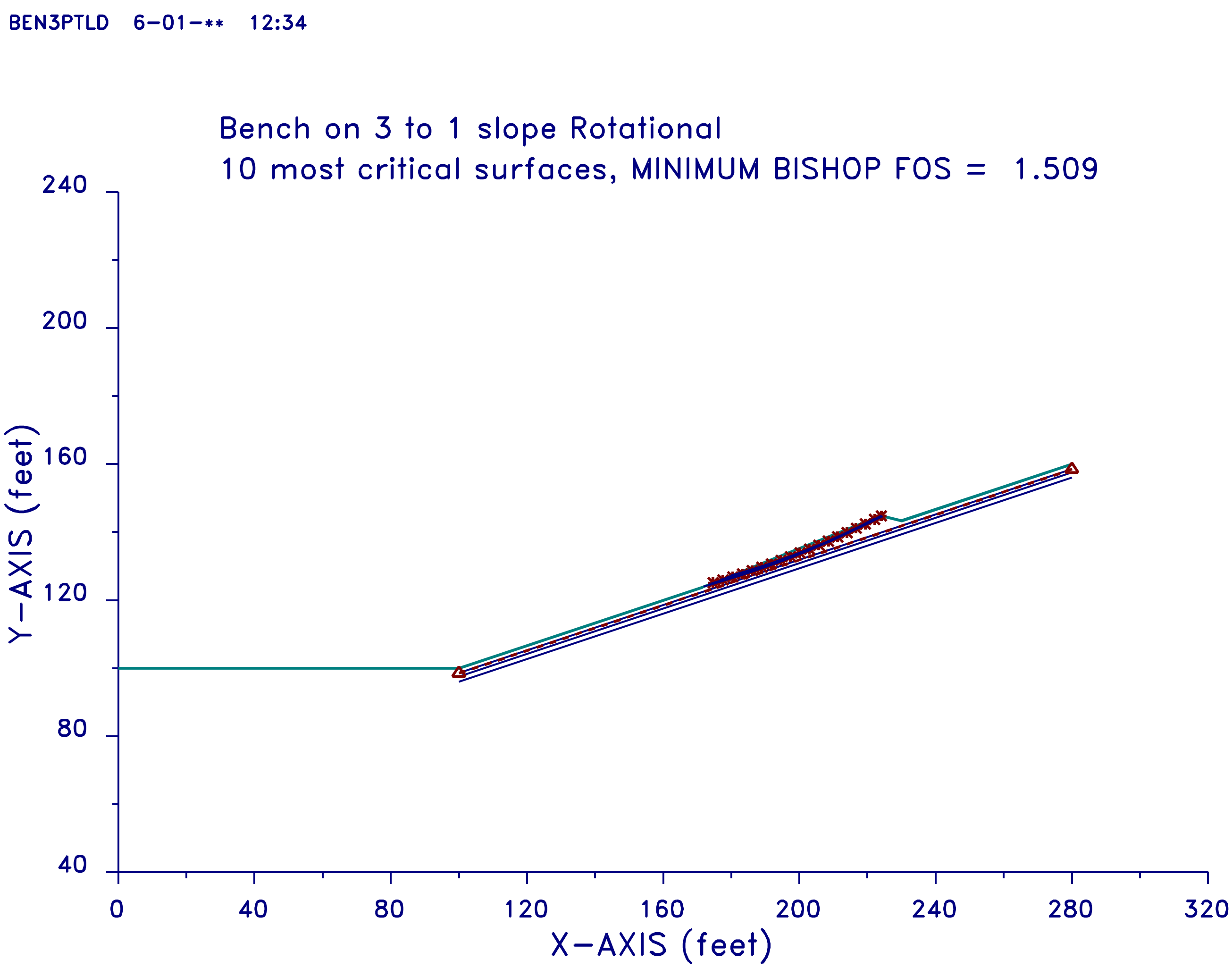

**Figure 9-9** The relationship between maximum horizontal seismic acceleration at the base and crest of 100 feet of refuse, on top of deep cohesionless soils, and on top of earth dams. Singh and Sun, 1995, Figure 3.

**Shallow Rotational Failure within Tack-on Benches - Example Computer Output**

XSTABL File: BEN3PTLD   6-01-**   12:34

```
                X S T A B L

        Slope Stability Analysis
              using the
          Method of Slices

        Copyright (C) 1992 - 98
              Interactive Software
         Designs, Inc.
        Moscow, ID 83843, U.S.A.

          All Rights Reserved

          Ver. 5.20296 ) 1697
```

Problem Description : Bench on 3 to 1 slope Rotational



## 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 | 100.0 | 100.0 | 100.0 | 1 |
| 2 | 100.0 | 100.0 | 171.5 | 123.8 | 1 |
| 3 | 171.5 | 123.8 | 224.0 | 144.8 | 1 |
| 4 | 224.0 | 144.8 | 230.0 | 143.3 | 1 |
| 5 | 230.0 | 143.3 | 280.0 | 160.0 | 1 |

3 SUBSURFACE boundary segments

| Segment No. | x-left (ft) | y-left (ft) | x-right (ft) | y-right (ft) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | 100.0 | 98.5 | 280.0 | 158.5 | 2 |
| 2 | 100.0 | 97.5 | 280.0 | 157.5 | 3 |
| 3 | 100.0 | 96.0 | 280.0 | 156.0 | 4 |

## ISOTROPIC Soil Parameters

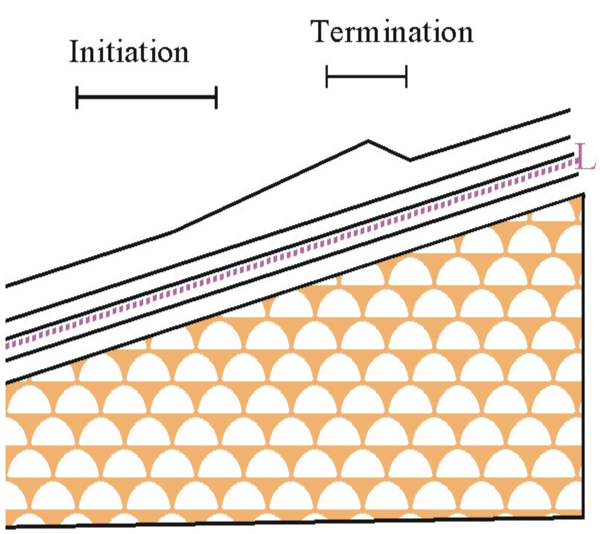
4 Soil unit(s) specified

| Soil Unit No. | Unit Moist (pcf) | Weight Sat. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Parameter Ru | Pressure Constant (psf) | Water Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 125.0 | .0 | 31.00 | .000 | .0 | 0 |
| 2 | 125.0 | 130.0 | .0 | 31.00 | .000 | .0 | 0 |
| 3 | 100.0 | 100.0 | .0 | .00 | .000 | .0 | 0 |
| 4 | 70.0 | 70.0 | 480.0 | 33.00 | .000 | .0 | 0 |

```
NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)


Soil Unit # 3

Point   Normal    Shear
 No.     Stress   Stress
         (psf)    (psf)

  1         .0       .0

  2      288.0    275.0

  3      576.0    300.0

  4     1440.0    350.0


-------------------------------------------------------------
BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED
-------------------------------------------------------------

LOWER limiting boundary of 1 segments:

Segment  x-left  y-left  x-right  y-right
  No.     (ft)    (ft)     (ft)     (ft)

   1     100.0    98.5    280.0    158.5
```

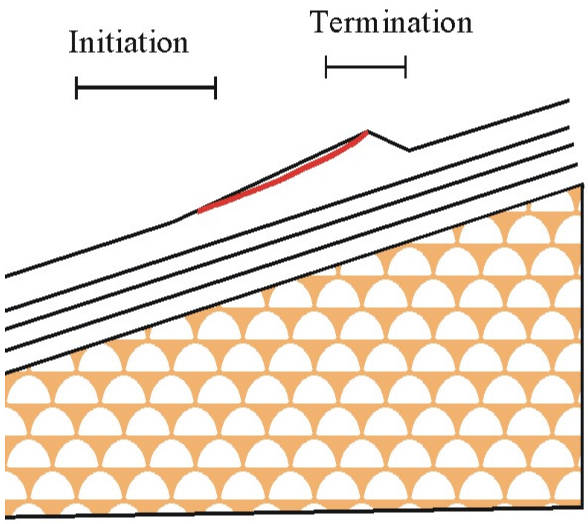
This limits the circular surfaces from being generated below the vegetative layer.

```
A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.


2500 trial surfaces will be
generated and analyzed.

50 Surfaces initiate from each of
50 points equally spaced
along the ground surface between
x = 160.0 ft and x = 180.0 ft

Each surface terminates
between x = 220.0 ft  and x = 230.0 ft


Unless further limitations were imposed,
the minimum elevation
at which a surface extends is y = .0 ft


 * * * * * DEFAULT SEGMENT LENGTH SELECTED BY
XSTABL * * * * *

 3.0 ft line segments define each trial failure
surface.
```



```
---------------------
ANGULAR RESTRICTIONS
---------------------

 The first segment of each failure surface will be inclined  within the angular range
defined by :

 Lower angular limit := -45.0 degrees
 Upper angular limit := (slope angle - 5.0) degrees

 Factors of safety have been calculated by the :

 * * * * * SIMPLIFIED BISHOP METHOD * * * * *

 The most critical circular failure surface
 is specified by 19 coordinate points
```

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 174.69 | 125.08 |
| 2 | 177.57 | 125.93 |
| 3 | 180.44 | 126.81 |
| 4 | 183.29 | 127.73 |
| 5 | 186.14 | 128.68 |
| 6 | 188.98 | 129.65 |
| 7 | 191.80 | 130.66 |
| 8 | 194.62 | 131.70 |
| 9 | 197.42 | 132.77 |
| 10 | 200.21 | 133.87 |
| 11 | 202.99 | 135.00 |
| 12 | 205.76 | 136.16 |
| 13 | 208.51 | 137.35 |
| 14 | 211.25 | 138.57 |
| 15 | 213.98 | 139.81 |
| 16 | 216.69 | 141.09 |
| 17 | 219.39 | 142.40 |
| 18 | 222.08 | 143.74 |
| 19 | 224.10 | 144.77 |




```
**** Simplified BISHOP FOS = 1.509 ****
```

The following is a summary of the TEN most critical surfaces

Problem Description : Bench on 3 to 1 slope Rotational

| | FOS (BISHOP) | Circle x-coord (ft) | Center y-coord (ft) | Radius (ft) | Initial x-coord (ft) | Terminal x-coord (ft) | Resisting Moment (ft-lb) |
|---|---|---|---|---|---|---|---|
| 1. | 1.509 | 97.54 | 390.48 | 276.38 | 174.69 | 224.10 | 8.729E+05 |
| 2. | 1.510 | 95.73 | 393.75 | 280.17 | 173.47 | 224.16 | 9.583E+05 |
| 3. | 1.510 | 118.26 | 341.12 | 222.78 | 178.78 | 222.26 | 5.755E+05 |
| 4. | 1.510 | 119.79 | 341.65 | 222.74 | 180.00 | 224.01 | 5.979E+05 |
| 5. | 1.510 | 95.05 | 401.95 | 287.88 | 176.73 | 224.32 | 8.436E+05 |
| 6. | 1.510 | 107.29 | 363.65 | 248.00 | 173.88 | 223.81 | 8.703E+05 |
| 7. | 1.511 | 115.46 | 346.84 | 229.29 | 176.73 | 223.44 | 7.129E+05 |
| 8. | 1.511 | 100.94 | 378.17 | 263.94 | 171.84 | 224.15 | 1.045E+06 |
| 9. | 1.511 | 91.97 | 407.48 | 294.28 | 174.69 | 224.41 | 9.819E+05 |
| 10. | 1.512 | 117.69 | 337.49 | 219.98 | 174.29 | 223.26 | 8.237E+05 |

* * * END OF FILE * * *

**Shallow Translational Failure with Tack-on Benches - Example Computer Output**

XSTABL File: BEN3RSLT 6-01-** 11:48

```
                X S T A B L

       Slope Stability Analysis
              using the
          Method of Slices

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              Interactive Software
        Designs, Inc.
       Moscow, ID 83843, U.S.A.

          All Rights Reserved

         Ver. 5.20296 ) 1697
```
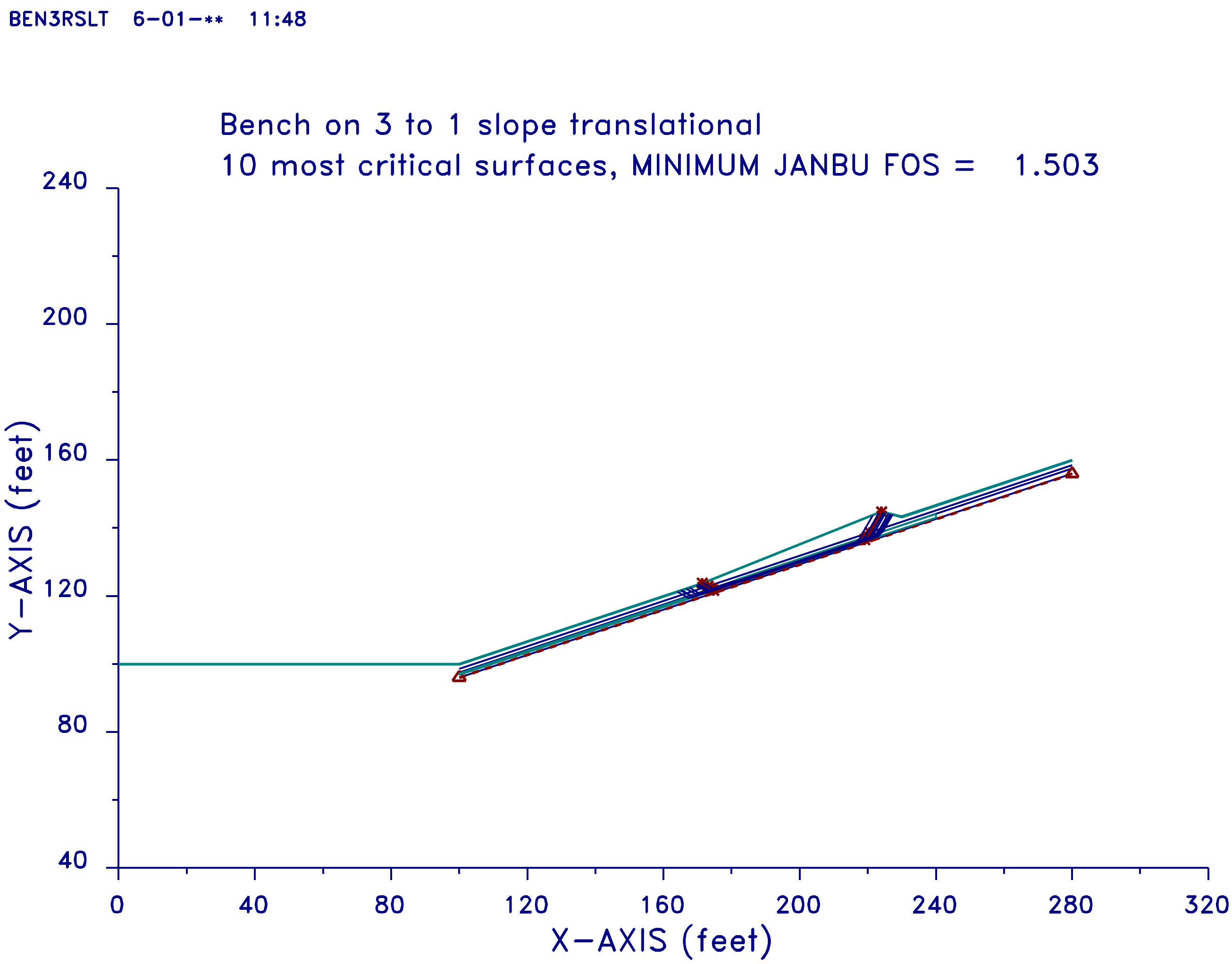



Problem Description : Bench on 3 to 1 slope translational

## 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 | 100.0 | 100.0 | 100.0 | 1 |
| 2 | 100.0 | 100.0 | 171.5 | 123.8 | 1 |
| 3 | 171.5 | 123.8 | 224.0 | 144.8 | 1 |
| 4 | 224.0 | 144.8 | 230.0 | 143.3 | 1 |
| 5 | 230.0 | 143.3 | 280.0 | 160.0 | 1 |

3 SUBSURFACE boundary segments

| Segment No. | x-left (ft) | y-left (ft) | x-right (ft) | y-right (ft) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | 100.0 | 98.5 | 280.0 | 158.5 | 2 |
| 2 | 100.0 | 97.5 | 280.0 | 157.5 | 3 |
| 3 | 100.0 | 96.0 | 280.0 | 156.0 | 4 |

The geosynthetic interfaces have been modeled as a 1.5-foot thick layer (highlighted), using a compound nonlinear shear strength envelope, so it is easier to force the failure surfaces through the geosynthetic. RSL was not modeled since the failure surface was not allowed below geosynthetic in the analysis.

## ISOTROPIC Soil Parameters

4 Soil unit(s) specified

| Soil Unit No. | Unit Weight Moist (pcf) | Unit Weight Sat. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Pore Pressure Parameter Ru | Pore Pressure Constant (psf) | Water Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 120.0 | 125.0 | .0 | 31.00 | .000 | .0 | 0 |
| 2 | 125.0 | 130.0 | .0 | 31.00 | .000 | .0 | 0 |
| 3 | 100.0 | 100.0 | .0 | .00 | .000 | .0 | 0 |
| 4 | 70.0 | 70.0 | 480.0 | 33.00 | .000 | .0 | 0 |

NON-LINEAR MOHR-COULOMB envelope has been specified for 1 soil(s)

Soil Unit # 3

| Point No. | Normal Stress (psf) | Shear Stress (psf) |
|---|---|---|
| 1 | .0 | .0 |
| 2 | 288.0 | 215.0 |
| 3 | 576.0 | 275.0 |
| 4 | 1440.0 | 350.0 |

The normal stresses chosen for soil unit #3 bracket the normal stresses expected after construction of the bench. The shear stresses are the minimum shear strengths the materials in the cap system will need to exhibit during conformance testing prior to construction.

A critical failure surface searching method, using a random technique for generating sliding BLOCK surfaces, has been specified.

The active and passive portions of the sliding surfaces are generated according to the Rankine theory.

2500 trial surfaces will be generated and analyzed.

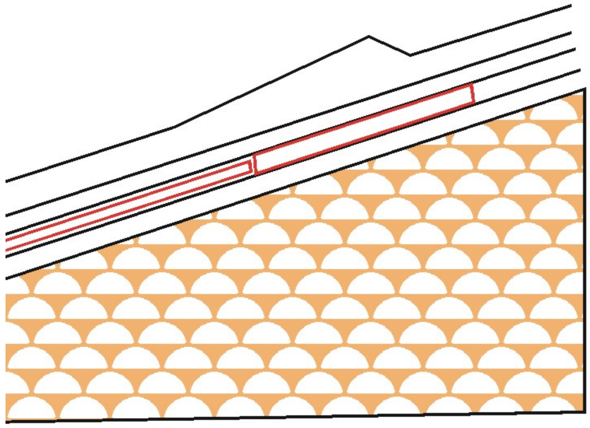
2 boxes specified for generation of central block base

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

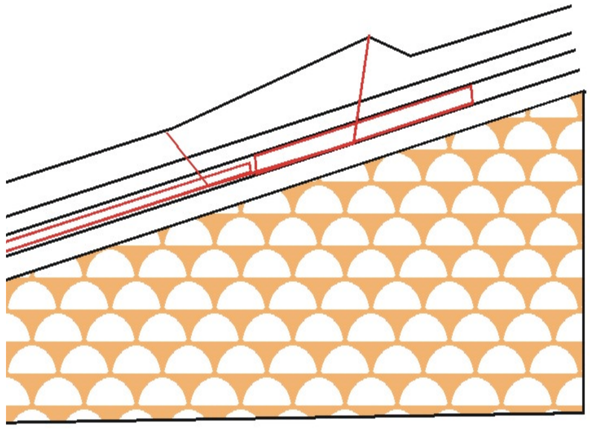
Length of line segments for active and passive portions of sliding block is 13.0 ft

| Box no. | x-left (ft) | y-left (ft) | x-right (ft) | y-right (ft) | Width (ft) |
|---|---|---|---|---|---|
| 1 | 102.0 | 97.5 | 175.0 | 122.0 | .5 |
| 2 | 181.3 | 124.3 | 240.0 | 143.6 | 1.0 |

Search boxes have been chosen so the randomly generated failure surfaces remain mostly within the layer representing the geosynthetic.



```
Factors of safety have been calculated by the :

* * * * *  SIMPLIFIED JANBU METHOD  * * * * *

The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by 8 coordinate points
```



| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 171.42 | 123.77 |
| 2 | 173.05 | 122.85 |
| 3 | 174.16 | 122.22 |
| 4 | 174.71 | 121.67 |
| 5 | 219.10 | 136.64 |
| 6 | 219.93 | 137.48 |
| 7 | 220.63 | 138.71 |
| 8 | 224.07 | 171.42 |

```
 ** Corrected JANBU FOS =  1.503 ** (Fo factor = 1.043)

Failure surface No. 2 specified by 8 coordinate points
```

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 167.75 | 122.55 |
| 2 | 169.39 | 121.63 |
| 3 | 170.50 | 121.00 |
| 4 | 170.90 | 120.59 |
| 5 | 221.58 | 137.30 |
| 6 | 222.67 | 138.39 |
| 7 | 223.37 | 139.62 |
| 8 | 226.01 | 144.30 |

```
 ** Corrected JANBU FOS =  1.511 ** (Fo factor = 1.041)

Failure surface No. 3 specified by 8 coordinate points
```

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 169.72 | 123.21 |
| 2 | 171.35 | 122.28 |
| 3 | 172.47 | 121.66 |
| 4 | 173.01 | 121.12 |
| 5 | 216.09 | 135.34 |
| 6 | 217.38 | 136.63 |

```
   7    218.08  137.86
   8    221.42  143.77

** Corrected JANBU FOS =  1.511 ** (Fo factor = 1.045)

 Failure surface No. 4 specified by 8 coordinate points

 Point  x-surf  y-surf
   No.    (ft)    (ft)
     1  170.48  123.46
     2  172.11  122.54
     3  173.22  121.91
     4  173.61  121.52
     5  219.95  136.95
     6  220.75  137.75
     7  221.45  138.98
     8  224.65  144.64

  ** Corrected JANBU FOS =  1.511 ** (Fo factor = 1.036)

 Failure surface No. 5 specified by 8 coordinate points

 Point  x-surf  y-surf
  No.    (ft)    (ft)
   1    166.80  122.24
   2    168.44  121.31
   3    169.55  120.68
   4    169.84  120.39
   5    221.92  137.18
   6    223.37  138.62
   7    224.07  139.86
   8    226.51  144.17

** Corrected JANBU FOS =  1.515 ** (Fo factor = 1.035)

 Failure surface No. 6 specified by 8 coordinate points

 Point  x-surf  y-surf
  No.    (ft)    (ft)
   1    165.49  121.80
   2    167.13  120.88
   3    168.24  120.25
   4    168.74  119.74
   5    222.78  137.49
   6    224.19  138.90
```

| | | |
|---|---|---|
| 7 | 224.88 | 140.13 |
| 8 | 227.09 | 144.03 |

** Corrected JANBU FOS = 1.516 ** (Fo factor = 1.032)

Failure surface No. 7 specified by 8 coordinate points

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 170.32 | 123.41 |
| 2 | 171.95 | 122.48 |
| 3 | 173.06 | 121.85 |
| 4 | 173.26 | 121.66 |
| 5 | 217.82 | 135.95 |
| 6 | 219.06 | 137.19 |
| 7 | 219.75 | 138.42 |
| 8 | 223.18 | 144.47 |

** Corrected JANBU FOS = 1.520 ** (Fo factor = 1.044)

Failure surface No. 8 specified by 8 coordinate points

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 164.30 | 121.40 |
| 2 | 165.93 | 120.48 |
| 3 | 167.04 | 119.85 |
| 4 | 167.51 | 119.38 |
| 5 | 220.37 | 136.89 |
| 6 | 221.47 | 137.99 |
| 7 | 222.16 | 139.22 |
| 8 | 225.16 | 144.51 |

** Corrected JANBU FOS = 1.522 ** (Fo factor = 1.037)

Failure surface No. 9 specified by 8 coordinate points

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 165.26 | 121.72 |
| 2 | 166.89 | 120.80 |
| 3 | 168.01 | 120.17 |
| 4 | 168.47 | 119.71 |
| 5 | 222.99 | 137.56 |
| 6 | 224.39 | 138.96 |

| | | |
|---|---|---|
| 7 | 225.09 | 140.20 |
| 8 | 227.23 | 143.99 |

** Corrected JANBU FOS =  1.523 ** (Fo factor = 1.034)

Failure surface No.10 specified by 8 coordinate points

| Point No. | x-surf (ft) | y-surf (ft) |
|---|---|---|
| 1 | 171.68 | 123.87 |
| 2 | 173.32 | 122.94 |
| 3 | 174.44 | 122.31 |
| 4 | 174.81 | 121.94 |
| 5 | 222.87 | 137.98 |
| 6 | 223.59 | 138.70 |
| 7 | 224.29 | 139.93 |
| 8 | 226.67 | 144.13 |

** Corrected JANBU FOS =  1.524 ** (Fo factor = 1.032)

The following is a summary of the TEN most critical surfaces

Problem Description : Bench on 3 to 1 slope translational

| | Modified JANBU FOS | Correction Factor | Initial x-coord (ft) | Terminal x-coord (ft) | Available Strength (lb) |
|---|---|---|---|---|---|
| 1. | 1.503 | 1.043 | 171.42 | 224.07 | 1.414E+04 |
| 2. | 1.511 | 1.036 | 167.75 | 226.01 | 1.571E+04 |
| 3. | 1.511 | 1.045 | 169.72 | 221.42 | 1.381E+04 |
| 4. | 1.511 | 1.041 | 170.48 | 224.65 | 1.445E+04 |
| 5. | 1.515 | 1.035 | 166.80 | 226.51 | 1.613E+04 |
| 6. | 1.516 | 1.032 | 165.49 | 227.09 | 1.677E+04 |
| 7. | 1.520 | 1.045 | 170.32 | 223.18 | 1.394E+04 |
| 8. | 1.522 | 1.037 | 164.30 | 225.16 | 1.628E+04 |
| 9. | 1.523 | 1.032 | 165.26 | 227.23 | 1.682E+04 |
| 10. | 1.524 | 1.034 | 171.68 | 226.67 | 1.485E+04 |

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## REFERENCES

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.

Fox, P. J., Rowland, M. G., and Scheithe, J. R. (1998). “Internal shear strength of three geosynthetic clay liners,” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, No. 10, pp. 933-944.

Fox, P. J., Stark, T. D., and Swan, Jr. R. H. (2004). “Laboratory measurement of GCL shear strength,” Advances in Geosynthetic Clay Liner Technology: 2nd Symposium, STP 1456, Mackey, R. E. and von Maubeuge, K., eds., ASTM International, West Conshohocken, Pennsylvania, U.S.A, in press.

Gilbert, R. B., Scranton, H. B., and Daniel, D. E. (1997). “Shear strength testing for geosynthetic clay liners,” *Testing and Acceptance Criteria for Geosynthetic Clay Liners*, STP 1308, L. W. Well, ed., ASTM International, West Conshohocken, Pennsylvania, U.S.A, pp. 121-135.

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.

Giroud, J. P., Bachus, R. C. and Bonaparte, R., 1995, “Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes,” Geosynthetics International, Vol. 2, No. 6, pp. 1149 - 1180.

Giroud, J. P., Zhao, A., and Richardson, G. N., 2000, “Effect of Thickness Reduction on Geosynthetic Hydraulic Transmissivity,” Geosynthetic International, Vol. 7, Nos. 4 - 5.

Giroud, J. P., Zornberg, J. G., and Zhao, A., 2000, “Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers,” Geosynthetic International, Vol. 7, Nos. 4 - 5.

Huff, F. A., and Angle, J. R., 1992, “Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71.

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

Koerner, R. M., and Daniel, D. E., 1997, “Final Covers for Solid Waste Landfills and Abandoned Dumps,” ASCE Press, New York.

Matasovic, N., 1991, “Selection of Method for Seismic Slope Stability Analysis,” Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, St. Louis, Missouri, Paper 7.20, pp. 1057 - 1062.

Richardson, G. N., and Zhao, A., 1999, Design of Lateral Drainage Systems for Landfills.

Singh, S. and Sun, J. I., 1995, "Seismic Evaluation of Municipal Solid Waste Landfills," Proc. Geoenvironment 2000, ASCE Speciality Conference, New Orleans, Louisiana, February 22 - 24. pp. 1081 - 1096.

Soong, T. Y. and Koerner, R. M., 1997, “The Design of Drainage Systems over Geosynthetically Lined Slopes,” GRI Report #19.

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 Engrg., 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.

Thiel, R .S., and Stewart, M. G., 1993, “Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest,” Geosynthetics ‘93 Conference Proceedings, IFAI, St. Paul, Minnesota, pp. 1131 - 1144.

Triplett, E. J. and Fox, P. J. (2001). “Shear strength of HDPE geomembrane/geosynthetic clay liner interfaces,” Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 6, pp. 543-552.

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