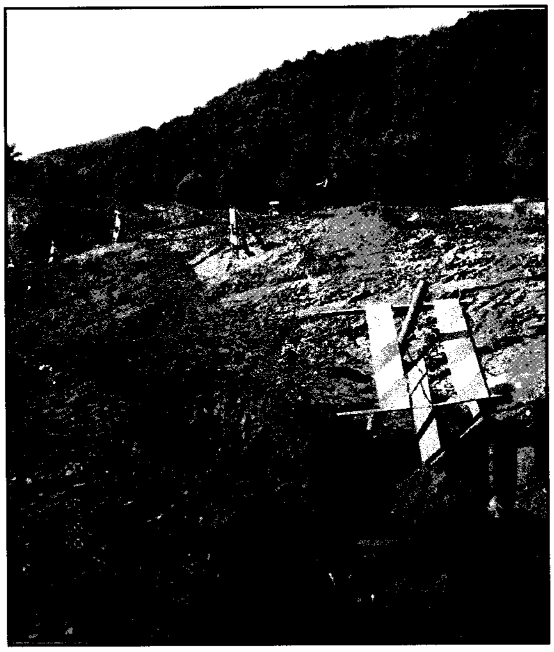
OhioEPA

State of Ohio Environmental Protection Agency

Geotechnical Resource Group (GeoRG)

Geotechnical and Stability Analyses for Ohio Waste Containment Facilities



September 14, 2004

THIS POLICY DOES NOT HAVE THE FORCE OF LAW.

Cover photos:
(Left) Division of Emergency and Remedial Response - CERCLA closure site failure.
(Top Right) Division of Solid and Infectious Waste Management - commercial municipal solid waste landfill failure.
(Bottom Right) Division of Surface Water - captive waste water ash impoundment berm failure.

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FOREWORD

READ THIS FIRST

This policy is designed to assist owners of regulated waste containment facilities in demonstrating that stability requirements set forth in Ohio EPA's rules have been satisfied. The information provided in this policy may be applied to all plans, applications, or requests submitted to any division of Ohio EPA for approval, concurrence, or comment. This policy is particularly applicable to waste containment facility designs that include natural or engineered components where the movement of soil, rock, waste, geosynthetics, or other materials may occur because of gravitational influence.

The information in this policy will be useful to anyone proposing any excavating, stockpiling, filling, or construction activity that is at, or close to, an Ohio EPA regulated waste containment facility.

The information contained herein is intended to apply to design, requests for authorization, construction, and closure of waste containment facilities to assist facilities in satisfying Ohio EPA's rule requirements for demonstrating stability. However, the applicable statutes and rules should also be consulted directly, as this policy is intended to ensure the activities undertaken to demonstrate stability satisfy the requirements of the appropriate statutes and rules. In addition, individual site-specific circumstances may exist that affect the stability analyses for any given facility, thereby requiring alternatives to the procedures and methods included in this policy to be used by the responsible party.

This policy recommends specific items be included in geotechnical and stability analyses and includes definitive performance criteria established by rule to use for documenting stability to Ohio EPA. This policy addresses when stability analyses are needed, the content of geotechnical and stability analyses reporting documents, subsurface investigation, materials testing, static and seismic stability analyses, and certain other geotechnical analyses.

Any examples or case studies referred to in this policy are intended to demonstrate how compliance may be achieved, but are not intended to establish a requirement for how the applicable statutes or rules must be satisfied. The methods and procedures included in this policy have been evaluated by Ohio EPA and have been shown to be useful for demonstrating that a waste containment facility will meet the rule requirements for stability. Alternative methods or procedures may be used if they are fully documented as being valid and appropriate for demonstrating compliance with stability requirements in rule and are acceptable to Ohio EPA.

THE USE OF REQUIREMENTS VS. RECOMMENDATIONS

This policy describes requirements when:

- a specific or general Ohio statute or rule exists that includes the requirement,
- published standards, such as American Society for Testing and Materials (ASTM) methods, contain the requirement, or
- the assumptions of a theoretical model or method being used for analysis and/or calculations require it for the analysis or calculations to be valid and applicable.

Requirements are notated in this manual with language such as "shall," "must," or "required."

This policy describes recommendations when:

- none of the above criteria apply,
- published standards or state of the practice offer multiple acceptable alternatives, or
- the state of the practice is not sufficiently developed to provide a definitive selection of a best practice. When this occurs, the manual reflects the best understanding of a current approach that seems appropriate for use in Ohio.

Recommendations are notated in this manual with such language as "should," "may," or " recommends."

Responsible parties are obligated to comply with rule requirements even if the same activities are included in this policy as recommendations.

DEFINITIONS AND ACRONYMS

Throughout this policy the defined words and phrases are italicized to remind the reader that the terms are defined. Although not necessarily defined in Ohio's regulations, the following definitions are useful for understanding this policy.

AASHTO	American Association of State Highway and Transportation Officials.
ASCE	American Society of Civil Engineers.
ASTM	American Society for Testing and Materials.
Bedrock	Solid rock underlying *unconsolidated materials* (*soil units*). Syn: *consolidated stratigraphic unit.*
Book values	Values derived from charts, tables, or other generalized presentations of data found in textbooks, periodicals, and manuals. Book values often represent broad generalities derived from data that are unlikely to accurately portray localized site-specific conditions, but may be useful when used in a very conservative manner and in accordance with proper assumptions. For example, using book values to estimate the sheer strength of competent bedrock is likely to be appropriate.
Borings	Any means of mechanical penetration into the subsurface for the purposes of characterizing material properties or collecting material *samples*.
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
Compressible layer	Soil or filled materials that may settle after establishing a facility, and may continue to settle after a facility has closed.
Conformance testing	Testing conducted before construction on *samples* from materials that will be used during construction, the results of which are compared to the approved design specifications to ensure that the materials used in construction will perform as required. Syn: Preconstruction testing
Consolidated material	See: *Bedrock.*
Consolidated stratigraphic unit	See: *Bedrock.*
CPT	Cone Penetrometer Test.
Critical layer	A potentially liquefiable layer, or a thickness of soil or waste material that has a *drained* or *undrained shear strength* that may cause a failure if all or part of the mass of a facility were suddenly put in place. *Critical layers* may be only a few inches thick to tens of feet thick. *Critical layers* may include parts of one or more stratigraphic *soil units*.

CU	Consolidated-Undrained.
DERR	Division of Emergency and Remedial Response
Differential settlement	The difference in settlement across a relatively small area that may result in damage to engineered components due to increased stresses.
DL	Granular drainage layer.
Drained conditions	The state that exists when a soil layer cannot experience excess pore water pressure given the expected stress conditions. This may occur because the layer has a high enough hydraulic conductivity that pore water pressure dissipates quickly when loading occurs.
Drained shear strength	The shear strength exhibited by a soil layer when no excess pore water pressure is present. *Drained shear strength* is used for conducting an effective stress analysis.
EPA	Environmental Protection Agency.
Facility bottom	The base of a facility that is usually sloping five (5) percent or less so that water, leachate, and other liquids can drain from a facility. The term "*facility bottom*" excludes *internal slopes* or *interim slopes* (see **Figure f-1**). Interfaces on *facility bottoms* that have grades of 5 percent or less may be assigned *peak shear strength* during stability analyses, if appropriate.
Final slopes	Slopes that exist when the final grades for a facility have been achieved, including the cover system, if any (see **Figure f-1**). Interfaces on *final slopes* that will never be loaded with more than 1,440 pounds/ft^2 (psf) may be assigned *peak shear strength* during stability analyses, if appropriate.

Figure f-1 An example of a typical landfill progression showing *internal*, *interim*, and *final slopes*, and the *facility bottom*. These types of slopes may also be present at other types of *waste containment facilities*.

FML	Flexible membrane liner. Syn: Geomembrane
fps	Feet per second.
FS	Factor of safety.
GCL	Geosynthetic clay liner.
GDL	Geocomposite drainage layer.
HDPE	High density polyethylene.
Higher quality data	Data produced from laboratory methods or cone penetrometer tests (CPTs) that, when properly conducted, provide the most definitive measurements obtainable of the characteristics of a *specimen*.
Interim slopes	Slopes that exist at a *waste containment facility* because of daily filling or because a phase or unit has reached its limits, including cover soils. An *interim slope* will exist for only part of the facility life and is not part of the engineered components of the facility (see **Figure f-1** on page xii).
Internal slopes	Slopes excavated below grade and/or constructed using berms, including, as applicable, the liner/leachate collection system, *protective layers*, and other engineered components (see **Figure f-1** on page xii). Interfaces on *internal slopes* that exceed a grade of five (5) percent must be assigned *residual shear strength* during deep-seated failure analysis, but may be assigned *peak shear strength* during shallow failure analysis, if appropriate.
Lower quality data	Data produced by field testing (other than CPTs) that are good for relative comparison of characteristics, but even when the test is run properly, do not necessarily provide a definitive measurement of the characteristic. Examples of methods that produce *lower quality data* include, but are not limited to, blow counts and pocket penetrometers.
MSW	Municipal solid waste.
OCR	Overconsolidation ratio.
ODNR	Ohio Department of Natural Resources.
ODOT	Ohio Department of Transportation.
Overall settlement	The settlement of an entire *waste containment facility*, as it relates to facility geometry, appurtenances, pipes, roads, culverts, leachate drainage ways, and surface water drainage ways.
pcf	Pounds per cubic foot.

Peak shear strength The maximum shear stress recorded during a shear test as strain is increased (see **Figure f-2**) on page xiv.

Phreatic surface A surface that represents the water level in an unconfined *saturated* zone. Examples: *saturated* portions of soil or waste that are not confined by an overlying layer; the surface created by leachate on a landfill liner; the water level in a waste water lagoon; or the *saturated* portion of a clay soil layer, all create *phreatic surfaces*. Syn: Water table.

Piezometric surface A surface that represents the actual pressure head relative to a confined *saturated* zone. For example, the surface created by water level readings from wells screened in a *saturated* sand overlain by heavy clay such that the water level surface is measured above the top of the sand. Syn: Potentiometric surface.

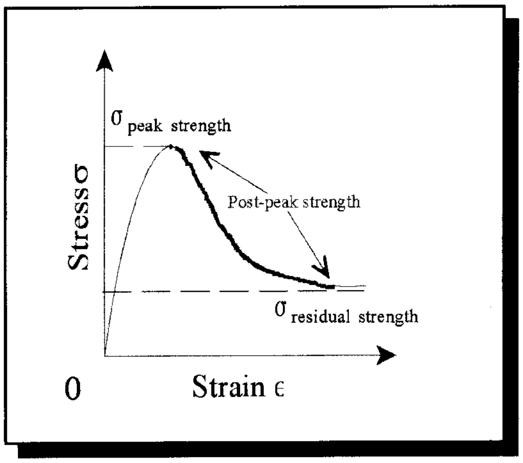


Figure f-2 Typical stress-strain response of a soil *specimen*. After: Bardet, 1997, Experimental Soil Mechanics. Figure 1. (b), pp. 362

Primary consolidation See: *Primary settlement.*

Primary settlement The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids (ASTM D 653). Syn: *Primary consolidation.*

Protective layer A layer made of soil or granular material designed to protect underlying geosynthetics and recompacted soil layers from damage due to construction, operations, maintenance, freezing, or weathering. Examples of *protective layers* include, but are not limited to, a granular leachate collection layer with underlying geotextile cushion layer, a soil layer placed on top of a drainage layer in a cap, or a granular material with an underlying geotextile cushion layer used to protect lagoon and pond liners.

psf Pounds per square foot.

QA/QC Quality assurance and quality control.

Residual shear strength The steady state shear stress recorded after the strain is increased beyond the *peak shear strength* of a *specimen* (see **Figure f-2** on page xiv). *Residual shear strength* is measured or can be conservatively estimated based on the results of applicable tests.

Responsible party — The persons in control of the property, facilities, and activities that occur at a *waste containment facility*, including, but not limited to, applicant, permittee, owner, operator, or potentially responsible party (PRP).

RSL — Recompacted soil layer (liner or barrier layer depending upon context).

Sample — **noun:** Used in this manual to describe a volume of material from which *specimens* are prepared for testing. One *sample* may provide one or more *specimens* for testing. **verb**: Used in this manual to refer to the activities necessary to collect *samples* of materials.

Saturated — **a:** for shallow failure analysis: the *protective layers* over a cap drainage layer or over a geocomposite leachate collection layer are at field capacity, and are discharging water to underlying drainage layers at a rate equal to the effective hydraulic conductivity of the *protective layer*. When a *protective layer* is a leachate collection layer prior to waste placement, then *saturated* means the state when head exists due to the occurrence of a design storm. **b:** for laboratory methods: a *specimen* has, to the extent possible, all voids full of water. **c:** for subsurface conditions, one or more *soil units*, or part of a *soil unit* has most of the voids filled with water.

Secondary compression — See: *Secondary settlement.*

Secondary settlement — The reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids (ASTM D 653). Syn: *secondary compression.*

Soil stratigraphy — The vertical and lateral or spatial arrangement of *soil units* at a facility.

Soil unit — **a:** A discrete layer or body of *unconsolidated material* that can be readily and consistently distinguished from adjacent materials based on one or more characteristics or features, usually composition (e.g., grain size distribution, mineralogy, or percent organic material); structure (e.g., layering, interbedding, or fracturing/jointing); and/or soil engineering (physical) properties (e.g., plasticity, bulk density, or permeability). Depending on facility conditions, designation of layers or bodies of minespoil or fill materials as *soil units* may be appropriate. Individual *soil units* might not be laterally continuous across a facility. **b:** a stratum of soil within the *soil stratigraphy* of the facility. Syn: *Unconsolidated stratigraphic unit.*

Specimen — A specific volume of material subjected to testing. For example, a volume of soil material trimmed out of a *sample* and placed into a triaxial compression apparatus to be tested for shear strength.

SPT — Standard Penetration Test.

Strain incompatibility — The condition that exists when the displacement necessary to mobilize the peak internal or peak interface shear strength is different for two or more materials that comprise a composite system, such as a berm and its foundation, or different layers of a composite liner system. If *strain incompatibility* is not taken into account, it may cause computer modeling software to overlook the critical failure surface.

Total settlement — The settlement at any given point caused by the sum of immediate, *primary*, and *secondary settlement*.

Unconsolidated — In geology, used to differentiate between *bedrock* (*consolidated material*) and other materials such as weathered *bedrock* and soils (*unconsolidated material*). This is different from the geotechnical terms of "*unconsolidated*," "normally consolidated," and "overconsolidated" used to describe the stress history of a soil material.

Unconsolidated Stratigraphic Unit — See: *Soil unit*.

Undrained conditions — The state that exists when a soil layer experiences excess pore water pressure. This occurs during loading of a *compressible layer* of *saturated* soil and may occur during loading of a *compressible layer* of partially *saturated* soil.

Undrained shear strength — The shear strength exhibited by a *saturated* soil when experiencing an increase in stress that causes excess pore water pressure to develop. *Undrained shear strength* is used for conducting a total stress analysis.

Unsaturated — **a:** As used in shallow failure analysis, it means that the *protective layer* over a cap drainage layer or a *protective layer* over a geocomposite leachate collection layer has not reached field capacity, and is not discharging sufficient water to the drainage layer to create head on the underlying layer. When the *protective layer* is the leachate collection layer, it means that no head exists within the collection layer. **b:** As used in discussing laboratory methods, it means that a *specimen* has a measurable amount of void space that is not filled with water. **c:** As used in the discussion of subsurface in situ conditions, it means that no portion of a *soil unit* has most of the voids filled with water.

USACOE — United States Army Corps of Engineers.

USDA — United States Department of Agriculture.

USGS — United States Geological Survey.

USCS — Unified Soil Classification System.

UU — Unconsolidated-undrained.

Waste containment facility	One or more tracts of land that contain one or more *waste containment units*. This includes, but is not limited to, facilities regulated by Ohio EPA under the authority of Ohio Revised Code Chapters 3734, 6111, and 3714, and Federal Regulations, such as RCRA and CERCLA.
Waste containment system	One or more engineered components used singly or in aggregate to control waste that has been placed onto or into the ground.
Waste containment unit	A group of *waste containment systems* or a discrete area within a facility used for storage, treatment, or disposal of wastes, such as waste piles, landfills, surface impoundments, and closure units.

References

Bardet, J., 1997, Experimental Soil Mechanics. Prentice-Hall, New Jersey.

Holtz, R. D., and Kovacs, W. D., 1981, An Introduction to Geotechnical Engineering, Prentice Hall, Englewood Cliffs, New Jersey.

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.

BASIC CONCEPTS OF SLOPE STABILITY

Slope and foundation materials can move due to shearing stresses created within a material or at material interfaces by external forces (e.g., gravity, water flow, tectonic stresses, seismic activity). This tendency is resisted by the shear strength of the materials and interfaces and is expressed by the Mohr-Coulomb theory as:

$$\tau_f = c + \sigma \cdot \tan\phi$$ (see **Figure f-3** on page xx)

where τ_f = shear strength of material,
c = cohesion strength of material,
σ = normal stress applied to material, and
ϕ = friction angle.

In terms of effective stress (*drained condition*):

$$\tau_f' = c' + (\sigma - u)\tan\phi'$$

where τ_f' = shear strength of material,
c' = effective cohesion strength of material,
σ = normal stress applied to material,
u = pore water pressure, and
ϕ= = friction angle in terms of effective stress.

The relationship between the angle of failure and the internal angle of friction can be described as:

$$\alpha = 45^\circ + \frac{\phi}{2}$$ (see **Figure f-3** on page xx)

where, α = angle of failure in the material, and
ϕ = friction angle.

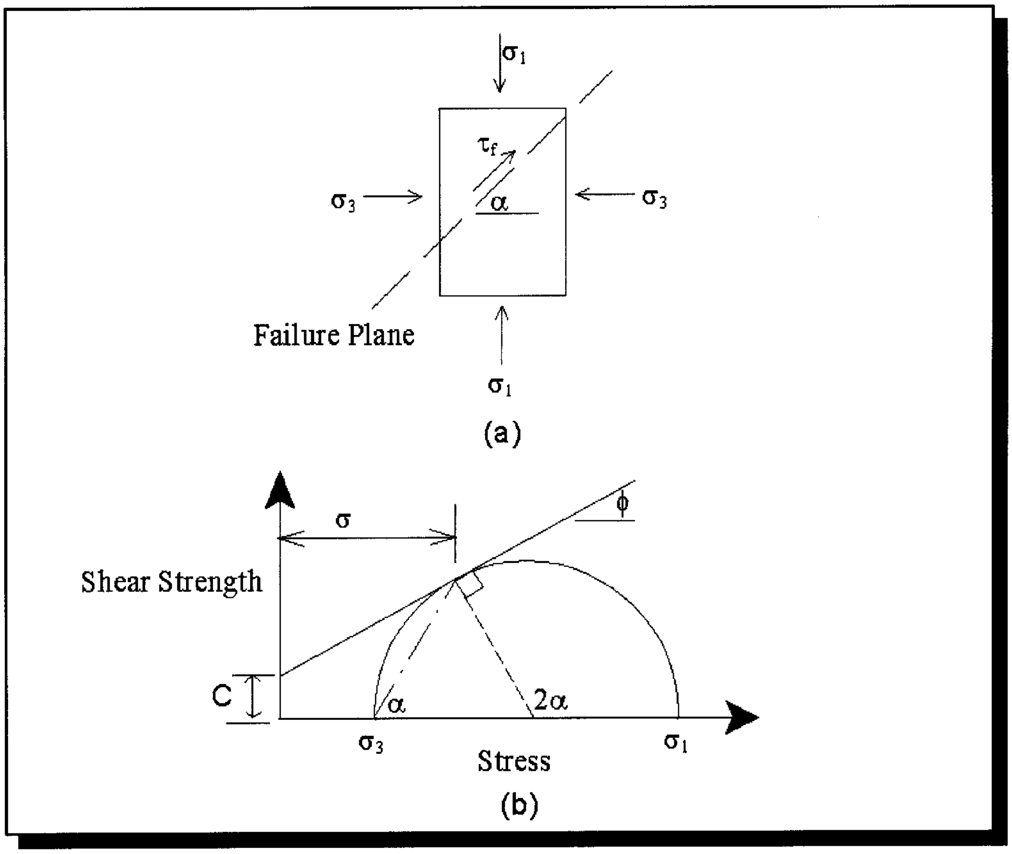


Figure f-3 Mohr-Coulomb envelope. (a) Soil element. (b) Shear strength envelope. Adapted from Abramson, et al, 1996, Slope Stability and Stabilization Methods, Figure 1.20, pg 37, and Holtz and Kovacs, 1981, An Introduction to Geotechnical Engineering, Figure 10.7a.

Symbols for Figure f-3.

τ_f = shear strength of material,
c = cohesion strength of material,
σ = stress applied to material,
σ_1 = major principal stress,
σ_3 = minor principal stress,
ϕ = friction angle, and
α = angle of failure in the material.

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.

Holtz, R. D., and Kovacs, W. D., 1981, An Introduction to Geotechnical Engineering, Prentice Hall, Englewood Cliffs, New Jersey.

CHAPTER 1

INTRODUCTION

Figure 1-1 An Ohio landfill near Cincinnati experienced a massive slope failure in 1996 that resulted in 18 fires during the 9 months it took to cover the exposed waste.

Stability failures at *waste containment facilities* are associated with many risks. These include risks to human health, the environment, communities, governments, and *responsible parties*. Risks to human health include the possibility of injury or death to individuals and disease from exposed waste. Many risks to the environment exist from stability failures. Ground water contamination can occur from ruptured lining systems or infiltration through an impaired cover system. Surface water contamination and flooding can occur from waste, wastewater, or engineered components that slide into rivers, creeks, and lakes; and from contaminated runoff from exposed waste due to a damaged cover system. Air contamination can occur from fires that ignite exposed waste or gases released during stability failures. Waste collection, treatment, and disposal may be interrupted for communities or for the *responsible party* (for a captive facility) serviced by a particular *waste containment facility*.

Stability failures can present large unanticipated costs to federal, state, and local governments for oversight of mitigation and remediation efforts. *Responsible parties* may accrue liabilities that include financial and legal responsibility for injuries, damages, lost income, redesign, agency re-approval, repair, and extended monitoring and maintenance.

> Stability failures are not necessarily large mass movements of materials. Damaging stability failures can be slight movements of a waste mass or cover system that may not be detectable through casual observation.
>
> In 1996, at an Ohio landfill near Youngstown, approximately 300,000 cubic yards of waste shifted and destroyed several acres of the composite liner system. The only indications that a slope failure occurred were the appearance of cracks in the daily cover soils and a slight heave near the toe of the slope (Stark et al, 1998).

The complexities involved in estimating the stability of a modern *waste containment facility* cannot be overstated. These projects are often massive structures that heavily affect the structural integrity of the in situ soils, support structures, and geosynthetics. Often, the largest variables to contend with are the interactions that occur between the individual components of a *waste containment system*. Interactions between these materials occur during the construction, filling, and any settlement or deformation of the facility, and are difficult to predict with a high degree of accuracy. Because of this, site-specific, *higher quality data*, state of the practice analysis, and factors of safety are employed to ensure that *waste containment facilities* will be stable when they are constructed.

FACTORS CONTRIBUTING TO STABILITY FAILURES

Stability failures are often caused by processes that increase the applied shear stress or decrease the shear resistance of a soil mass, an interface between two geosynthetics, or an interface between a geosynthetic and soil (see Table 1 on page 1-3). Engineering design attempts to identify any vulnerable materials or configurations so that *waste containment facilities* can be designed to account for natural forces such as gravity, water flow, and biodegradation. Even so, construction and operational activities trigger most slope failures at *waste containment facilities*. These activities are often planned or performed independently of the design process and subsequently cause circumstances that were unforseen during the design of the facility. Examples of these activities include, but are not limited to:

- placement of soil or waste from the top of a slope downward,
- lengthy or unplanned excavations,
- regrading of waste for operational or closure purposes,
- leachate recirculation,
- overfilling,
- blasting,
- stockpiling materials,
- waste relocation,
- relocation of access roads,
- suddenly increasing or reducing the freeboard in lagoons, and
- inadequate base liner length on the *facility bottom* to resist driving forces caused by the waste on the associated *internal slope.*

The numerous failures that have occurred due to these activities underscore the need for ongoing coordination and involvement between the persons involved in design, construction, and operations.

An example of an operational or construction activity that may affect the stability of a *waste containment facility* is the necessity for providing ample tie-in distance beyond the previously constructed portion of the facility. This is so that no excavation of previously placed waste, cover soils, or berms will be needed in order to expose the engineered components from the previous construction. This is important for stability purposes because removing waste or soil from the tie-in area may decrease the resisting force for that portion of the facility and trigger a stability failure, especially if the tie-in is at the toe of a slope.

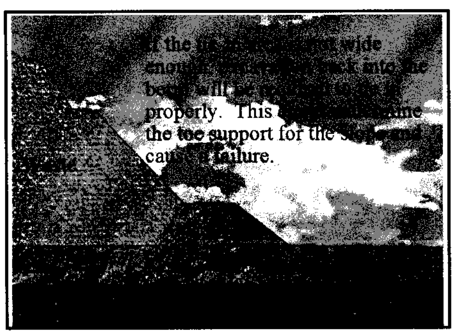


Table 1. Some factors that can adversely affect stability of *waste containment facilities*.

TYPES	COMMON CAUSES
Removal of toe support	
Natural causes	Erosion due to flow of ditches, streams, and rivers; wave action or lake currents; successive wetting and drying.
	Natural movement due to gravity such as falls, slides, and settlements away from toe; reduction in water levels after flooding.
Human activity	Cuts and excavations; removal of retaining walls or sheet piles; draw-down or filling of bodies of water (e.g., ponds, lagoons); excavation of waste; quarrying; borrowing soil.
Removal of underlying materials that provide support	
Natural causes	Weathering; underground erosion due to seepage (piping); solution of foundation materials from groundwater.
Human activity	Excavating; mining.
Decreasing the shear resistance of materials	
Natural causes	Water infiltration into cracks, fissures, and interfaces of engineered components; freeze/thaw cycles; expansion of clays; hydrostatic uplift.
Human activity	Using different materials causing lower interface shear strengths; using different or inappropriate construction methods causing lower internal or interface shear strengths of installed materials.
Increasing shear stresses	
Natural causes	Weight of precipitation (e.g., rains, snow, ice); increase in water levels in lagoons and ponds due to flooding; earthquakes.
Human activity	Stockpiling or overfilling; equipment travel or staging; water leakage from culverts, water pipes, and sewers; constructing haul roads; regrading of waste; increasing water levels in lagoons and ponds; increasing the density or loading rate of waste; blasting; vibrations from long trains passing by a location.

WHEN GEOTECHNICAL AND STABILITY ANALYSES ARE NEEDED

The appropriateness of conducting geotechnical and stability analyses must be considered whenever a *responsible party* is applying to Ohio EPA for authorization to permit, establish, modify, alter, revise, or close any type of *waste containment facility*. Usually, geotechnical and stability analyses are required by rule for these types of projects. Geotechnical and stability analyses should also be considered whenever circumstances indicate that doing so is prudent. Examples of circumstances indicating the need for geotechnical and stability analyses to be conducted include, but are not limited to:

- The facility experiences an earthquake that has a horizontal ground acceleration that approaches or exceeds the acceleration used in the stability analyses.

- A *phreatic surface* exceeds the maximum level evaluated in the stability analyses. This applies to flood waters against exterior berms, increased water levels in lagoons and ponds, and excessive leachate head in landfills, among others.

- New information is discovered about the characteristics of the *soil units* or engineered components that indicates the data used in the stability analyses may be incorrect or unconservative.

- After a failure, slip, or slump occurs that affects any engineered component of the facility.

- It becomes apparent to the *responsible party* that the design in the authorizing document must be changed while construction is occurring.

When a facility has experienced a failure or an earthquake or flood that approaches or exceeds design assumptions, a forensic geotechnical investigation and subsequent stability analyses should be conducted. These activities are conducted to evaluate the effects, if any, that the occurrence had on the engineered components and the stability of the *waste containment facility*. The results of all geotechnical investigations, stability analyses, and forensic investigations must be promptly submitted to Ohio EPA for review.

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.

Stark, T. D., Arellano, D., Evans, W. D., Wilson, V., and Gonda, J., 1998, "Unreinforced Geosynthetic Clay Liner Case History," Geosynthetics International Journal, Industrial Fabrics Association International (IFAI), Vol. 5, No. 5, pp. 521-544.

CHAPTER 2

CONTENT OF GEOTECHNICAL AND STABILITY ANALYSES

This chapter summarizes the components that should be considered parts of the geotechnical and stability analyses of a *waste containment facility* in Ohio. This chapter also summarizes the minimum information that should be reported to Ohio EPA once the analyses are complete. The specific contents for any given geotechnical and stability analyses report may change depending upon the specific set of circumstances surrounding each individual facility.

REPORT CONTENT

More details regarding report content can be found in the reporting section of each chapter of this policy. All drawings and cross sections should be referenced to the facility coordinate system, and northing and easting lines should be shown. Using tabs and a clear organizational format for the data will make it easier to find information when needed.

> 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.

Subsurface Investigation

Ohio EPA recommends that the results of the subsurface investigation be included in their own section of the geotechnical and stability analyses report (see Chapter 3 for more details). At a minimum, the following information about the subsurface investigation should be reported to Ohio EPA:

- A summary narrative describing the rationale behind the site investigation, assumptions used, methodologies used, the identification of the *critical layers*, *compressible layers*, temporal high *phreatic surfaces*, and temporal high *piezometric surfaces*, why they were selected, and what characteristics they have,

- One or more tables summarizing all field test data and laboratory test data gathered from all *borings* conducted and *samples* collected at the facility. The tables should clearly identify the *sample* locations and *borings* associated with each test result, the units of measurement of the test results, and test results associated with the *critical layers* and the *compressible layers* to be used in geotechnical and stability analyses,

- One or more topographic maps that show and identify each *boring* location and *sample* collection point at the facility. The maps can be used to identify the cross sections provided in the report. They can also be used to show the lateral extent of each *critical layer* and each *compressible layer* that exists at the facility, the elevations of the temporal high *phreatic surfaces*, and the elevations of the temporal high *piezometric surfaces*. Plan view maps should show the limits of the *waste containment unit(s)*,

- Cross sections that clearly show the *soil stratigraphy*, temporal high *phreatic surfaces*, and temporal high *piezometric surfaces* at the facility, and the characteristics of each *soil unit*,

- The preliminary investigation results, including a discussion of the findings of the preliminary investigation, and the sources of information used,

- A description of the site characterization results stating the activities, methods, and findings,

- A description of the investigation of *critical layers*, *compressible layers*, *phreatic surfaces*, and *piezometric surfaces*, and

- Any figures, drawings, or references relied upon during the investigation marked to show how they relate to the facility.

Materials Testing

Ohio EPA recommends that the results of all materials testing completed during the design of the *waste containment facility* be included in the subsurface investigation report. The subsurface investigation report is described in Chapter 3. At a minimum, the following information about materials testing results should be reported to Ohio EPA whenever testing is conducted (see Chapter 4 for more details):

- A narrative and tabular summary of the scope, extent, and findings of the materials testing,

- A description of collection and transport procedures for *samples*,

- The test setup parameters and protocols for each test,

- The characterization of each *specimen* used in each test,

- The intermediate data created during each test,

- The results of each test, and

- Any figures, drawings, or references relied upon during the testing marked to show how they relate to the facility.

> The results of *conformance testing* of materials completed after the design work, but prior to use of the materials in construction must be reported to Ohio EPA in their own report prior to use of the materials. In addition to the reporting requirements listed in this chapter and Chapter 4, a comparison of conformance test results to the requirements contained in rule, the authorizing document, or the assumptions used in the geotechnical and stability analyses should be included.

Liquefaction Potential Evaluation and Analysis

Ohio EPA recommends the liquefaction evaluation and analysis results be included in their own section of the geotechnical and stability analyses report (see Chapter 5 for more details). At a minimum, the following information about the liquefaction evaluation and analysis should be reported to Ohio EPA:

- A summary discussion of the findings of the liquefaction evaluation and analysis,

- A detailed discussion of the liquefaction evaluation including:

  - Evaluation of the geologic age and origin, fines content, plasticity index, saturation, depth below ground surface, and soil penetration resistance of each of the *soil units* that comprise the *soil stratigraphy* of the *waste containment facility*,

  - The scope, extent, and findings of the subsurface investigation as they pertain to the liquefaction evaluation,

  - A narrative description of each potentially liquefiable layer, if any, at the facility, and

  - Any figures, drawings, or references relied upon during the evaluation marked to show how they relate to the facility.

- If the liquefaction evaluation identifies potentially liquefiable layers, then the following information should be included in the report:

  - A narrative and tabular summary of the results of the liquefaction analysis completed for each potentially liquefiable layer,

  - Plan views of the facility that include the northings and eastings, the lateral extent of the potentially liquefiable layers, and the limits of the *waste containment unit(s)*,

  - Cross sections of the facility stratigraphic *soil units* that fully depict the potentially liquefiable layers, the characteristics that identify them as such, and show the engineered components of the facility,

  - The scope, extent, and findings of the subsurface investigation as they pertain to potentially liquefiable layers,

  - A description of the methods used to calculate the factor of safety (FS) against liquefaction,

  - Liquefaction analysis input parameters and assumptions, including the rationale for their selection,

  - The actual calculations and/or computer output, and

  - Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

Settlement Analyses and Bearing Capacity

The results of the settlement analysis for the facility, and the results of the bearing capacity analysis for vertical sump risers, if any are used, should be included in their own section of the geotechnical and stability analyses report (see Chapter 6 for more details). At a minimum, the following information about the bearing capacity analysis for vertical sump risers, if any are used, and the settlement analysis should be reported to Ohio EPA:

- A narrative and tabular summary of the results of the settlement analyses,
- A summary and a detailed discussion of the results of the subsurface investigation that apply to the settlement analyses and how they are used in the analyses,
- A summary of the approach, methodologies, and equations used to model settlement of the facility,
- If any of the settlement parameters were interpolated by using random generation or another method, then information must be provided that explains in detail, including equations and methodology, how the settlement parameters were generated,
- Plan view maps showing the top of the liner system, the liquid containment and collection system, the location of the points where settlement is calculated, the expected settlement associated with each point, and the limits of the *waste containment unit(s).*
- Drawings showing the critical cross sections analyzed. The cross sections should include the:
    - *Soil stratigraphy,*
    - Temporal high *phreatic surfaces*,
    - The range of the tested settlement parameters of each layer,
    - Depth of excavation,
    - Location of engineered components of the facility that may be adversely affected by settlement,
    - The amount of settlement calculated at each point chosen along the cross section,
    - The detailed settlement calculations of the engineering components,
    - Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility, and
    - The detailed tensile strain analysis.

- If vertical sump risers are included in the facility design, then include:
  - A narrative and tabular summary of the results of the bearing capacity analysis,
  - A summary and a detailed discussion of the results of the subsurface investigation that apply to the bearing capacity and how they were used in the analyses,
  - A summary of the approach, methodologies, and equations used to model the bearing capacity of the facility.

> Ohio EPA discourages the use of vertical sump risers in solid waste and hazardous *waste containment units*. This is due to the inherent difficulties they present during filling operations and the potential they create for damaging liner systems.

Hydrostatic Uplift Analysis

Ohio EPA recommends the results of the hydrostatic uplift analysis be included in their own section of the geotechnical and stability analyses report (see Chapter 7 for more details). At a minimum, the following information about the hydrostatic uplift analysis should be reported to Ohio EPA:

- A narrative and tabular summary of the results of the hydrostatic uplift analysis,
- A summary and discussion of the results of the subsurface investigation that apply to hydrostatic uplift analysis and how they were used in the analysis,
- A summary of the worst-case scenarios used to analyze the hydrostatic uplift potential of the facility,
- Isopach maps comparing excavation and construction grades with temporal high *phreatic surfaces* and temporal high *piezometric surfaces* as applicable to the facility. These drawings should show the limits of the *waste containment unit(s)*,
- The cross sections that were analyzed showing the characteristics of the *soil stratigraphy*, temporal high *phreatic surfaces*, temporal high *piezometric surfaces*, excavation grades, and engineered components, as applicable,
- The detailed hydrostatic uplift calculations, and
- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

Deep-Seated Failure Analysis

Ohio EPA recommends the results of the deep-seated failure analysis be included in their own section of the geotechnical and stability analyses report (see Chapter 8 for more details). At a minimum, the following information about the deep-seated failure analysis should be reported to Ohio EPA:

- A narrative summary of the results of the deep-seated failure analysis,
- One or more tables summarizing the results of the deep-seated failure analysis on all the analyzed cross sections,
- One or more tables summarizing the internal and interface shear strengths used to model the various components of the *internal, interim,* and *final slopes,*
- Graphical representations of the failure envelopes of each interface, material, and composite system,
- The scope, extent, and findings of the subsurface investigation as they pertain to the analysis 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 logic used to search for failure surfaces,
- Plan views of the *internal, interim,* and *final slope* grading plans clearly showing the location of the analyzed cross sections, the northings and eastings, and the limits of the *waste containment unit(s),*
- The analyzed cross sections, showing the engineered components and the underlying *soil stratigraphy,* including the temporal high *phreatic surfaces* and the temporal high *piezometric surfaces,*
- Static stability calculations (both inputs and outputs) for *internal, interim,* and *final slopes,* assuming *drained conditions* in the *soil units* 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

> 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 *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.

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*, and if applicable, *undrained conditions*, beneath the facility,

- Any other calculations used to analyze the deep-seated translational and rotational failure mechanisms for the facility, and

- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

Shallow Failure Analysis

Ohio EPA recommends the results of the shallow failure analysis be included in their own section of the geotechnical and stability analyses report (see Chapter 9 for more details). At a minimum, the following information about the shallow failure analysis should be reported to Ohio EPA:

- A summary narrative 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*,

- Graphical portrayal of any non-linear failure envelopes being proposed for each interface and material,

- A narrative justifying the assumptions used in the calculations, including a discussion of the applicable data from the subsurface investigation,

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

- The worst-case cross sections showing the engineered components, underlying *soil units*, waste, and the temporal high *phreatic surfaces*, and the temporal high *piezometric surfaces*,

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

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

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

- Any other necessary calculations used to evaluate shallow translational and rotational failure mechanisms at the facility, and

- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

THE COMPONENTS OF GEOTECHNICAL AND STABILITY ANALYSES

The geotechnical analyses should include a subsurface investigation and evaluations of hydrostatic uplift, liquefaction, settlement, and bearing capacity. The stability analyses should include a static evaluation and a seismic evaluation for *internal*, *interim*, and *final slopes*, each for deep and shallow translational failure surfaces and deep and shallow rotational failure surfaces.

Several unique conditions should be evaluated for any given facility. Examples of these conditions include, but are not limited to:

- *drained conditions* (no excess pore water pressure exists in the soil),
- *undrained conditions* (excess pore water pressure exists in soil materials), and
- *saturated protective layers* causing head in the drainage layers during the design storm.

Figure 2-1 on page 2-9 and **Figure 2-2** on page 2-10 provide an overview of the components of stability analyses that should be completed for any given *waste containment facility*. **Figure 2-3** starting on page 2-11 is a flowchart of a complete geotechnical and stability analyses for a *waste containment facility*.

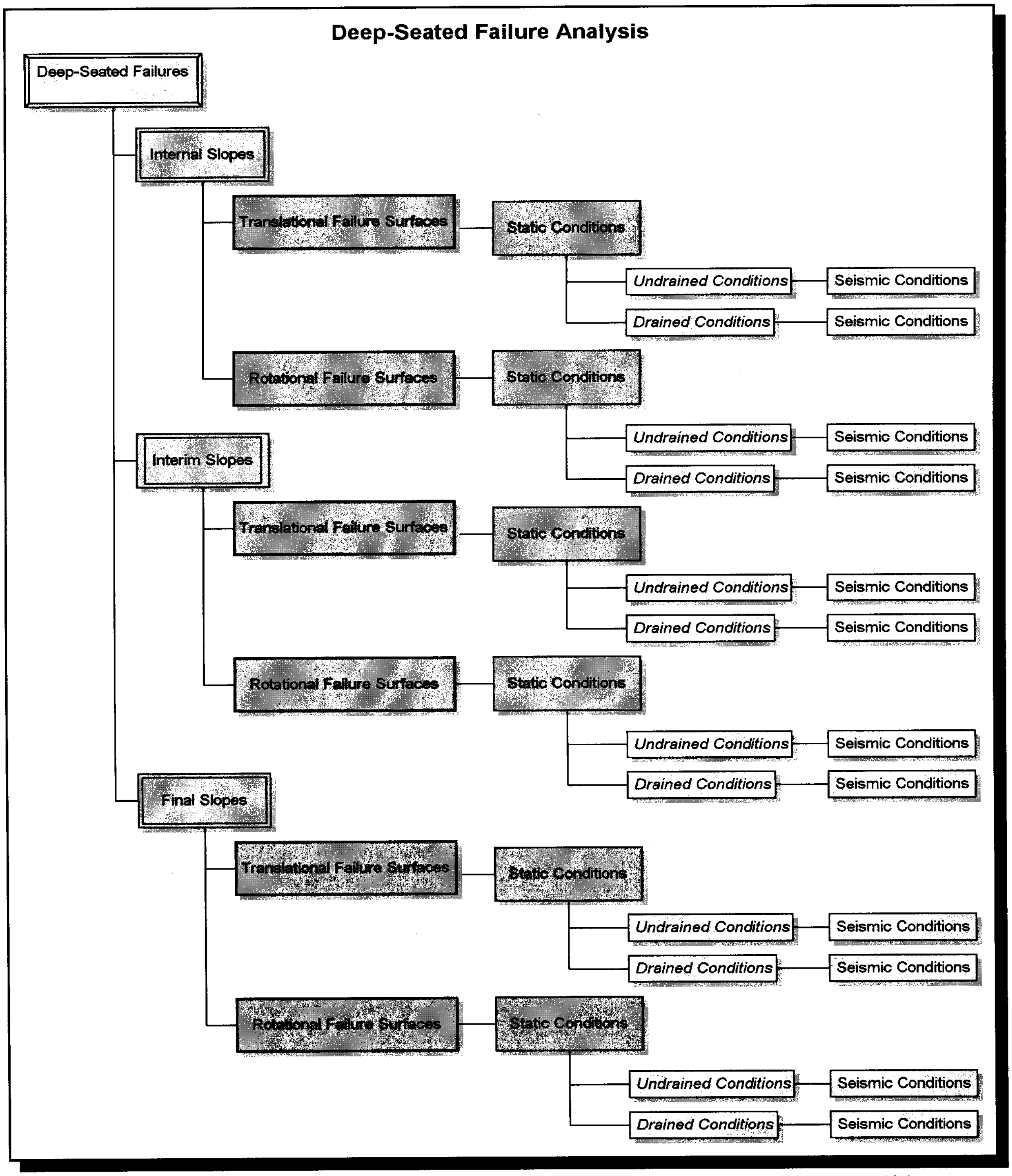


Figure 2-1 Organizational chart of the components of a deep-seated failure surface stability analysis. Note: If there are no *soil units* that may exhibit excess pore water pressure at a facility, then undrained analysis may not be required, and slope stability analysis of *internal slopes* and *interim slopes* under seismic conditions may not be necessary (see Chapter 8 for details).

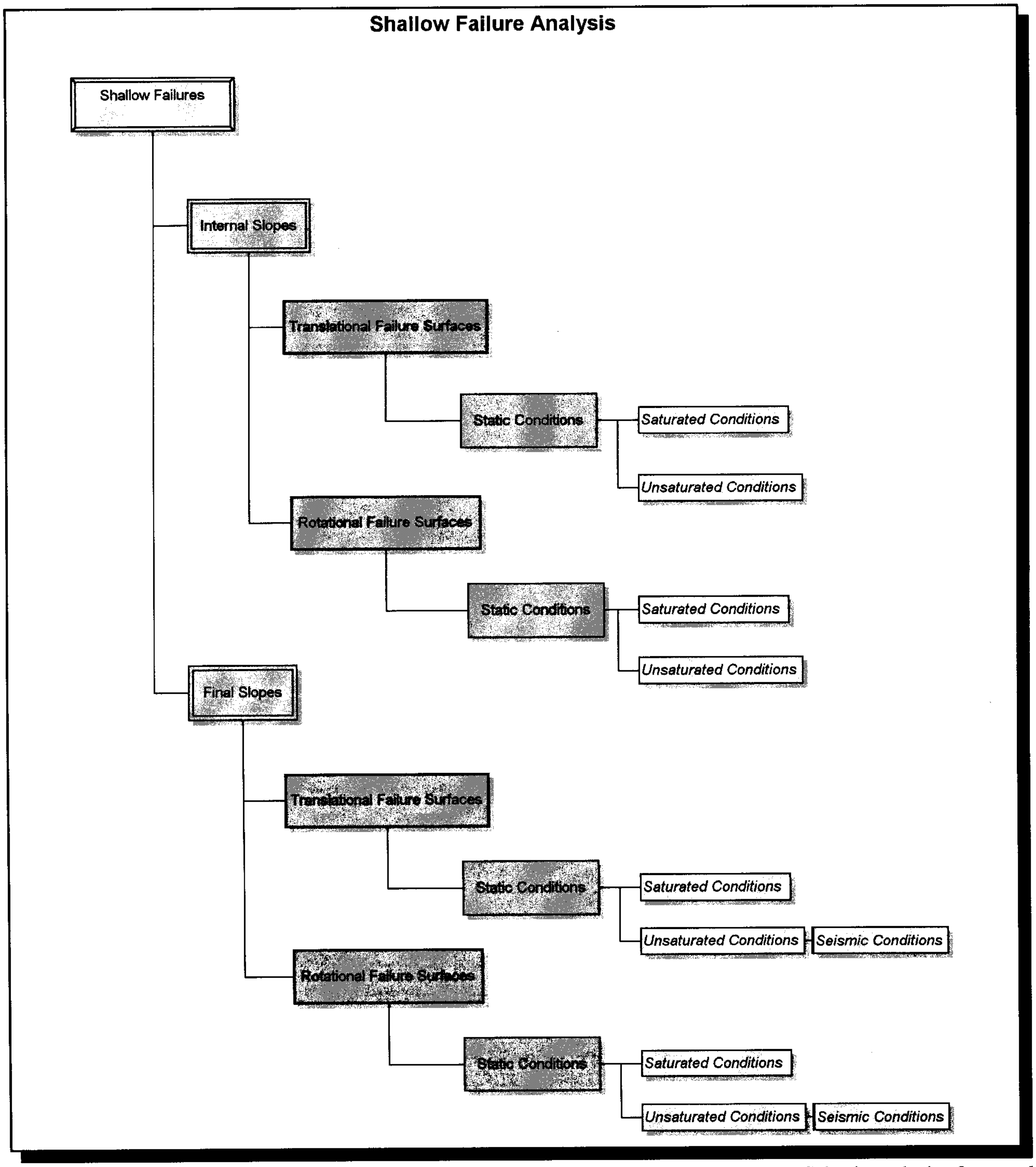


Figure 2-2 Organizational chart of the components of a shallow failure stability analysis. Note: Seismic analysis of *internal slopes* assuming *unsaturated* conditions may be required in some circumstances (see Chapter 9 for details).

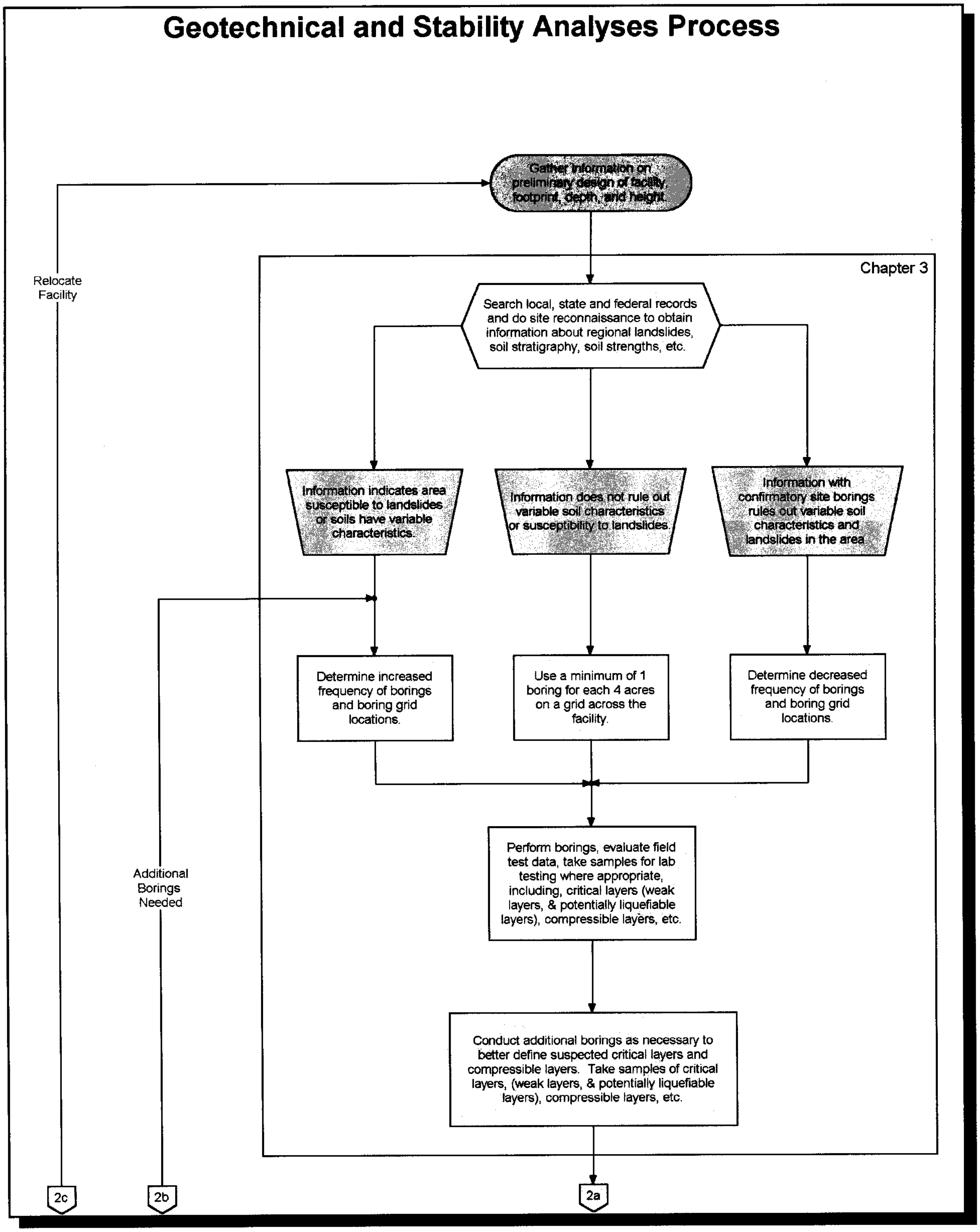
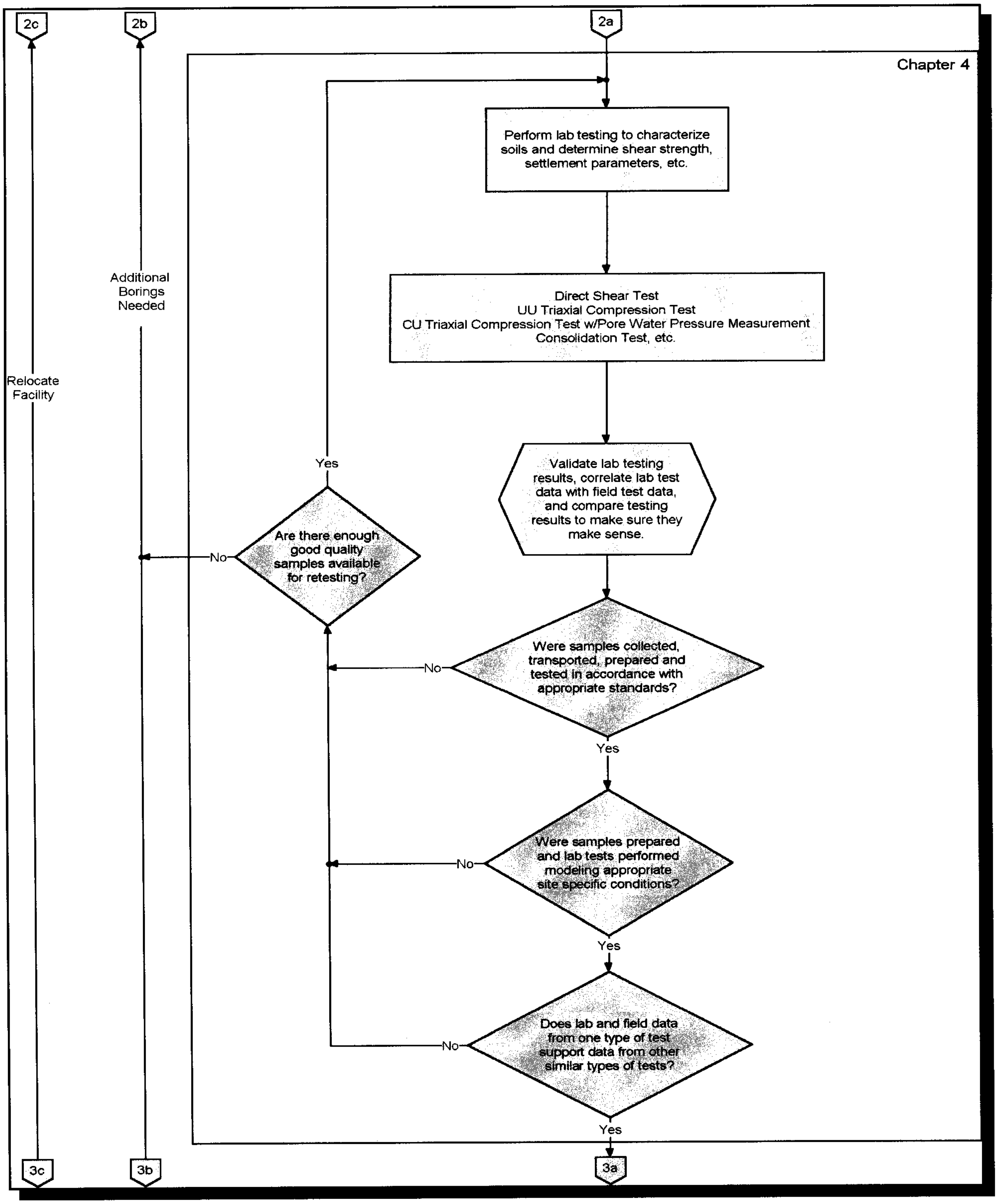
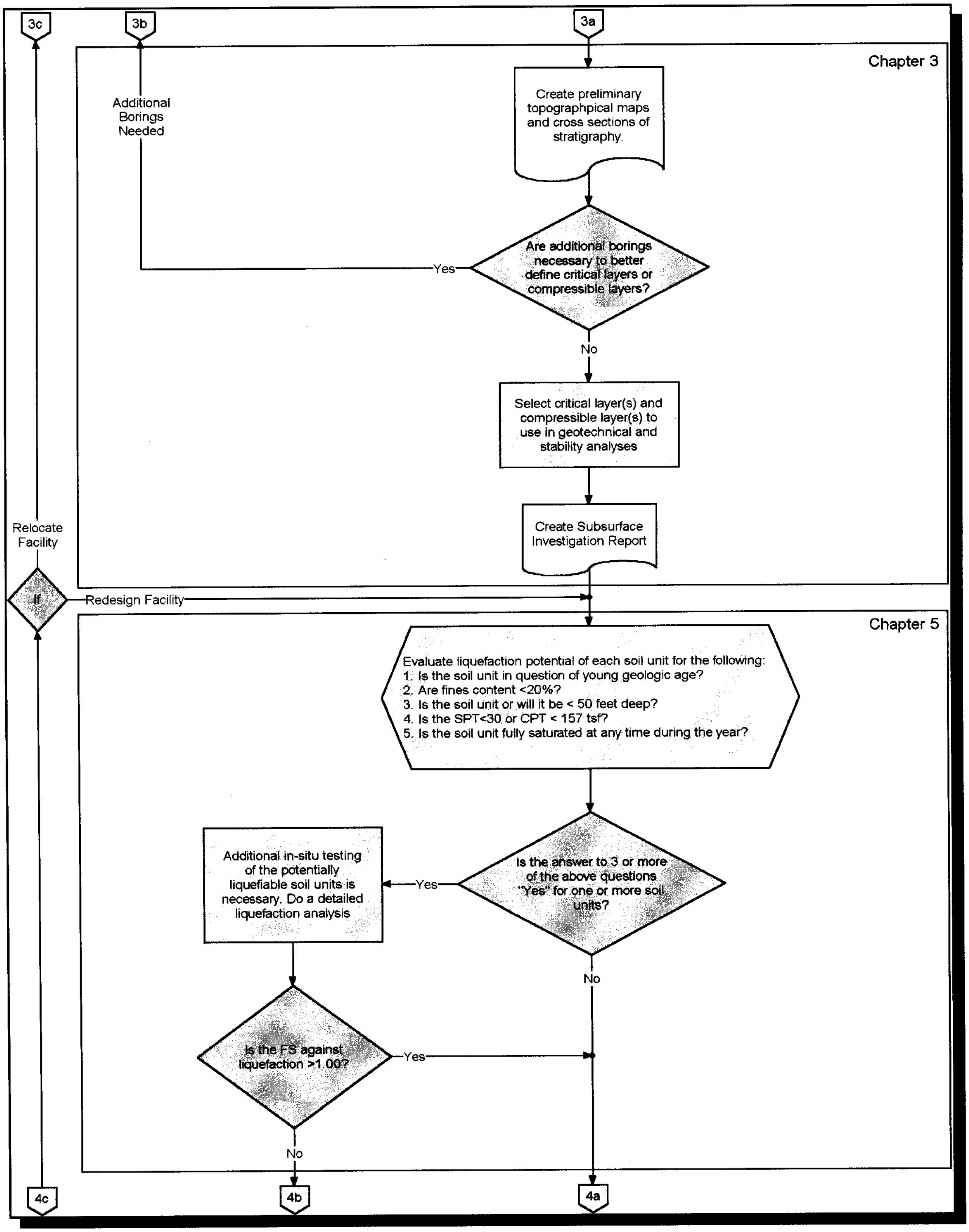


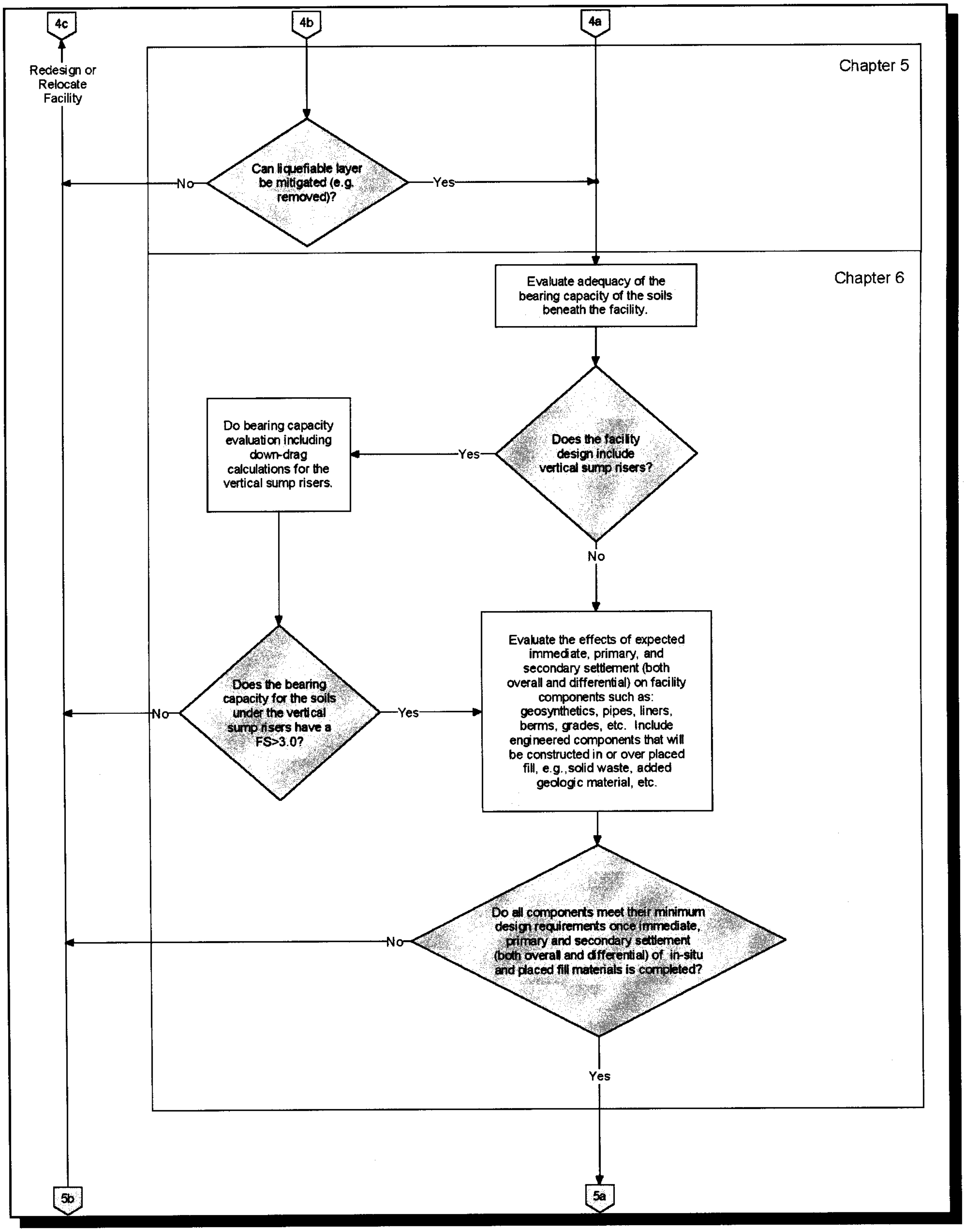
Figure 2-3 Page 1. Geotechnical and stability analyses flow chart.



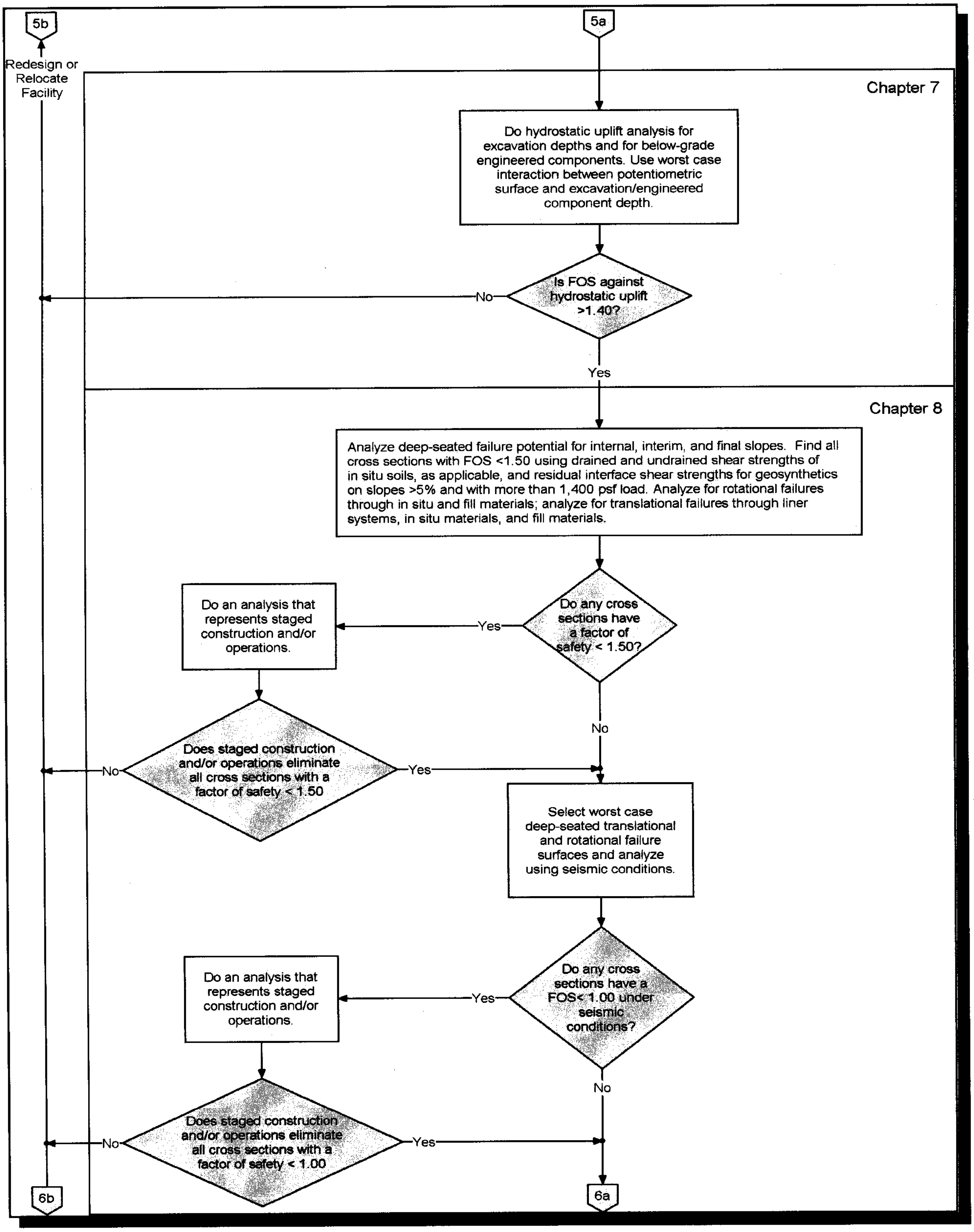
Page 2. Geotechnical and stability analyses flow chart.



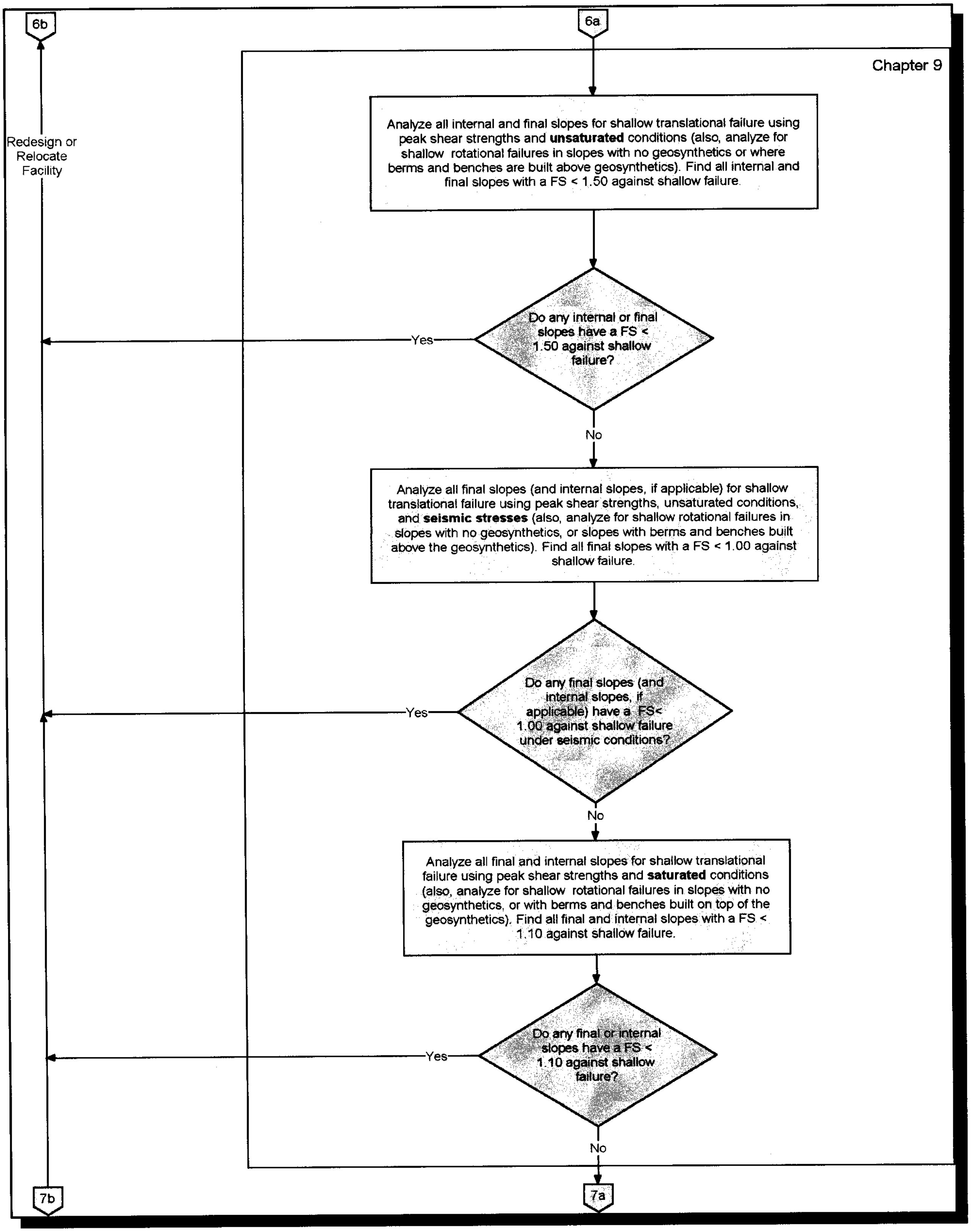
Page 3. Geotechnical and stability analyses flow chart.



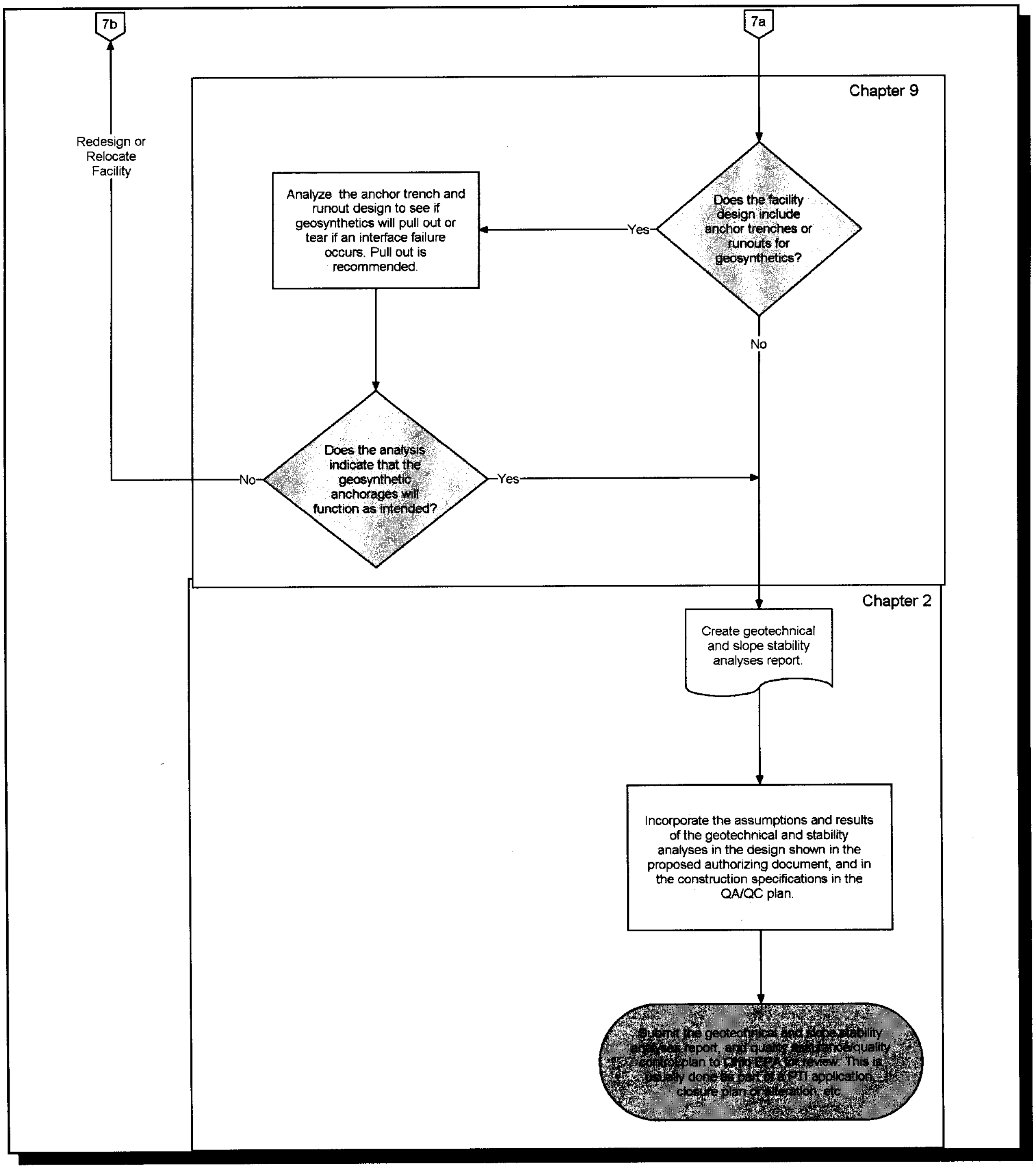
Page 4. Geotechnical and stability analyses flow chart.



Page 5. Geotechnical and stability analyses flow chart.



Page 6. Geotechnical and stability analyses flow chart.



Page 7. Geotechnical and stability analyses flow chart.

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CHAPTER 3

SUBSURFACE INVESTIGATION

This chapter provides information to use when characterizing the *unconsolidated stratigraphic units* (*soil units*) beneath a proposed or existing *waste containment facility* in Ohio. This chapter also includes the recommended format for submitting the results of a subsurface investigation to Ohio EPA for review.

The purpose of characterizing subsurface conditions is to determine if the soils beneath a facility exhibit properties that ensure the facility will remain stable under static and seismic conditions during construction and operation and after it is closed. A complete comprehensive *soil stratigraphy* should be developed that will adequately characterize the lateral and vertical extent of all *soil units* beneath the proposed facility. Characteristics to be measured include, but are not limited to, shear strength, liquefaction potential, compressibility, *phreatic surface* elevations, *piezometric surface* elevations, and the water content of the soil materials. Any *piezometric surfaces* associated with *bedrock* that may affect the facility during excavation, construction, operations, or closure must also be identified. <u>Part of this investigation involves identifying all *critcal layers* beneath the facility</u>. <u>A *critical layer* is any thickness of soil material that has a *drained* or *undrained shear strength* suspected of being capable of causing a failure if all or part of the mass of a facility were suddenly put in place</u>. <u>*Critical layers* may be only a few inches thick to tens of feet thick</u>. <u>*Critical layers* may include parts of one or more *soil units*</u>. <u>Any layer that is potentially liquefiable must also be identified as a *critical layer*</u>.

In addition, the subsurface investigation must be used to identify and characterize all *compressible layers*. *Compressible layers* are soil or fill materials that may settle after establishing a facility, and may continue to settle after a facility has closed. *Compressible layers* must be identified and characterized to determine the bearing capacity and settlement potential of the in situ soils, fill, and stabilized materials that exist on the site. Analysis must show that bearing failure will not occur. Analysis must also show that the engineered components of the facility will meet minimum design requirements during construction, operation, closure, and post-closure of the facility after settlement is complete (at least 100% of *primary settlement*, and the *secondary settlement* expected using a time-frame of 100 years or another time-frame acceptable to Ohio EPA).

A subsurface investigation is typically performed in distinct stages, although some activities of one stage may overlap with other stages. First, a preliminary investigation is conducted to gather and review all available regional and site-specific information. Second, a site-specific investigation is conducted to identify and characterize the *soil stratigraphy* of the site and identify those *soil units* that need further investigation. The *phreatic* and *piezometric surfaces* that exist at the facility are also determined. Finally, *samples* are gathered to be used to produce *higher quality data* from the *critical* and *compressible layers*.

REPORTING SUBSURFACE INVESTIGATION RESULTS

> 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.

Ohio EPA recommends that all of the information be organized and presented so the conclusions are clear and have been justified. The location, extent, and characteristics of all *soil units*, including the *critical layers* and the *compressible layers*, and the elevations of the temporal high *phreatic surfaces* and the temporal high *piezometric surfaces* should also be included (see Table 2 on page 3-8). Laboratory test reports should include all intermediate data gathered during the test along with the results. Reporting should be performed according to the ASTM reporting requirements for the methods being used when reporting requirements exist. Rejected and failed test results should also be reported to Ohio EPA. A brief narrative describing the reasons each test was rejected or considered failed should be included. Ohio EPA recommends that all data be organized and tabbed so that they can be easily located.

To expedite the review process, present the results and conclusions of the investigation with the following sections in the order described. Specific recommendations for each section of the subsurface investigation report are discussed below.

Summary Narrative

The summary narrative should describe the rationale behind the site investigation, the assumptions and methodologies used, the *critical layers* and *compressible layers* selected, the temporal high *phreatic surfaces* and temporal high *piezometric surfaces* defined, and the characteristics of each item identified. The summary narrative should also include recommendations for the values for the characteristics of each material and interface tested to use during modeling, design, and construction.

Summary Table

A summary of all field test data and laboratory test data obtained from all *borings* conducted and *samples* collected at the facility should be presented in one or more tables. The data in these tables that represent the *critical layers* and *compressible layers* should be identified as such. Each record in the table should be referenced to the laboratory testing data sheets, *boring* logs, or other appropriate source.

Topographic Maps

The summary and conclusions section should include one or more topographic maps of the facility that show the location and identification of each *boring* and *sample* collection point at the facility. The limits of the *waste containment unit(s)* should also be shown. These maps can be used to identify the cross sections provided in the report, to show the lateral extent of each *critical layer* and each *compressible layer* that exists at the facility, and to show the elevations of the temporal high *phreatic surfaces*, and the elevations of the temporal high *piezometric surfaces*.

Cross Sections

Cross sections should be included for each length and width of the grid created by the site characterization *borings*. All *borings* that intersect each cross section should be shown in two cross sections oriented roughly perpendicular to each other. Any additional *borings* that intercept the *critical layers* or the *compressible layers* should also be included on appropriate cross sections.

The cross sections should show the vertical and lateral limits of each *soil unit* using the Unified Soil Classification System (USCS) or the American Association of State Highway and Transportation Officials (AASHTO) *unconsolidated material* classification. The vertical and lateral limits of *soil units* should be grouped together or further divided based on the characteristics that affect the geotechnical and stability analyses. These characteristics include, but are not limited to, shear strength, compressibility, liquefaction potential, Atterberg limits (including liquidity index), corrected blow counts, cone penetrometer data, and permeability. When *samples* have been taken from a *boring*, the classification and characterization data obtained from the *samples* should be shown with the *boring* at the *sample* elevation in each cross section that it appears. The *critical layers* and *compressible layers* should be noted as such on the cross section, along with the temporal high *phreatic* and *piezometric surfaces* that exist at the facility. The cross sections should show the proposed and/or existing vertical and lateral limits of the facility excavation and engineered components as encountered by each cross section.

Preliminary Investigation Results

This section of the report should include a discussion of the findings of the preliminary investigation and the sources of information used. The information included in this section should describe evidence that was found, if any, that indicates *critical layers* or *compressible layers* may exist in the area. It should also include a summary of the evidence, if any, of historical mass movements of soil or *bedrock* materials or settlement sufficient to cause damage at the facility or in the region. If *critical layers, compressible layers*, occurrences of mass movements of soil or *bedrock* materials, or landslides exist in the region, then a discussion must be included to describe the steps taken to incorporate these findings into the site characterization.

Site Characterization Results

A summary of the activities, methods, and findings that resulted from the site characterization should be included at the front of this section. A description of the information used to identify the possible *critical layers* and the *compressible layers* designated for further investigation should be included in this section. Also included in this section should be the information used to determine the temporal high *phreatic* and *piezometric surfaces.* All data gathered during the site characterization and field testing should be organized, tabbed, and included in this section. This includes all *boring* logs for the subsurface investigation, blow counts, field test results, and any other information used for defining the potentially *critical layers* and the potentially *compressible layers.*

Results of the Investigation of Critical Layers and Compressible Layers

A summary of the activities, methods, and findings that resulted from the investigation of potentially *critical layers* and *compressible layers* should be included in the front of this section. This section should also include a detailed description of data that were relied upon and why they were used to determine the lateral and vertical extent and characteristics of the *critical layers* and the *compressible layers*. This section should include the methodologies used for laboratory testing, and a discussion that identifies the criteria used to determine the meaning of each test. The laboratory sheets and field data sheets created during sampling and analyses of the *critical layers* and the *compressible layers* should be organized, tabbed, and included in this section.

CONDUCTING THE INVESTIGATION

Preliminary Investigation

The purpose of a preliminary investigation is to gather existing information regarding in situ soils and *bedrock* material strengths, liquefaction potential, and compressibility of the soils from the facility and the surrounding region. All potential sources of information should be checked for evidence of landslides, mass movements of soil material or *bedrock*, strength data, and stratigraphy. Many potential sources for this information exist, such as:

- Field reconnaissance, including a site walkover and field mapping,
- Existing site information such as *boring* logs, open excavations, and utilities installations,
- Local sources such as the health department, soil and water conservation districts, building inspection departments, the county auditor's office, and local newspaper articles,
- State sources such as the Ohio Department of Natural Resources' (ODNR's), Division of Geological Survey and Division of Mineral Resources Management, the Department of Transportation (ODOT), Ohio EPA,
- Federal sources such as the United States Department of Agriculture (USDA), the Natural Resources Conservation Service under USDA, and the United States Geological Survey (USGS).

> Site topography can reveal evidence of historic slope failures and the potential for failures occurring. For example, some indications that downslope movement has occurred or is occurring include:
>
> - Leaning trees, telephone poles, and fence lines,
> - Sections of roads, fences, or telephone lines that are displaced relative to others on either side,
> - Hummocks of grass and vegetation that look like rumpled carpet at the toe of slopes,
> - Surface springs or artesian wells,
> - Flood plain (alluvium) or erosion deposits (colluvium),
> - Cracks near the shoulder of a slope running roughly parallel to the toe of the slope,
> - Cracks that when viewed from a distance create an inverted arc,
> - The existence of near vertical escarpments, and
> - Aerial photographs that show what appears to be a flow of material down and away from an elevated area.

These and other sources can provide information such as aerial photographs, *boring* logs, and reported incidences of mass movements of *bedrock* and soil material that may have occurred in the area. Information about the *soil stratigraphy* in the area can also be gained from these types of sources.

During the preliminary investigation, existing field and laboratory test data from the site might be obtained. When this happens, the data must be evaluated to determine if they were appropriately validated and are thus still usable. This evaluation can be done by applying many of the same procedures to the data as they are discussed later in this chapter and in Chapter 4. If the data are valid and applicable, they can be used, as appropriate, along with newly acquired data. However, any data that cannot be verified to be valid and reliable must be excluded for use.

Site Characterization and Screening

The purpose of site characterization and screening is to identify the temporal high *phreatic surfaces*, the temporal high *piezometric surfaces*, and the vertical and lateral extent of all potentially *critical layers*, and all potentially *compressible layers*. Site characterization and screening are generally performed using investigation and sampling methods that produce *lower quality data*. The data obtained are often well-suited for comparing relative characteristics of different soils, but are unreliable for determining the best obtainable definitive measurement of any given characteristic.

Besides gravity, water is one of the most important factors in stability. Water can affect stability in at least five ways:

1. Reduces shear strength,
2. Changes the mineral constituents through chemical alteration and solution,
3. Changes the bulk density,
4. Generates pore pressures, and
5. Causes erosion.

The areas to be investigated should include the *soil units* from the original ground surface to at least 50 feet below the depth of the deepest excavation proposed at the facility. Extending the investigation deeper to ensure the facility will remain stable may be necessary, especially when evidence exists of *critical layers* or *compressible layers* more than 50 feet below the deepest excavation. All *phreatic surfaces* and *piezometric surfaces* that are likely to affect the stability of the facility must be identified, regardless of the depth or materials associated with the surfaces.

Critical layers may be relatively thin. The site characterization should be planned and conducted so that all *critical layers* will be found, even if they are only a few inches thick. *Critical layers* may be only part of a single broader stratigraphic or hydrogeologic *soil unit*. Averaging of strength values across part or all of a *soil unit* is unacceptable because it may mask the lower strength values of the *critical layer*(s) within a *soil unit*.

Averaging the characteristics of *compressible layers* should also be avoided so that *differential* and *total settlement* can be properly estimated. Enough valid data must be provided to ensure the identification of all *critical layers* and *compressible layers* and all temporal high *phreatic* and *piezometric surfaces* that may affect the stability of the facility. To accomplish this, initial exploratory *borings* should be performed at a minimum frequency of one (1) *boring* for every four (4) acres on a fairly uniform grid across the facility. This is to help ensure the data gathered are representative and increase the likelihood that local geological discontinuities are discovered. *Borings* may be moved laterally from the grid to accommodate site topography and features. Site-specific knowledge should always be used to enhance the site investigation. Some *borings* must be conducted near areas of a site where engineered components will be placed that may be especially sensitive to settlement (e.g., landfill sumps, shallow grade piping, waste water outlet structures, or dikes having relatively little freeboard).

Figure 3-1 Drill rig and operator conducting a standard penetration test (SPT).

A lower frequency of *borings* may be acceptable to Ohio EPA at facilities that have comprehensive and reliable information from the preliminary investigation and information from existing or confirmatory site *borings* that demonstrate that soil materials at the facility are uniform in liquefaction potential, shear strength, and compressibility. Sites that have little preliminary investigation data available, exist in areas where landslides or mass movements of soil materials have occurred, or have evidence of variable soil characteristics will likely be required to increase the frequency of *borings*. Additional *borings* may also be necessary to define the lateral and vertical extent of potential *critical* and *compressible layers* adequately.

Except as modified in this policy or in the Ohio Administrative Code, the procedure for exploratory *borings* should follow ASTM D 420 "Guide to Site Characterization for Engineering, Design, and Construction Purposes." Standard penetration tests (SPTs) with corrected blow counts, CPTs, or another method should be conducted in each *boring.* To find thin *critical layers*, initial exploratory *borings* conducted on a grid pattern should be *sampled* and logged continuously for a minimum of 50 feet below the elevation of the deepest excavation (see Table 3 on page 3-9). *Borings* may need to be *sampled* and logged continuously even deeper if evidence exists indicating that deeper *critical layers* or *compressible layers* may affect the stability of the *waste containment facility.*

In some cases, it is necessary to stabilize a borehole due to heaving soils. The use of hollow-stem augers, or drilling mud has been proven effective for stabilizing a borehole without affecting the blow counts from a standard penetration test. Casing off the borehole as it is advanced has also been used, but it has been found that for non-cohesive soils, such as sands, it has an adverse effect on the standard penetration test results (Edil, 2002).

If CPTs are used, though blow counts will not be measured, the other physical testing discussed below will still need to be performed during the investigation of the *critical layers* and the *compressible layers.* If hydrological data are not otherwise available, temporal high *phreatic* and *piezometric surfaces* must be determined in relation to the local *soil stratigraphy* via piezometers, on-site groundwater monitoring wells, or other field methods.

Investigating Critical Layers and Compressible Layers

Once the *critical layers* and *compressible layers* are located, additional *borings* may be needed to obtain *samples* of each layer, to determine the lateral and vertical extent of each layer, and to define the range of shear strengths and compressibility parameters, along with other characteristics that may affect the stability of a facility. To accomplish this, a representative number of *samples* of each *critical layer* and *compressible layer* must be collected and analyzed. When *borings*, in addition to those performed during the site characterization and screening, are being conducted specifically to obtain *samples* of *critical layers* or *compressible layers*, logging is not required beyond what is necessary to ensure that *samples* are being collected from the targeted *critical layers* and *compressible layers*.

> Residual soil and weathered *bedrock* can be weakened by preexisting discontinuities such as faults, bedding surfaces, foliations, cleavages, sheared zones, relict joints, and soil dikes. Relict joints and structures in residual soils often lose shear strength when *saturated.* Slickensided seams or weak dikes may also preexist in residual soil and weathered rock slopes. Faults, bedding surfaces, cleavages, and foliations often have more influence on rock stability than soil stability.

Characterizing *critical layers* is generally accomplished using investigation and sampling methods that produce *higher quality data*. The data obtained are well-suited for determining the best obtainable definitive measurement of any given characteristic. To provide enough accurate and reliable *higher quality data* to characterize a facility adequately, undisturbed *samples* from each *critical layer* and each *compressible layer* encountered should be collected and laboratory tested from at least ten (10) percent of the *borings* passing through such layers, or a minimum of three (3) undisturbed *samples* from each *critical layer* and each *compressible layer* should be collected and laboratory tested, whichever is greater.

If CPT data or other valid definitive field shear strength data can be used to identify the *critical layer*(s), and if for analytical purposes, it can be appropriately assumed that the weakest layer exists under the entire facility, then undisturbed *samples* from only the weakest *critical layer* need be collected and analyzed, unless evidence suggests doing otherwise. However, consolidation parameters must be obtained from all *compressible layers* to analyze *differential settlement* properly. The lateral and vertical extent of each *critical layer* and each *compressible layer* are to be defined based on results of testing and the location of *borings*.

Laboratory testing and analyses should include, but are not limited to, determining Atterberg limits (including liquidity index), grain size distribution, natural moisture content, dry density, soil classification, consolidation parameters, and shear strength testing. The stress history and existing overburden stresses experienced by each *sample* while in situ must be taken into account during shear testing. Consolidation testing must be conducted to provide information for estimating immediate settlement, *primary settlement*, and *secondary settlement* associated with the facility and its underlying soils (see Chapter 4 for more details about testing methods).

In addition to testing *critical layers* and *compressible layers*, it is recommended that any soils that are identified for use as structural fill or recompacted soil layers be tested during the site investigation. The testing should be conducted at the lowest density and the highest moisture content that is likely to be specified for use during construction. Care should be taken to ensure that soils expected to exhibit the weakest shear strengths are included in the testing. This will allow the use of appropriate values for the shear strength of structural fill and recompacted soil components during stability analyses.

Table 2. An example subsurface investigation report table of contents.

Section No.	**Section Title**
1.0	Summary and Conclusions
1.1	Site Description
1.2	Rationale of Investigation
1.3	Assumptions
1.4	Methodologies
1.5	Description of Critical Layers due to Shear Strength
1.6	Description of Critical Layers due to Liquefaction Potential
1.7	Description Compressible Layers
1.8	Tables
1.9	Figures
1.10	Topographical Maps
1.11	Cross sections
2.0	Preliminary Investigation Results
2.1	Results and Conclusions of the Preliminary Investigation
2.2	Description of the Preliminary Investigation
3.0	Site Characterization
3.1	Results and Conclusions of the Site Characterization and Screening
3.2	Description of Site Characterization and Screening
3.3	Field Test Results
Tab FT1	Field Test Type 1
	Results
	Methods
Tab FT #...	Field Test Type #...
	Results
	Methods
4.0	Investigation of Critical and Compressible Layers
4.1	Laboratory Test Results
Tab LT1	Laboratory Test Type 1
	Results
	Methods, QA/QC, Data Validation, etc.
Tab LT #...	Test Type #...
	Results
	Methods, Laboratory QA/QC, Data Validation, etc.

Table 3. An example *boring* log.

OHIO LANDFILL
LOG OF BORING NO. SPT-3

COORDINATES: N 2418.60 E 4159.13
SURFACE EL: 681.08

Elev. (ft MSL)	Depth (Ft)	Sample #	Type	Blows / 6 in.				N	Recovery (in)	USCS	Description
680		1	SPT	2	2	2	3	4	19	ML	Top soil soft orange-brown, moist to wet, no laminations, silt and clay w/ trace fine sand
		2	SPT	3	7	11	14	18	23	CL	stiff to very stiff, orange-brown and gray, moist, mottling no laminations, silt some clay trace fine sand and gravel
	5	3	SPT	5	9	12	18	29	20	ML	same as above less clay
		4	SPT	4	6	8	13	18	20	CL	same as above more clay
	10	5	SPT	5	6	8	11	17	23	CL	same as above more clay
670		6	SPT	4	4	4	4	9	24	CL	stiff, orange-brown and yellow-brown, wet, no mottling laminations, silt and very fine sand trace clay
		7	SPT	2	4	7	15	12	24	CL	stiff, orange-brown and yellow-brown, wet, mottled, silt some clay trace fine sand and gravel
	15	8	SPT	9	10	11	18	22	24	CL	same as above
		9	SPT	4	6	7	9	13	24	CH	stiff, red-brown, laminated, moist, clay trace silt, highly plastic
		10	SPT	2	3	3	3	6	24	SC	loose, yellow brown sand, wet
660	20	11	SPT	2	2	3	2	5	24	SC	same as above
		12	SPT	2	2	2	2	4	24	CH	soft, yellow brown silt, laminated with red brown clay, moist to wet, highly plastic.
	25	13	SPT	50	-	-	-	-	-		refusal

Date Project Began: 12- 3-97	ground water elev: 662 Date: 12- 7-97	notes: (*boring* continues)
Date Project Ended: 12-12-97	ground water elev: Date:	Below 5' N has been normalized
Field Geologist: CLW	Drilling method: 4 1/4" I.D. H.S. Auger with continuous	using a method recommended in
Checked By: FTR	standard split spoon sampling w/liner, w/standard safety hammer. $N = N_{60} 0.77 \log_{10}(20 / overburden\ pressure)$	Peck Hansen and Thornburn, 1974

Note: Shelby tube *samples* should be taken from the layers with relatively lower blow counts at the site and from layers with compressible materials present.

Note: Though Shelby tube *samples* of the loose sand at 20' are not necessary, the sand layer would be considered a compressible material to be taken into account during settlement analysis. In this instance, immediate settlement of the sand would be the primary concern.

Note: If a nonstandard sampler or nonstandard hammer was used, the characteristics of the nonstandard equipment must be described.

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CHAPTER 4

MATERIALS TESTING PROGRAM METHODS AND ASSUMPTIONS

This chapter provides information to use when conducting or reviewing testing results that will be used in geotechnical and stability analyses for a *waste containment facility* in Ohio. It also addresses selecting appropriate test results for materials and interfaces that will be used for design or construction.

At a minimum, testing of in situ soil materials must occur during the subsurface investigation when preparing to design a *waste containment facility*. Testing of soil materials that will be used for structural fill, recompacted soil layers, and other engineered components can be conducted during the subsurface investigation (recommended) or as *conformance testing* before construction. Testing of the interface shear strengths of geosynthetics and the internal shear strengths of geosynthetic clay liners (GCL), is likely to occur as *conformance testing*. This is due to frequent changes in geosynthetic materials on the market and the time between design and construction. However, designers may want to evaluate their designs against appropriate test results for typical materials that are available. This will allow the designer to evaluate the likelihood that appropriate materials will be available when needed.

It is expected that the appropriate ASTM test methods or other applicable standards will be followed whenever testing of materials is being performed. When using approved test methods, ensure the testing apparatuses and the *specimens* are prepared and used so that the test results are appropriately conservative in representing the field conditions in which the soils and geosynthetics will be used. Common tests used during geotechnical investigations addressed in this chapter are:

For soils;

- Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D 3080),
- Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (ASTM D 2850),
- Standard Test Method for Unconfined Compressive Strength of Cohesive Soil (ASTM D 2166),
- Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils (ASTM D 4767), and
- Standard Test Method for One-Dimensional Consolidation Properties of Soils (ASTM D 2435).

For interface testing;

- Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method (ASTM D 5321), and

- Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method (ASTM D 6243).

General Criteria for Modeling Site Conditions when Preparing Samples and Running Tests

In 1974, Ladd stated, "The results of research have shown that major variations in strength can be caused by *sample* disturbance, strength anisotropy, and strain-rate effects. None of these effects is explicitly included in present design practice. The reason the present methods generally work is that the variations frequently tend to be self-compensating. It is therefore quite possible for the resulting design to be either unsafe or overly conservative, particularly in view of the large scatter often found in triaxial test data." Additional research since then has continued to confirm these findings (e.g., Jamiolkowski, et al, 1985).

Failure planes propagate through the materials and interfaces that exhibit the weakest shear strength at any given loading. The materials and interfaces that are the weakest are likely to change as the normal load and displacement changes. As a result, failure planes may propagate through several different interfaces and materials. At many *waste containment facilities*, a large array of materials and combinations of materials often exist under varying normal loads that need to be evaluated for shear strength. Furthermore, *waste containment facilities* can have widely varying site conditions that may affect the applicability and/or validity of testing results, and the site conditions are likely to change over time. Because of these variables, it is extremely important to ensure that *samples* of soil and construction materials are prepared and tested so that they conservatively represent the expected worst-case field conditions for each facility-specific design.

Factors Affecting the Validity and Accuracy of Soil Shear Strength Testing

The commonly used unconfined compression tests and unconsolidated-undrained triaxial compression tests tend to produce values of *undrained shear strengths* that exceed field values because of the triaxial compression stress condition and the high strain rate used (60%/hr). However, *sample* disturbance, on the other hand, tends to cause lower values of *undrained shear strength* provided that drying of the *sample* is avoided. These effects may compensate each other and yield a reasonable average design shear strength. However, the method is highly empirical and these compensating factors are not controlled or controllable, but in practice, the disturbance effects can be greater than the testing effects and thus the resulting *undrained shear strengths* are often conservative. The situation is further confused by the tendency for *sample* disturbance effects to increase with depth and to obscure shear strength variations in the profile. *Sample* disturbance typically underestimates the *undrained shear strength* of a *sample* from 20 to 50%. Stress-strain anisotropy can cause differences between the *undrained shear strength* obtained by different tests to vary by a factor of 1.5 to 2.5. For triaxial compression tests, each log cycle decrease in strain rate is typically accompanied by a 10 to 15% decrease in *undrained shear strength.* For highly plastic, creep susceptible clays, triaxial compression strength obtained from consolidated *samples* failed at an axial strain rate of 60%/hr (typical for UU triaxial and Unconfined Compression tests) can be 1.2 to 1.3 times the shear strength obtained at 0.5%/hr (typical for CU triaxial tests w/pore water pressure measurement) (Quoted and adapted from Ladd, 1974). The variability discussed by Ladd is largely independent of the triaxial compression test conducted and thus is inherent in the variability of soil material properties and the difficulties experienced during sampling. As a result, variations in values of *undrained shear strength* are still found in testing today (Stark, 2002).

It is important to model failure surface propagation through a composite system at varying normal loads. To do this, the individual failure envelopes of each material and interface in the composite system can be plotted on one shear stress vs. normal stress graph. The weakest compound envelope (see **Figure 4-1**) can then be determined and used for calculating or verifying the stability of the composite system (see Conformance Testing starting on page 4-15 for more details).

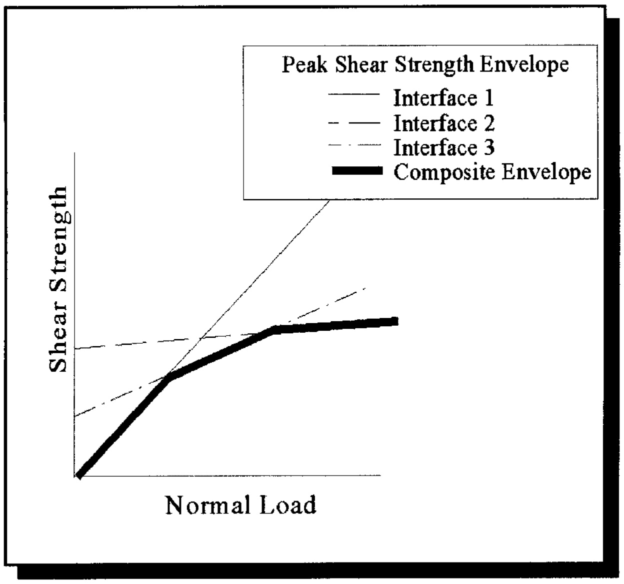

Figure 4-1 Example of a compound *peak shear strength* envelope for a multi-layered engineered component of a *waste containment facility*.

At some facilities, the shear strength of a material cannot be ascertained through laboratory testing. Using empirical relationships then becomes the only alternative. On the rare occasion that this is necessary, the theoretical or empirical correlation that produces the weakest reasonable estimate of the shear strength should be used. For example, when using correlations between liquid limit and shear strength, the highest liquid limit measured that is representative of the *soil unit* should be used to estimate the shear strength, instead of averaging a number of liquid limits from several *samples*.

In situ foundation materials and project-specific materials must be tested for internal and interface shear strengths over the entire range of normal stresses that will be encountered by the materials and interfaces for a given design. The range of normal stresses that need to be evaluated can be extensive, varying from low values at the perimeter of a facility to much higher values under the deepest areas of a facility. For cover systems, this range includes the low normal stresses caused by the cap materials and any additional stresses that may be induced by surface water diversion benches, roads, or other structures constructed above the cover system, and equipment.

If a reasonable expectation exists that at a future time the *waste containment facility* may be expanded in a manner that will increase the normal stresses associated with the facility, then the *responsible party* should ensure that materials and interfaces selected for construction are tested at the higher normal loads. Otherwise, future expansion may be precluded because it will be unknown if the existing materials can maintain stability under the higher normal loads, and the materials that were used may no longer be manufactured or otherwise available for testing.

Shear strength tests are performed by shearing different *specimens* of the same material or interface at three to five different normal loads to develop the failure envelope. For each test, at least one *specimen* should be sheared at a load that is as near as possible, or preferably below, the lowest expected normal stress that will be experienced by the material or interface in the field. One *specimen* should be sheared at a load that is at least 110 percent of the maximum normal stress expected to be experienced by the material or interface in the field. The remaining *specimens* should be sheared at normal loads well distributed between the low and high loads.

Care must be taken to prevent damage or changes to undisturbed *samples* that would invalidate test results. For example:

- Undisturbed *samples* of soil should be sealed in moisture-proof containers immediately after collection.
- During shipping, the *samples* should be protected from vibration, shock, and extreme heat or cold in accordance with ASTM D 4220, "Standard Practices for Preserving and Transporting Soil Samples."
- Preparation of undisturbed *specimens* should be conducted in an environment that will minimize the gain or loss of moisture, disturbances, and changes in cross sections.

> "...the shear strength of a given soil is also dependent upon the degree of saturation, which may vary with time in the field. Because of the difficulties encountered in assessing test data from *unsaturated samples*, it is recommended that laboratory test samples be saturated prior to shearing in order to measure the minimum shear strengths. Unsaturated samples should only be tested when it is possible to simulate in the laboratory the exact field saturation (that is matric suction) and loading conditions relevant to the design." (Abramson, et al, pp 270)

The hydration necessary for determining the shear strength of in situ materials is dependant upon site-specific conditions. Any fine-grained material that is currently, or may become, *saturated* in the field should be tested for *undrained shear strength* in a fully *saturated* condition using the UU triaxial compression test. It is typically assumed that fine-grained in situ materials are or will be *saturated.* For rare cases when fine-grained in situ materials are not *saturated* and are unlikely to become *saturated* in the field, an effective stress analysis using *drained shear strength*s may be conducted using the CU triaxial compression test with pore water pressure measurements and the appropriate site-specific range of normal loads.

The procedures specified in each test method must be followed closely. Other procedures such as setting the rate of the shear stress and the amount of confining stress should be selected carefully to mimic field conditions as much as possible and to avoid obtaining questionable results.

REPORTING

The results of all materials testing completed during the design of the *waste containment facility* should be included in the subsurface investigation report. The subsurface investigation report is described in Chapter 3. At a minimum, the following information about materials testing results should be reported to Ohio EPA whenever it is conducted:

> In addition to the other items included in this chapter, when reporting the results of *conformance testing*, include a comparison of the test results with the requirements contained in rule, the authorizing document, and the assumptions used in the geotechnical and stability analyses, whichever is applicable.

- A narrative and tabular summary of the scope, extent, and findings of the materials testing,
- A description of collection and transport procedures for *samples*,

- The test setup parameters and protocols for each test,
- The *specimen* preparation and pre-test characterization used in each test,
- The intermediate data created during each test,
- The results of each test, and
- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

STANDARD TEST METHOD FOR DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Recommended Uses

The test results from this method are used to assess the shear strength of the material in a field situation where consolidation has occurred under existing normal stresses and no excess pore water pressure is expected to develop during construction or placement of loads on the material. Examples of components that may be tested using this method are granular drainage layers and soils that will be used for structural fill.

> The direct shear device consists of two metal boxes, or "frames," oriented so their open sides face each other. A *specimen* is placed in the direct shear device and consolidated using a normal compressive load representative of field loading conditions. Then one frame is displaced horizontally while the other frame remains at rest. The displacement must be at a constant rate resulting in the ability to measure the shearing force and horizontal displacements during the shearing process.

This test is not usually used when trying to determine the *drained shear strength* of fine-grained cohesive soils, such as in situ foundation soils or recompacted soil liners. Several reasons for this are:

- The consolidation of the *specimen* and the shear rate during testing must be performed very slowly for these types of materials to ensure that the soil *specimen* remains in a *drained condition* during the test. This makes the test inconvenient and often expensive for testing fine-grained cohesive soils.
- The results of this test may not be applicable to fine-grained cohesive in situ foundation soils and recompacted soil layers that will be subjected to high normal loads after they are constructed. This is because the loading experienced by these layers during construction and operations can cause excess pore water pressure to develop.
- During the test, a rotation of principal stresses occurs that may not model field conditions.
- The weakest failure plane through the material may not be identified because the test forces the failure plane to be horizontal through the middle of the *specimen*.

Ohio EPA recommends using triaxial compression testing methods for determining the *drained* and *undrained shear strengths* of fine-grained cohesive soils.

The testing must be continued until a *residual shear strength* is determined or can be conservatively estimated. For slopes that will be permanently loaded with less than 1,440 psf (i.e., final cap), determining the *residual shear strength* may not be necessary. However, it should be carefully considered whether knowing the *residual shear strength* of such a slope will be needed in the future and if it is appropriate for use in current design analysis.

> *Residual shear strength* should be achieved or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative is to use a torsion ring shear device to determine *residual shear strength* for soils and many types of interfaces (Stark and Poeppel, 1994).

Data Validation

Numerous parameters exist that can be checked to verify that the test was performed correctly resulting in valid data. Some of these parameters are:

- Adherence to the maximum particle size restrictions of this method. If these size restrictions are not used, then the ASTM method requires that the grain size distribution of the *specimen* be reported with the shear test results.

- Remolded *specimens* may be adequate to assess the shear strength of structural fill and recompacted soil materials. However, to ensure that the results are applicable to the design or construction of the facility, the materials should be remolded to represent the lowest density and highest moisture content specified during construction, and materials should be chosen from the soils expected to exhibit the lowest shear strengths at those specifications.

> Exceeding the maximum grain size restrictions of the method may result in erratic and inaccurate test results, due to interference with shear plane development and scale effects created by shearing the larger particles. (ASTM D 3080)

STANDARD TEST METHOD FOR UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS (ASTM D 2850)

Recommended Uses

This test is used to determine the *undrained shear strength* of soil. It is applicable to situations where fine-grained soils will be in a *saturated* condition and loading is expected to take place at a rate that overwhelms the ability of the soil materials to dissipate excess pore water pressure.

If *specimens* are *saturated* at the beginning of this test, it is unlikely that consolidation will take place because the drainage lines are closed, allowing the *undrained shear strength* to be determined. The *undrained shear strength* of several similar *specimens* will be approximately the same at different normal loads, resulting in an

> During a triaxial compression test, a cylindrical *specimen* that is wrapped in a membrane is placed into the triaxial chamber, which consists of a top and bottom plate with a stiff walled cylinder in between. A confining pressure using fluid and air is created within the traxial chamber. The *specimen* is then subjected to an axial load until the *specimen* fails. No drainage is allowed to occur during the test.

internal angle of friction of zero. This shear strength measurement should be representative of field conditions that exist when a fine-grained soil material is experiencing excess pore water pressure. Ohio EPA recommends the use of this test when fine-grained soils exist at a facility that are or may become *saturated.*

> *Undrained shear strength* testing is appropriate when the field conditions are such that the loading rate allows insufficient time for induced pore water pressures to dissipate, reducing the shear strength of the materials. Accepted practice is to assume in situ clay materials will be *saturated* for the purposes of shear strength testing, unless site investigation provides a conclusive determination that they are not currently *saturated* and will not become *saturated* at any point during construction, operations, or closure of the *waste containment facility.*

If *specimens* are <u>partially</u> *saturated* at the beginning of this test, compaction (densification by expelling air) will occur before shearing. The shear strength exhibited by the *specimen* will be different at different normal loads, resulting in an angle of friction greater than zero. The shear strength exhibited by the *specimen* will be applicable only when the soils represented by the *specimen* exist in the field at the same saturation as the *specimen* and are subjected to the same range of normal loads as those used in the test. This is unlikely to occur at most facilities that have in situ fine-grained soils in their foundation. For example, a fine-grained soil *sample* collected in August may have a saturation of 75 percent and exhibit a higher shear strength than the same *sample* if it were collected in April, when it may have a much higher level of saturation. Partially *saturated specimens* should not be used for determining the shear strength of in situ foundation soils using the UU triaxial compression test. This is because the conditions represented by the partially *saturated specimen* are unlikely to represent worst-case conditions that are reasonably expected to occur.

Data Validation

A comparison of the pretest density and moisture content vs. the post-test density and moisture content should show that little or no change has occurred, and thus the *specimen* was *saturated* at the start of testing.

It is expected that any given *specimen* of soil will exhibit a similar *undrained shear strength* despite the normal stress used during the test. However, due to variability in the accuracy and precision of the test procedure, Ohio EPA recommends multiple *specimens* of the same soil be sheared at different normal loads as confirmation.

STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL (ASTM D 2166)

> This test is run by placing a trimmed *specimen* of soil between two platens. The *specimen* is not wrapped or confined in any way. The loading platen is lowered at a constant speed until the *specimen* shears. Both the displacement and the shear force are recorded.

Recommended Uses

This test is used to obtain a rapid approximation of the *undrained shear strength* for *saturated* fine-grained cohesive soils. It can be conducted on undisturbed, remolded, or compacted *specimens.*

This test is not appropriate for dry or cohesionless soils. If this test is used, the saturation of each *specimen* before beginning the test must be reported.

If the *specimen* is completely *saturated* at the beginning of the test, the results approximate *undrained shear strength* of the *specimen*. If the *specimen* is only partially *saturated*, then the results approximate the total stress analysis, similar to conducting a UU Triaxial Compression test on a partially *saturated specimen*.

ASTM D 2166 is not a substitute for ASTM D 2850. Ohio EPA recommends ASTM D 2850 be used to develop more definitive data regarding *undrained shear strength* of cohesive soils. Because of the speed, low cost, and potential inaccuracy of ASTM D 2166, Ohio EPA recommends using this test as a screening test to identify weak soil layers that should then have *specimens* tested using ASTM D 2850. ASTM D 2166 results can also be used to augment the understanding of the shear strength of cohesive soils at a facility in conjunction with the results of ASTM D 2850. To do this, the soil *specimen* must be *saturated* and a confining membrane should be used around the *specimen*. ASTM D 2850 includes testing at least three *specimens* from each *sample*, thus producing at least three data points at three different normal stresses. ASTM D 2166 involves testing only one *specimen* from each *sample*. As a result, ASTM D 2166 would need to be run three times for each *sample* under the preceding conditions to produce the same number of data points as one test run in accordance with ASTM D 2850.

Data Validation

The saturation level of each *specimen* needs to be known to determine whether the results are approximating *undrained shear strength* or total stress analysis.

No water should be expelled from the *specimen* during trimming or compression. If this occurs, the material must be tested using the UU triaxial compression test.

Dry and crumbly soils, fissured or varved materials, silts, peats, and sands cannot be tested with this method.

Multiple tests should be conducted for confirmation of the results.

> The expulsion of water from the *specimen* during compression indicates that consolidation of the *specimen* is occurring. The consolidation will increase the apparent shear strength of the *specimen*, rendering the test results unusable for undrained analyses.

STANDARD TEST METHOD FOR CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST (ASTM D 4767)

Recommended Uses

This test is used to determine the total stress, effective stress, and axial compression of cohesive soils by measuring axial load, axial deformation, and pore-water

> A *sample* of in situ fine-grained soil has been subjected to overburden stresses from overlying soils and possibly other geologic occurrences prior to retrieving it from the field. When a *sample* is retrieved, the overburden stress is relieved, and the *sample* may also be disturbed. To increase the representativeness of the shear strength obtained from the CU triaxial test, it is important that a *specimen* is sheared under conditions that mimic, as closely as possible, the in situ stresses.

pressure. This test is to be conducted using *undrained conditions,* while measuring pore water pressure to determine the *drained shear strength* of the *specimen*. The test is applicable to field conditions where soils have been consolidated and are subjected to a change in stress without time for consolidation to recur. To ensure that the test results are applicable to the design of the facility, the test should be run using stress conditions that are similar to the expected worst-case field conditions for the facility. Ohio EPA recommends the use of this test whenever in situ or compacted materials are partially *saturated* and conclusive data shows that it is unlikely that excess pore water pressures will occur during construction of the facility. Ohio EPA also recommends using this test when stability of the *waste containment facility* is being analyzed for the point in time when the pore water pressure in the materials has dissipated (e.g., a staged loading sequence, the point in time after the maximum mass of the facility has been placed and the pore water pressure has dissipated).

Data Validation

For the test results to be meaningful, the over consolidation ratio (OCR) of the *specimen* that existed at the beginning of the test must be known. To accomplish this, the *specimen* must be reconsolidated back to its virgin compression line. For *specimens* that were normally consolidated in situ, the OCR is equal to unity. Therefore, the *specimen* can be sheared after consolidation back to an effective stress greater than that experienced in situ. For *specimens* of overconsolidated in situ materials, the in situ OCR must be calculated from the results of *higher quality data* such as those obtained from oedometer tests. The *specimen* must be reconsolidated back to the virgin consolidation line, and then the effective stress should be reduced to bring the *specimen* back to the in situ OCR. Once the OCR of the *specimen* in the test apparatus matches that of the *sample* in situ, shearing can take place.

The stress history of each *sample* must be carefully investigated to determine how much consolidation must occur to get the *specimen* to return to its virgin compression curve. Usually, *specimens* will need to be consolidated between 1.5 and 4 times the in situ overburden pressure before shearing. For *samples* that were overconsolidated in situ, the apparatus stresses are then reduced so that the OCR in the apparatus is equal to the in situ OCR. The apparatus is set to the normal stress applicable to the design of the facility and to record pore water pressure measurements. The *specimen* is then sheared at a recommended rate of 0.5 percent to 1 percent axial strain/hr.

> Shear testing of quick clays and naturally cemented clays are unlikely to exhibit normalized behavior because the structure of the soil is significantly altered during consolidation to higher stresses. Varved clays may also create difficulties in properly estimating shear strength due to the anisotropy of the soil (Ladd & Foott, 1974). For soils such as these, several different types of shear tests may be necessary, including the direct shear test, to determine the weakest shear plane.

STANDARD TEST METHOD FOR ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

Recommended Uses

The consolidation (oedometer) test is used to determine the rate of primary compression and *secondary compression* of a soil. This test will provide the effective stress-void ratio

> The test apparatus consists of a cylindrical dish that contains the *specimen*. A piston is pushed into the dish under a load to compress the *specimen*. The apparatus allows drainage from the *specimen* as it is being consolidated. The displacement is measured during the test.

(log σ'-e curve), the swelling index (C_s), the compression index (C_c), the preconsolidation pressure (σ_p'), the variation of the consolidation coefficient (C_v) vs. effective stress (σ'), and the *secondary compression* coefficient (C_α). The compressibility (M_v), the permeability coefficient (k)[a] , void ratio vs. effective stress plots, the average degree of consolidation as a function of the time factor [$U(T_v)$] vs. square root of time plots, the void ratio vs. log pressure plots, and the dial reading vs. log time curves should also be reported. The results of this test can be used to evaluate the settlement that is likely to occur under the design loads of a *waste containment facility.*

Data Validation

The test method assumes the following:

- The *specimen* is *saturated* and has isotropic properties (i.e., the *specimen* tested must be representative. The more variation encountered in a *soil unit,* the more *samples* that will need to be tested),
- The compressibility of soil particles and pore water is negligible compared to the compressibility of the soil skeleton,
- The stress-strain relationship is linear throughout the load increment,
- The ratio of soil permeability to soil compressibility is constant throughout the load increment, and
- Darcy's law for flow through porous media applies.
- The void ratio vs log time plot can be used to ensure that the consolidation made a transition from primary to secondary before the next load was added. If no transition is visible in the curve, then check with the lab to find out why subsequent loading was done before the transition into secondary consolidation of the *specimen* had occurred.
- The void ratio vs. log pressure plot can be used to ensure that the void ratio decreased with each new load. If it does not, then this indicates a'problem with the test.

If the above assumptions do not apply to the *specimen,* then this test method may not be appropriate for the selected *specimen*.

The test results are strongly affected by the saturation of the *specimen*. Fully *saturated specimens* must be used. The pre-test saturation level of each *specimen* must be determined and reported.

If more than one *compressible layer* exists at a facility, each layer should be tested to be able to calculate the *differential* and *total settlement* for the facility properly. In addition, enough *samples* from each *compressible layer* should be tested to be able to identify lateral and vertical differences in consolidation

[a] Bardet, J., 1997, Experimental Soil Mechanics. Prentice-Hall, New Jersey. pp. 350.

and compressibility parameters. For example, if the facility has a lower glacial till that is partly overlain by an upper lacustrine deposit, both layers should be tested to obtain an understanding of the lateral and vertical variability of their respective consolidation/compressibility parameters.

The range of the applied stress during the test should cover from the lowest to the highest normal stresses expected to be exerted by the facility.

During testing, the load should be changed after the consolidation caused by the current load reaches 100 percent. However, the load may be changed at convenient times if consolidation exceeds 90 percent for the current load. Generally, each load is in place for 24 hours. For some soils, more than 24 hours under each load may be necessary to allow complete consolidation to occur.

To be able to calculate *secondary settlement,* the load should be maintained at each stage for as long as necessary to determine the *secondary compression* coefficient.

> Obtaining the coefficient of *secondary compression* through testing is only necessary for plastic materials. Published literature can be used to estimate *secondary compression* coefficients for non-plastic materials if they are appropriately representative of the non-plastic materials found at the site.

If excavations are to occur during the construction of the facility that will be filled later with water, waste, or other materials; or if the facility will be filled and then cut during construction or operations; one or more rebound cycles will be created within the foundation soils. A description of the loading that identifies the rebound cycles should be evaluated and communicated to the lab. This is so the loads representing the cutting and filling can be included in the testing.

Test results are affected by *sample* disturbance, affecting the preconsolidation pressure most significantly. The *specimen* selection and preparation methods should not disturb the *specimen* any more than is absolutely necessary when collecting and preparing the *specimen* for testing.

STANDARD TEST METHOD FOR DETERMINING THE COEFFICIENT OF SOIL AND GEOSYNTHETIC OR GEOSYNTHETIC AND GEOSYNTHETIC FRICTION BY THE DIRECT SHEAR METHOD (ASTM D 5321)

Recommended Use

> The test is usually run within a "large box" direct shear apparatus. A constant normal stress is applied to the *specimen* while a shear force is applied to the apparatus.

This test is used to determine the shear resistance of a geosynthetic against soil or another geosynthetic. Using site-specific geosynthetic material and remolded or undisturbed *specimens* of soils from the *waste containment facility* is important. Ohio EPA recommends using this test for determining the *peak shear strengths* and *residual shear strengths* for all interfaces with a geosynthetic that are part of the facility design. However, this test should not be used when testing GCL. Instead, use ASTM D 6243 when testing internal or interface shear strength of a GCL. Sometimes, Ohio EPA may require composite systems containing 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* comprising all the layers in a composite system. For example, if all of the *peak shear strengths* for each interface and material are near each other, but a wide range of *residual shear strengths* exists, 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.

Residual shear strength should be determined or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative is to use a torsion ring shear device to determine *residual shear strengths* for many types of interfaces. (Stark and Poeppel, 1994)

The test must be continued until the *residual shear strength* has been determined or can be conservatively estimated. Sometimes, such as for geosynthetics with maximum permanent loads less than 1,440 psf (e.g., final cap systems), determining the *residual shear strength* may not be necessary. However, even here, it should be carefully considered whether knowing the *residual shear strength* of the interfaces is necessary for current or future design needs.

Data Validation

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 strengths necessary to meet the stability requirements of the design. Interface testing is often not performed during design testing, but is performed during *conformance testing* due to the length of time from design to construction and the changes that may occur in materials that are available. However, at a minimum, designers should review published literature pertaining to the materials anticipated for use in construction to ensure that it is likely that they can meet the minimum required design shear strengths. If no literature exists, then it is recommended that testing occur during the design phase of a project.

To ensure the appropriateness of this test, it must be set up to represent the expected worst-case field conditions. When testing interfaces between geosynthetics and soils, careful consideration should be given to the following:

- Soils used during the test should be recompacted using the highest moisture content and lowest density specified during construction.

- The soil selected should represent soils with the lowest internal shear strength of the soils that will be placed during construction.

Shear strength tests of interfaces with a geomembrane should be conducted fully wetted. This is performed by following the ASTM recommendation for submerging the soil *specimen* before shearing or using a spray bottle to wet the interface thoroughly.

Interfaces with the top of a flexible membrane liner (FML) become wetted in the field either from precipitation or from the liquids contained by the unit. Interfaces with the bottom of an FML become wetted in the field from condensation and from consolidation water.

Samples of geosynthetics used for testing interface shear strength should be selected from the geosynthetic rolls that will be used at the facility or from rolls that represent the materials that will be used at the facility. Materials are considered representative if they are from the same manufacturer, use the same raw materials, use the same manufacturing process, and have the same manufacturing specifications.

STANDARD TEST METHOD FOR DETERMINING THE INTERNAL AND INTERFACE SHEAR RESISTANCE OF GEOSYNTHETIC CLAY LINER BY THE DIRECT SHEAR METHOD (ASTM D 6243)

Recommended Use

> The test is usually run within a "large box" direct shear apparatus. A constant normal stress is applied to the *specimen* while a shear force is applied to the apparatus.

This test is used to determine the shear resistance of a GCL against soil or a geosynthetic. It is also used to determine the internal shear strength of a GCL. Site-specific GCL, geosynthetic materials, and undisturbed *specimens* of soils or *specimens* of soils from the facility remolded using construction specifications and then hydrated to mimic field conditions must be used. Ohio EPA recommends using this test for determining the *peak shear strengths* and *residual shear strengths* of interfaces with GCL, and for determining the internal *peak shear strength* and *residual shear strength* of a GCL.

The test must be continued until *residual shear strength* has been determined or can be conservatively estimated.

Data Validation

> *Residual shear strength* should be determined or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative method, such as torsion ring shear, can also be considered for determining *residual shear strengths*. (Stark and Poeppel, 1994)

The test must be set up and performed to represent the expected worst-case field conditions that will be experienced by the GCL. When testing GCL internal or interface shear strength, careful consideration should be given to the following:

- The soil selected should represent soils with the lowest internal shear strength of the soils that the GCL will be placed in contact with during construction and should be recompacted using the highest moisture content and lowest density specified during construction.

- *Samples* of geosynthetics that will create interfaces with the GCL should be selected from rolls of materials that are representative of the materials that will be used at the facility. Materials are considered representative if they are from actual rolls that will be used during construction. They are also considered representative *samples* if they are collected from rolls that are from the same manufacturer, use the same raw materials, use the same manufacturing process, have the same manufacturing specifications, and are selected from rolls that will create the weakest interfaces.

- *Samples* of GCLs should be selected from rolls of materials that are representative of the materials that will be used at the facility. Materials are considered representative if they are from actual rolls that will be used during construction. They are also considered representative *samples* if they are collected from rolls that are from the same manufacturer, use the same raw materials, use the same manufacturing process, have the same manufacturing specifications, and are selected from rolls that will create the weakest interfaces or the weakest internal shear strength. If needle punched GCL is selected for testing, the test *specimen* should have a peel strength similar to the lowest peel strength sold by the manufacturer (15 pounds with ASTM D 4632 or 2.5 ppi with ASTM D 6496 is the typical minimum average roll value accepted in the United States) or the lowest peel strength specified for use during construction at the facility. An example of this would be choosing *samples* of needle punched GCL from a roll created just before replacing the needles.

> An accelerated hydration procedure can be used to reduce the in-device time for GCL *specimens* to reach hydration time (Fox *et al.* 1998a). According to this method, a GCL *specimen* is hydrated outside of the shearing device for two days under a very low normal stress (• 1 kPa) by adding just enough water to reach the expected final hydration water content (estimated from previous tests). The *specimen* is then placed in the shearing device and hydrated with free access to water for two additional days under the desired (normal seating load) $\sigma_{n,h}$. Most GCL *specimens* attain equilibrium in less than 24 hours using this procedure (Fox *et al.* 1998a, Triplett and Fox 2001) (Fox et al. 2004).

- The hydrating of GCL test *specimens* should be preformed by submerging the GCL *specimen* at a normal seating load approximately equivalent to the initial load placed on the GCL in the field (e.g., 0.8 psi for a one foot drainage layer with a 120 pcf gravel). ASTM D 6243 requires that the swelling of the *specimen* come to equilibrium before beginning to load the test *specimen.* A GCL can be considered fully hydrated when swelling has slowed to less than a five percent change in thickness in twelve hours (Gilbert et al., 1997).

- The loading of GCL test *specimens* from the hydration normal stress to the shearing normal stress should be performed in a manner that allows time for the *specimens* to consolidate. If insufficient time is allowed between loading increments, bentonite will extrude from the *specimen*. If insufficient time is allowed for the final load to consolidate, excess pore pressures will remain in the *specimen* at the start of shearing. These improper loading procedures will produce inaccurate results. A normal stress increment of no more than 50% every half-day (e.g., 0.8 psi, 1.2 psi, 1.8 psi...) has resulted in successful consolidation. If bentonite extrudes from the *specimen* during loading, the test should be repeated with a lower normal stress increment.

- The rate of shear displacement for shear strength tests of interfaces with a GCL should be slow enough so that insignificant excess pore water pressure exists at failure. However, the rate of shear displacement should not exceed 1.0 millimeters per minute (mm/min) until the shear box traverses its maximum length.

- Most studies indicate that internal shear strength increases with increasing displacement rate, although some key studies have produced contradictory results. Until this issue is resolved, a maximum displacement rate of 0.1 mm/min is recommended for GCL internal shear tests. It should be noted that some data sets indicate that an even slower displacement rate is necessary. More research is needed on this issue (Fox et al., 2004).

- A failed GCL or GCL interface test *specimen* should be inspected after shearing to assess the surface(s) on which failure occurred and the general nature of the failure. Unusual distortion or tearing of the *specimen* should be recorded and may indicate problems with the gripping system. The condition of the geosynthetics at the end clamps (if present) should also be recorded. Evidence of high tensile forces at the clamps, such as tearing or necking of the geosynthetics, is an indication that progressive failure probably occurred during the test. Depending on the extent of localized distress, such a test may be invalid and may need to be repeated using an improved gripping system (Fox et al., 2004).

CONFORMANCE TESTING

Conformance testing is conducted on materials that will be used for constructing a *waste containment facility*. *Conformance testing* is used to verify that the materials being used during construction will exhibit the internal and/or interface shear strengths necessary to provide the minimum required factors of safety approved by Ohio EPA. The shear strengths of in situ foundation and construction materials must be verified by comparing the results of the *conformance testing* with the shear strengths specified in the authorizing document as follows:

- In situ foundation soils must be thoroughly tested during the subsurface investigation. Additional testing during construction should not be needed, unless in situ materials are encountered during excavation that may exhibit weaker shear strengths than the values used during the stability analyses (see previous sections of this chapter and Chapter 3 for more information about investigating and testing in situ foundation materials).

- Materials that will be used for structural fill or recompacted soil layers (RSL) will need to be tested during the subsurface investigation (recommended) or during *conformance testing*. These types of materials must be tested using the lowest density and highest moisture content specified for use during construction. The results of two or more complete tests of each type of material being used for structural fill and RSL are needed. If the tests confirm that the materials will exhibit shear strengths that exceed the minimums specified in the authorizing documents, then the materials should not need to be tested again unless construction specifications change, or materials are encountered that may exhibit weaker shear strengths than those already tested (see previous sections of this chapter and Chapter 3 for more information about investigating and testing structural fill and RSL materials).

- Geosynthetic materials, including GCLs, need to be tested for interface shear strength (GCLs also need to be tested for internal shear strength) during *conformance testing*. A minimum of two complete shear tests must be conducted of each interface (as well as internal shear strength of each GCL) before the material is used for the first time at a facility. After that, one complete test must be conducted before each construction project (see previous sections of this chapter for more information regarding testing geosynthetic interfaces and internal shear strengths of GCLs).

The conformance test data for drained and undrained internal shear strengths, interface *peak shear strengths,* and interface *residual shear strengths* should be used to create compound nonlinear shear strength envelopes with each envelope starting at the origin. The type of shear strength (i.e., drained/undrained, peak/residual) used to compare to the specifications in the authorizing document must be the same type of shear strength that was assumed during the stability analyses. The type of shear strengths used during the stability analysis will typically be as follows:

- *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 analysis. This also allows *peak shear strengths* to be used, if appropriate, for shallow analysis of most final caps, granular drainage layers, and *protective layers* on *internal slopes* prior to the time waste has been placed.

- *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*.

- Internal *peak shear strengths* may be used for reinforced GCL, as long as the internal *peak 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.

- *Drained* or *undrained shear strengths*, as appropriate, are required to be used for foundation and construction soil materials. 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 materials that may develop excess pore water pressure.

Residual shear strengths may have been substituted for *peak shear strengths*, especially for interfaces, during the stability analyses. This is done when there is reason 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). If this assumption was used, then *residual shear strengths* derived from corresponding materials during *conformance testing* must be used instead of the *peak shear strengths*.

During stability analyses, a composite liner or composite cap system is often modeled as one layer using a linear shear strength envelope, adjusting the strength during modeling until the minimum required factors of safety are provided. To simplify comparison of the *conformance testing* results to the minimum shear strengths specified by the authorizing documents, a compound nonlinear shear strength envelope can be created for an individual material, interface, or system containing multiple interfaces and layers. Determining which shear stresses to plot when creating a compound nonlinear envelope depends upon the type of shear strength envelope being created as follows:

- For compound nonlinear *peak shear strength* envelopes, select the lowest *peak shear strength* measured for any material or interface at each tested normal compressive stress to define the envelope,

- For compound nonlinear *residual shear strength* envelopes, select the *residual shear strength* associated with the lowest *peak shear strength* exhibited by an interface or material at each tested normal compressive stress to define the envelope,

- For compound nonlinear *drained shear strength* envelopes, select the lowest *drained shear strength* measured at each tested normal compressive stress to define the envelope.

- Compound nonlinear *undrained shear strength* envelopes should not be used, select the lowest representative *undrained shear strength* measured for each material regardless of normal compressive stress.

Compound nonlinear shear strength envelopes can be helpful for describing the shear strength of a material and interface when:

- Several complete interface friction tests of the same interface are conducted, resulting in multiple shear stress values for each normal compressive stress used during the testing. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited in the field by that one interface when subjected to the range of normal compressive stresses used during testing.

- A composite system (e.g., a composite liner/leachate collection system, or composite cap system) has each interface and material tested for shear strength multiple times, resulting in multiple shear stress values at each normal compressive stress used during the testing. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited by the entire composite system in the field when subjected to the range of normal compressive stresses used during testing.

- A soil material to be used for structural fill, RSL, or an in situ material is tested several times resulting in multiple shear stress values at each normal compressive stress used during the test. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited by the soil material in the field when subjected to the range of normal compressive stresses used during testing.

An example methodology for creating compound nonlinear shear strength envelopes can be found starting on page 4-18.

Sometimes, Ohio EPA may require composite systems using multiple materials and having multiple interfaces with geosynthetics 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* comprising all the layers in a composite system. For example, if all of the *peak shear strengths* for each interface and material are near each other, but a wide range of *residual shear strengths* exists, 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.

Developing Compound Nonlinear Shear Strength Envelopes - Example Methodology

A stabilization plan for heavy metal contaminated soil at several locations on a property has been approved by Ohio EPA, DERR as part of a negotiated settlement. The plan includes a CERCLA retention unit. The unit will hold a maximum of 30 feet of stabilized soils. It has 3:1 *internal slopes* and 4:1 external slopes. The approved composite liner system includes four (4) feet of $1x10^{-7}$ cm/sec RSL and is overlain with 60 mils thick textured high density polyethylene (THDPE). The drainage layer includes a geocomposite with a one-foot thick *protective layer* of #57 gravel on top. **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22 show the results of the interface shear strength testing of the three interfaces at 1000 psf, 2000 psf, and 4000 psf normal compressive stress. The graphs show the lowest *peak shear strength* for each interface selected from the results of multiple tests of each interface. The 1000-psf test represents the normal compressive stress of about seven (7) feet of stabilized waste (@130 pcf). The 4000-psf test represents 110% of the normal compressive stress of the weight per square foot of the waste at its deepest point. To ensure that the full range of normal compressive stresses experienced in the field are included, another set of interface tests should have been run for each interface at a smaller normal compressive stress to represent one foot or less of the waste. This would be particularly important if these interfaces were also to occur in the composite cap system. Fortunately for this site, the shear stress from 0 psf to 1000 psf can be adequately estimated by connecting a line from the origin to the shear stress measured at 1000 psf for each interface (see **Figure 4-5** page 4-23 and **Figure 4-7** on page 4-25).

Compound Nonlinear Peak Shear Strength Envelopes

This methodology is appropriate when using *peak shear strengths*. It is used for composite systems comprising multiple layers and interfaces (e.g., composite liners and caps). It also applies when developing a nonlinear shear strength envelope for a single material or a single interface tested several times with varying results at each normal compressive stress. In this example, a compound nonlinear *peak shear strength* envelope will be created from the test results shown on **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22. **Figure 4-5** on page 4-23 shows the non-linear shear strength envelopes for three interfaces, and was created by taking the lowest peak shear stress measured from multiple tests of each interface at each normal compressive stress and plotting the points on a graph showing shear stress on the y-axis and normal compressive stress on the x-axis. The data points used to create **Figure 4-5** are found in Table 4.

To create a compound nonlinear shear strength envelope, select the lowest peak shear stress measured for any interface or material in the composite system at each normal compressive stress (see highlighted values in Table 4). Next, plot the selected peak shear stress values vs. the corresponding normal compressive stress values to produce a graph showing the compound nonlinear *peak shear strength* envelope. The shear stress of the system below the lowest normal compressive stress tested is estimated by connecting a line from the origin to the peak shear stress measured at the lowest normal compressive stress. The *peak shear strength* used when modeling the composite system is then plotted on the graph to verify that the entire nonlinear *peak shear strength* envelope plots above it (see **Figure 4-6** on page 4-24).

Table 4. An example of the lowest peak shear stress measured for three interfaces from a composite liner system at three different normal compressive stresses (data points obtained from **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22). The highlight marks the interface with the lowest peak shear stress at each normal compressive stress.

Interface	Peak Shear Stress (psf)		
	1000 psf Normal Compressive Stress	2000 psf NCS	4000 psf NCS
RSL vs. THDPE	782	1042	2371
THDPE vs. Geocomposite	**465**	1450	**2040**
Geocomposite vs. Protective Layer	568	**1013**	2354

Compound Nonlinear Residual Shear Strength Envelopes

This methodology applies to any composite system comprising multiple layers and interfaces (e.g., composite liners and caps). It also applies when developing a nonlinear *residual shear strength* envelope for a single material or interface tested several times with varying results at each normal compressive stress. The process for developing a compound nonlinear *residual shear strength* envelope is the same as the process for developing the compound nonlinear *peak shear strength* envelope with one exception. When creating the compound nonlinear *residual shear strength* envelope, instead of choosing the lowest *peak shear strength* at each normal compressive stress to plot, choose the residual shear stress associated with the lowest peak shear stress at each normal compressive stress (see highlighted values in Table 5).

Notice that in Table 5, for a normal compressive stress of 2000 psf, the residual shear stress of 984 psf was selected rather than the lowest residual shear stress of 614 psf. This is because 984 psf is the residual shear stress associated with the interface that has the lowest peak shear stress. To create a compound nonlinear *residual shear strength* envelope, use the selected residual shear stresses and the associated normal compressive stresses (see highlighted values in Table 5) to plot shear stress values vs. normal compressive stress values. To ensure that the full range of normal compressive stresses to be experienced in the field are included, another set of interface tests should have been run for each interface at a smaller normal compressive stress to represent one foot or less of the waste. This would be particularly important if these interfaces were to also occur in the composite cap system. To estimate the shear stress below the lowest normal compressive stress used during testing, connect a line from the origin to the residual shear stress measured at the lowest normal compressive stress used during the testing. The *residual shear strength* used when modeling the composite system is then plotted on the graph to verify that the entire nonlinear *residual shear strength* envelope plots above it (see **Figure 4-8** on page 4-26).

Table 5. Examples of the lowest residual shear stresses measured from multiple tests of three interfaces from a composite liner system at three different normal compressive stresses (data points obtained from **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22). The highlight marks the interface with the residual shear stress associated with the lowest peak shear stress at each normal compressive stress.

Interface	(Peak) and Residual Shear Stress (psf)		
	1000 psf Normal Compressive Stress	**2000 psf Normal Compressive Stress**	**4000 psf Normal Compressive Stress**
RSL vs. THDPE			
Peak	(782)	(1042)	(2371)
Residual	684	1003	2320
THDPE vs. Geocomposite			
Peak	(465)	(1450)	(2040)
Residual	**270**	614	**1187**
Geocomposite vs. Protective Layer			
Peak	(568)	(1013)	(2354)
Residual	555	**984**	2300

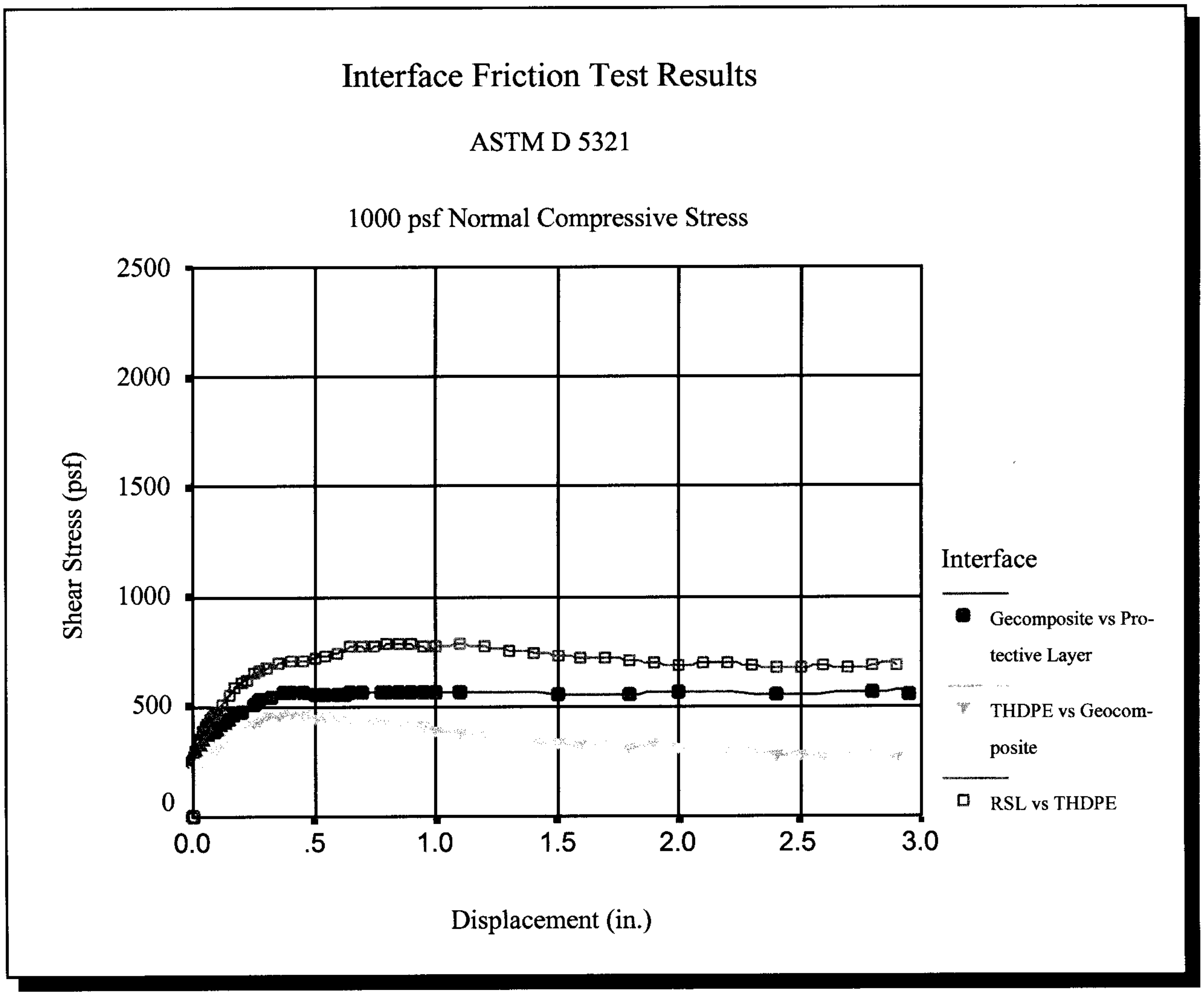

Figure 4-2 An example of interface friction test results for three interfaces of a composite liner system at 1000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

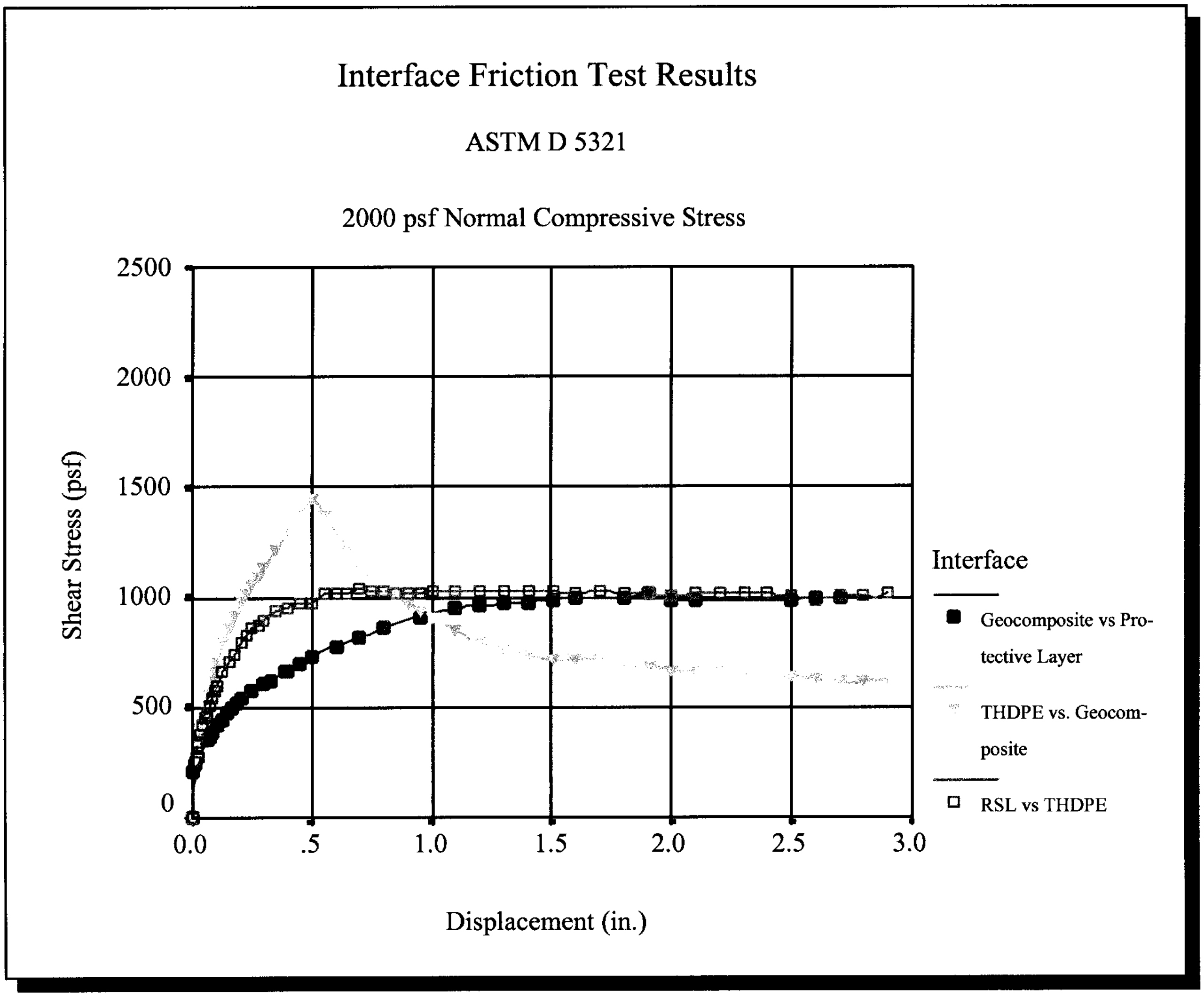

Figure 4-3 An example of interface friction test results for three interfaces of a composite liner system at 2000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

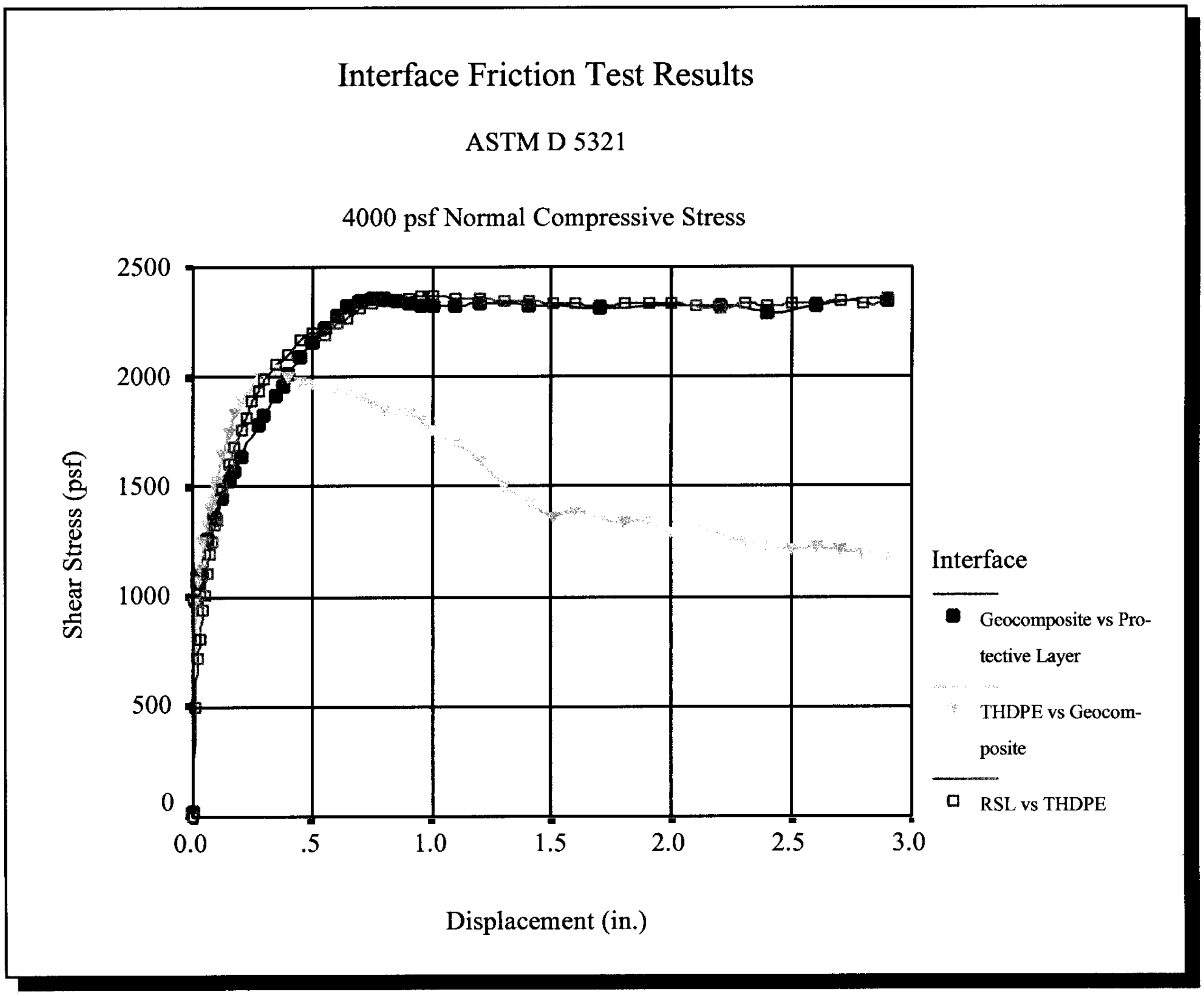

Figure 4-4 An example of interface friction test results for three interfaces of a composite liner system at 4000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

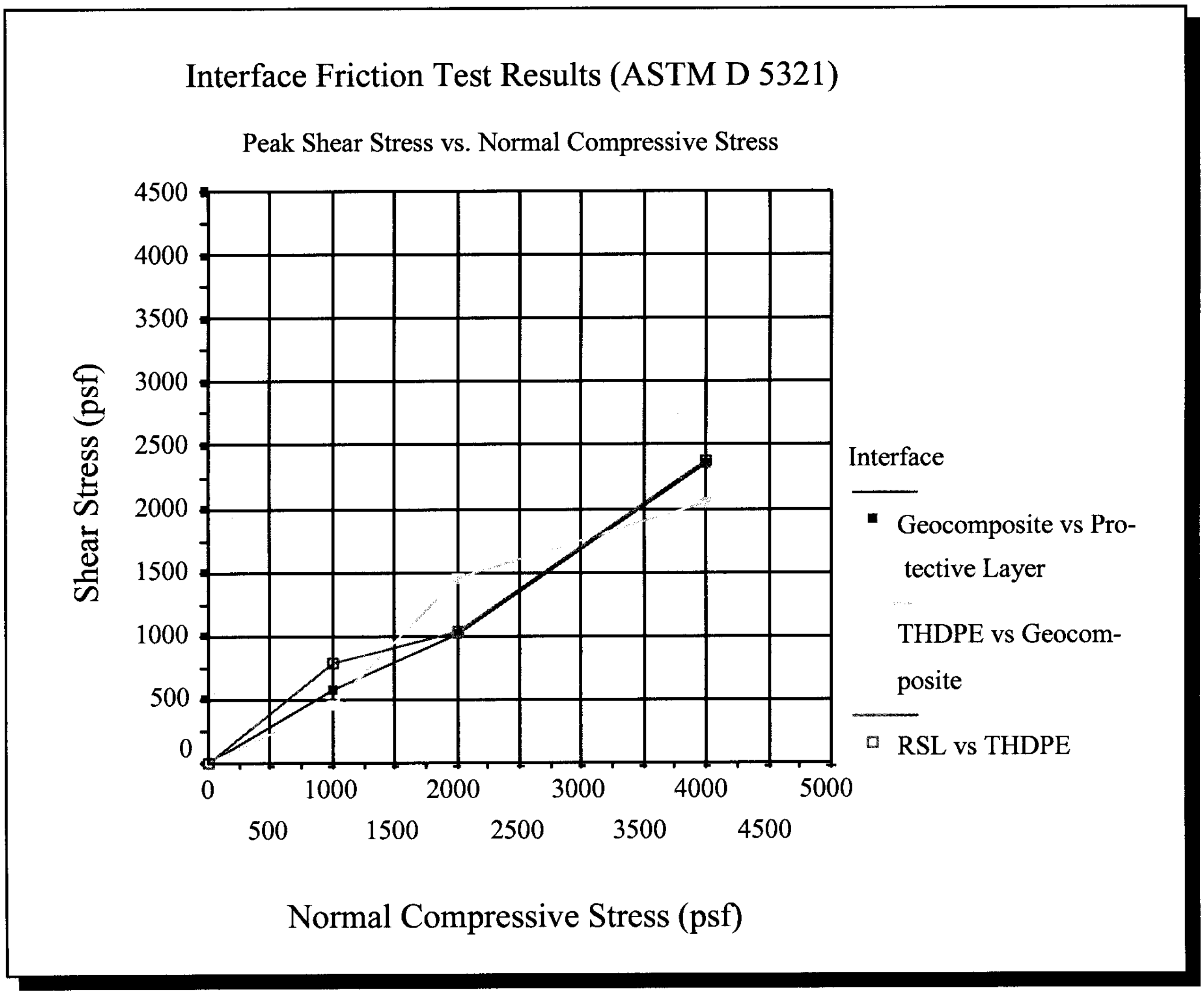

Figure 4-5 An example of individual nonlinear *peak shear strength* envelopes derived from the lowest peak shear testing data at each normal compressive stress for each of three interfaces in a composite system. The shear stress below 1000 psf normal compressive stress was estimated by drawing a line from the origin to the shear stress at 1000 psf normal compressive stress for each interface. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

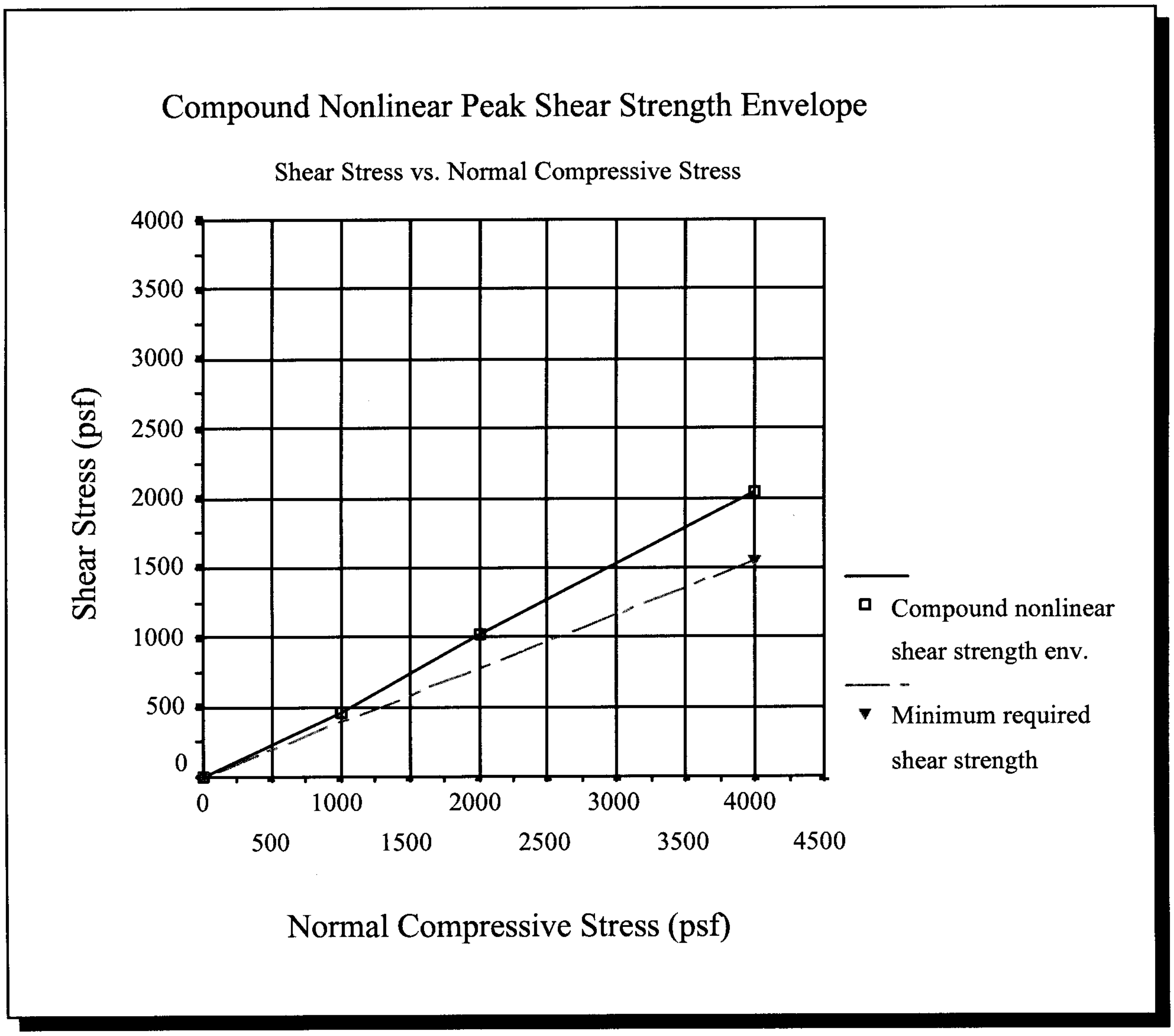

Figure 4-6 An example of a compound nonlinear *peak shear strength* envelope created from the individual nonlinear *peak shear strength* envelopes of three interfaces of a composite system. When the *peak shear strength* envelope is compared to the minimum *peak shear strength* specified in the authorizing document, it can be seen that the composite system exhibits enough *peak shear strength* at all normal compressive stresses expected at the facility, and thus the minimum required *peak shear strength* is exceeded. This ensures that all the tested materials can be used during construction of composite systems when *peak shear strength* conditions are expected. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

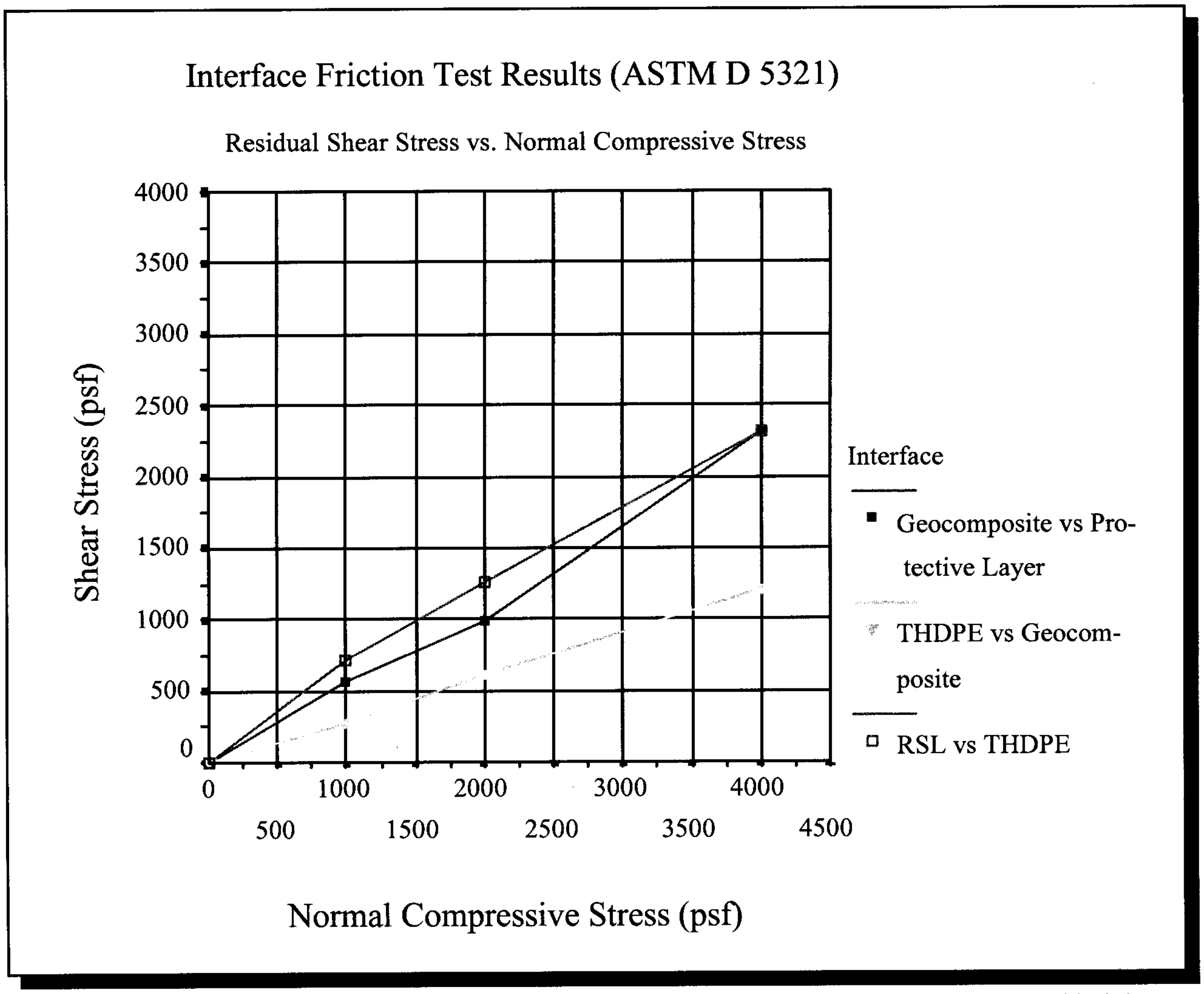

Figure 4-7 An example of individual nonlinear *residual shear strength* envelopes derived from the lowest residual shear testing data at each normal compressive stress for each of three interfaces in a composite system. The shear stress below 1000 psf normal compressive stress was estimated by drawing a line from the origin to the shear stress at 1000 psf normal compressive stress for each interface. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

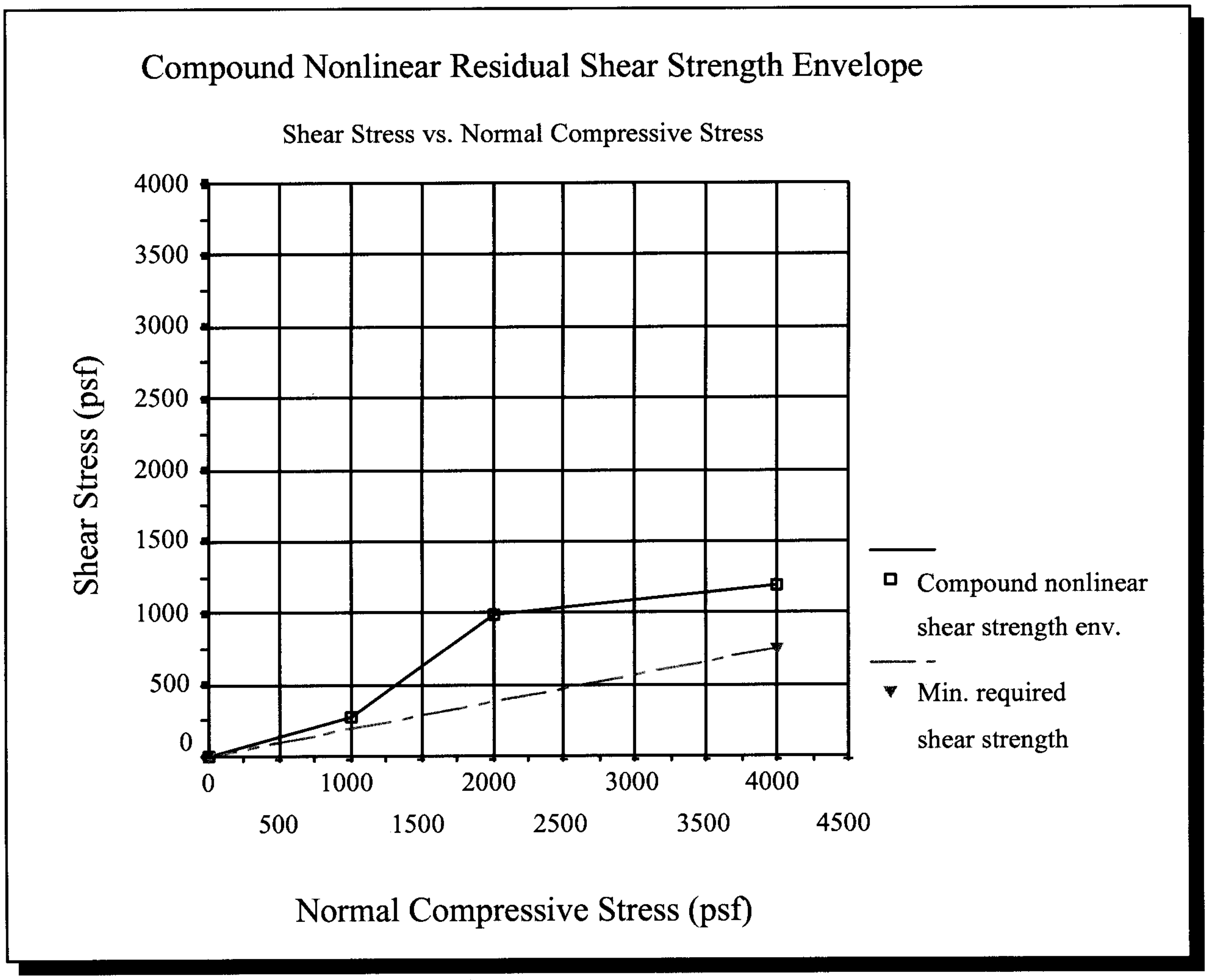

Figure 4-8 An example of a compound nonlinear *residual shear strength* envelope created from the individual nonlinear *residual shear strength* envelopes of three interfaces of a composite system. When the *residual shear strength* envelope is compared to the minimum *residual shear strength* specified in the authorizing document, it can be seen that the composite system exhibits enough *residual shear strength* at all normal compressive stresses expected at the facility that, and thus minimum required *residual shear strength* is exceeded. This ensures that all the tested materials can be used during construction of composite systems when *residual shear strength* conditions are expected. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

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CHAPTER 5

LIQUEFACTION POTENTIAL EVALUATION AND ANALYSIS

This chapter provides information to use when evaluating and analyzing the potential for failure due to liquefaction during a seismic event at an Ohio *waste containment facility*. Ohio EPA requires that the *soil units* at any *waste containment facility* be able to withstand the effects of a plausible earthquake and rule out the possibility of liquefaction. This is because it is generally expected that the engineered components of a *waste containment facility* will lose their integrity and no longer be able to function if a foundation soil layer liquefies.

> Soil liquefaction occurs in loose, *saturated* cohesionless *soil units* (sands and silts) and sensitive clays when a sudden loss of strength and loss of stiffness is experienced, sometimes resulting in large, permanent displacements of the ground. Even thin lenses of loose *saturated* silts and sands may cause an overlying sloping soil mass to slide laterally along the liquefied layer during earthquakes. Liquefaction beneath and in the vicinity of a *waste containment unit* can result in localized bearing capacity failures, lateral spreading, and excessive settlement that can have severe consequences upon the integrity of *waste containment systems*. Liquefaction-associated lateral spreading and flow failures can also affect the global stability of a *waste containment facility*.

REPORTING

This section describes the information that should be submitted to demonstrate that a facility is not susceptible to liquefaction. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report. At a minimum, the following information about the liquefaction evaluation and 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 and tabular summary of the findings of the liquefaction evaluation and analysis including all *soil units* evaluated.
- A detailed discussion of the liquefaction evaluation including:
  - A discussion and evaluation of the geologic age and origin, fines content, plasticity index, saturation, depth below ground surface, and soil penetration resistance of each of the *soil units* that comprise the *soil stratigraphy* of the *waste containment facility*,

- The scope, extent, and findings of the subsurface investigation as they pertain to the liquefaction potential evaluation.

- A narrative description of each potentially liquefiable layer, if any, at the facility, and

- All figures, drawings, or references relied upon during the evaluation marked to show how they relate to the facility.

- If the liquefaction evaluation identifies potentially liquefiable layers, then the following information should be included in the report:

    - A narrative and tabular summary of the results of the analysis of each potentially liquefiable layer,

    - Plan views of the facility that include the northings and eastings, the lateral extent of the potentially liquefiable layers, and the limits of the *waste containment unit(s)*,

    - Cross sections of the facility showing *soil units*, full depictions of the potentially liquefiable layers, and the following:

        - location of engineered components of the facility,
        - material types, shear strengths, and boundaries,
        - geologic age and origin,
        - fines content and plasticity index,
        - depth below ground surface,
        - soil penetration resistance,
        - temporal high *phreatic surfaces* and *piezometric surfaces*, and
        - in situ field densities and, where applicable, the in situ *saturated* field densities.

    - The scope, extent, and findings of the subsurface investigation as they pertain to the analysis of potentially liquefiable layers,

    - A description of the methods used to calculate the factor of safety against liquefaction,

    - Liquefaction analysis input parameters and assumptions, including a rationale for selecting the maximum expected horizontal ground acceleration,

    - The actual calculations and/or computer inputs and outputs, and

    - All figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

FACTOR OF SAFETY

The following factor of safety should be used, unless superseded by rule, when demonstrating that a facility will resist failures due to liquefaction.

Liquefaction 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 above factor of safety is appropriate, only if the design assumptions are conservative; site-specific, *higher quality data* are used; and the calculation methods chosen are shown to be valid and appropriate for the facility. It should be noted, however, that historically, occasions of liquefaction-induced instability have occurred when factors of safety using these methods and assumptions were calculated to be greater than 1.00. Therefore, the use of a factor of safety against liquefaction higher than 1.00 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 carried out 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.

Using a factor of safety less than 1.00 against liquefaction is not considered a sound engineering practice. This is because a factor of safety less than 1.00 indicates failure is likely to occur. Furthermore, performing a deformation analysis to quantify the risks and damage expected to the *waste containment facility* should liquefaction occur is not considered justification for using a factor of safety less than 1.00 against liquefaction. This is because the strains allowed by deformation analysis are likely to result in decreased performance and loss of integrity in the engineering components. Thus, any failure to the *waste containment facility* due to liquefaction is likely to be substantial and very likely to increase the potential for harm to human health and the environment. If a facility has a factor of safety against liquefaction less than 1.00, mitigation of the liquefiable layers will be necessary, or another site not at risk of liquefaction will need to be used.

> If the liquefaction analysis does not result in a factor of safety of at least 1.00, consideration may be given to performing a more sophisticated liquefaction potential assessment, or to liquefaction mitigation measures such as eliminating the liquefiable layer, or choosing an alternative site.
>
> A variety of techniques exist to remediate potentially liquefiable soils and mitigate the liquefaction hazard. Liquefaction of Soils During Earthquakes (National Research Council, Committee of Earthquake Engineering, 1985) includes a table summarizing available methods for improvement of liquefiable soil foundation conditions. However, Ohio EPA approval must be obtained prior to use of any methods for mitigation of liquefiable layers.

The *responsible party* should ensure that the designs and specifications in all authorizing documents and the quality assurance and quality control (QA/QC) plans clearly require that the assumptions and specifications used in the liquefaction 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 liquefaction 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 liquefaction 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 liquefaction analysis that uses assumptions and specifications appropriate for the change.

LIQUEFACTION EVALUATION

Ohio EPA requires the assessment of liquefaction potential as a key element in the seismic design of a *waste containment facility*. To determine the liquefaction potential, Ohio EPA recommends using the five screening criteria included in the U.S. EPA guidance document titled RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities, EPA/600/R-95/051, April 1995, published by the Office of Research and Development. As of the writing of this policy, the U.S. EPA guidance document is available at www.epa.gov/clhtml/pubtitle.html on the U.S. EPA Web site.

Recommended Screening Criteria for Liquefaction Potential

The following five screening criteria, from the above reference, are recommended by Ohio EPA for completing a liquefaction evaluation:

- Geologic age and origin. If a soil layer is a fluvial, lacustrine or aeolian deposit of Holocene age, a greater potential for liquefaction exists than for till, residual deposits, or older deposits.

- Fines content and plasticity index. Liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil. Cohesionless soils having less than 15 percent (by weight) of particles smaller than 0.005 mm, a liquid limit less than 35 percent, and an in situ water content greater than 0.9 times the liquid limit may be susceptible to liquefaction (Seed and Idriss, 1982).

- Saturation. Although low water content soils have been reported to liquefy, at least 80 to 85 percent saturation is generally deemed to be a necessary condition for soil liquefaction. The highest anticipated temporal *phreatic surface* elevations should be considered when evaluating saturation.

- Depth below ground surface. If a soil layer is within 50 feet of the ground surface, it is more likely to liquefy than deeper layers.

- Soil Penetration Resistance. Seed et al, 1985, state that soil layers with a normalized SPT blowcount $[(N_1)_{60}]$ less than 22 have been known to liquefy. Marcuson et al, 1990, suggest an SPT value of $[(N_1)_{60}]$ less than 30 as the threshold to use for suspecting liquefaction potential. Liquefaction has also been shown to occur if the normalized CPT cone resistance (q_c) is less than 157 tsf (15 MPa) (Shibata and Taparaska, 1988).

> In some cases, it is necessary to stabilize a borehole due to heaving soils. The use of hollow-stem augers or drilling mud has been proven effective for stabilizing a borehole without affecting the blow counts from a standard penetration test. Casing off the borehole as it is advanced has also been used, but it has been found that for non-cohesive soils, such as sands, it has an adverse effect on the standard penetration test results (Edil, 2002).

If three or more of the above criteria indicate that liquefaction is not likely, the potential for liquefaction can be dismissed. Otherwise, a more rigorous analysis of the liquefaction potential at a facility is required. However, it is possible that other information, especially historical evidence of past liquefaction or *sample* testing data collected during the subsurface investigation, may raise enough of a concern that a full liquefaction analysis would be appropriate even if three or more of the liquefaction evaluation criteria indicate that liquefaction is unlikely.

LIQUEFACTION ANALYSIS

If potential exists for liquefaction at a facility, additional subsurface investigation may be necessary. Once all testing is complete, a factor of safety against liquefaction is then calculated for each *critical layer* that may liquefy.

A liquefaction analysis should, at a minimum, address the following:

- Developing a detailed understanding of site conditions, the *soil stratigraphy*, material properties and their variability, and the areal extent of potential *critical layers*. Developing simplified cross sections amenable to analysis. SPT and CPT procedures are widely used in practice to characterize the soil (field data are easier to obtain on loose cohesionless soils than trying to obtain and test undisturbed *samples*). The data needs to be corrected as necessary, for example, using the normalized SPT blowcount $[(N_1)_{60}]$ or the normalized CPT. The total vertical stress (σ_o) and effective vertical stress (σ_o') in each stratum also need to be evaluated. This should take into account the changes in overburden stress across the lateral extent of each *critical layer*, and the temporal high *phreatic* and *piezometric surfaces*,

- Calculation of the force required to liquefy the critical zones, based on the characteristics of the critical zone(s) (e.g., fines content, normalized standardized blowcount, overburden stresses, level of saturation),

- Calculation of the design earthquake's effect on each potentially liquefiable layer should be performed using the site-specific in situ soil data and an understanding of the earthquake magnitude potential for the facility, and

- Computing the factor of safety against liquefaction for each liquefaction susceptible *critical layer*.

Liquefaction Potential Analysis - Example Method

The most common procedure used in practice for liquefaction potential analysis, the "Simplified Procedure," was developed by H. B. Seed & I. M. Idriss. Details of this procedure can be found in RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities (U.S.EPA, 1995). As of the publication date of this policy, the U.S. EPA guidance document was available from www.epa.gov/clhtml/pubtitle.html on the U.S. EPA Web site. Due to the expected range of ground motion in Ohio, the Simplified Procedure is applicable. However, if the expected peak horizontal ground acceleration is larger than 0.5 g, more sophisticated, truly nonlinear effective stress-based analytical approaches should be considered, for which there are computer programs available. The simplified procedure comprises the following four steps:

1. Identify the potentially liquefiable layers to be analyzed.

2. Calculate the shear stress required to cause liquefaction (resisting forces). Based on the characteristics of the potentially liquefiable layers (e.g., fines content, normalized standardized blowcount), the critical (cyclic) stress ratio (CSR_L) can be determined using the graphical methods included in the U.S. EPA guidance referenced above. Note: this determination is typically based on an earthquake of magnitude 7.5. If the design earthquake is of a different magnitude, or if the site is not level, the CSR_L will need to be corrected as follows.

$$CSR_{L(M-M)} = CSR_{L(M=7.5)} \cdot k_M \cdot k_\sigma \cdot k_\alpha \qquad (5.1)$$

where

$CSR_{L(M-M)}$ = corrected critical stress ratio resisting liquefaction,
$CSR_{L(M=7.5)}$ = critical stress ratio resisting liquefaction for a magnitude 7.5 earthquake,
k_M = magnitude correction factor,
$k\sigma$ = correction factor for stress levels exceeding 1 tsf, and
k_α = correction factor for the driving static shear stress if sloping ground conditions exist at the facility. Special expertise is required for evaluation of liquefaction resistance beneath ground sloping more than six percent (Youd, 2001).

The k-values are available from tabled or graphical sources in the referenced materials.

> The correction factors can be obtained from different sources, such as the 1995, U.S. EPA, Seismic Design Guidance, or the summary report from the 1996 and 1998 NCEER/NSF Liquefaction Workshops. The U.S. EPA document tends to be somewhat more conservative for earthquakes with a magnitude less than 6.5. In 1999, I.M. Idriss proposed yet a different method for calculating the empirical stress reduction factor (r_d), which was less conservative than the method included in the U.S. EPA guidance, but more conservative than the method included in the NCEER method. Designers should select correction factors based on site-specific circumstances and include documentation explaining their choices in submittals to Ohio EPA.

3. Calculation of the design earthquake's effect on the critical zone (driving force). The following equation can be used.

$$CSR_{EQ} = 0.65\left(\frac{a_{\max,z}}{g}\right) r_d \left(\frac{\sigma_0}{\sigma_0'}\right) \qquad (5.2)$$

where CSR_{eq} = equivalent uniform cyclic stress ratio induced by the earthquake,
σ_0 = total vertical overburden stress,
σ_0' = effective vertical overburden stress,
$a_{max,z}$ = the maximum horizontal ground acceleration, and
g = the acceleration of gravity.

Liquefaction Potential Analysis - Example Method (cont.)

$$a_{\max,z} = \left(a_{\max}\right)\left(r_d\right) \qquad (5.3)$$

where $a_{max,z}$ = the maximum horizontal ground acceleration,
a_{max} = peak ground surface acceleration, and
r_d = empirical stress reduction factor.

$$r_d = \frac{a_{\max@depth\ D}}{\sigma_{0@depth\ D}\left(\frac{a_{\max@surface}}{g}\right)} \qquad (5.4)$$

4. Calculate the factor of safety against liquefaction (resisting force divided by driving force).

$$FS_L = \frac{CSR_{L(M-M)}}{CSR_{EQ}} \geq 1.00 \qquad (5.5)$$

where FS_L = factor of safety against liquefaction,
$CSR_{L(M—M)}$ = shear stress ratio required to cause liquefaction, and
CSR_{EQ} = equivalent uniform cyclic stress ratio.

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CHAPTER 6

SETTLEMENT ANALYSES

This chapter provides information to use when analyzing the potential for failure due to settlement at an Ohio *waste containment facility*. It is important to account for settlement in the design of a *waste containment facility* because:

- *overall settlement* can result in changes to liquid drainage flow paths for leachate, surface water, or waste water, and can cause damage to pipes, destruction of geonets, and reduction or reversal of grades; and

- *differential settlement* can result in damage or failure of liner systems, piping, containment berms, and other engineered components.

> Evaluating waste and foundation settlement whenever a separatory liner will be used between old and new waste is important for determining tensile strain on components. For purposes of this policy, all references to a separatory liner will include any newly constructed separatory liner system or any previously placed cap system that will be converted to a separatory liner system.

Overall settlement and differential settlement should be analyzed for all of the following soil materials including, but not limited to: in situ soils, mine spoil, added geologic material, structural fill, recompacted soil liners, and waste materials. Differential settlement analyses should focus on areas where changes in foundation materials warrant evaluation, such as areas with high walls, separatory liner over waste, changes in *soil stratigraphy* laterally or vertically, and where significant abrupt changes in loading occur.

The vertical and lateral variability of settlement characteristics across a site, and the changes in the increase in vertical stress created by the geometry of the *waste containment facility* will cause each location of a facility to settle different amounts. The facility must be designed to account for the stresses and strains that result from settlement occurring in the foundation and waste mass.

REPORTING

> 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.

This section describes the information that should be submitted to demonstrate that a facility is not susceptible to damage from settlement. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report. At a minimum, the following information about an *overall settlement* and *differential settlement* analysis should be reported to Ohio EPA:

- A narrative and tabular summary of the results of the settlement analyses,
- A summary and a detailed discussion of the results of the subsurface investigation that apply to the settlement analyses and how they are used in the analyses,
- A summary of the approach, methodologies, and equations used to model settlement of the facility,
- If any of the settlement parameters were interpolated by using random generation or another method, then information must be provided to explain in detail, the equations and methodology, and how the settlement parameters were generated,
- Plan view maps showing the top of the liner system, the liquid containment and collection system, the location of the points where settlement is calculated, the expected settlement associated with each point, and the limits of the *waste containment unit(s).*

Drawings showing the critical cross sections analyzed. The cross sections should include the:

- *Soil stratigraphy*,
    - Temporal high *phreatic surfaces*,
    - The range of the tested settlement parameters of each layer,
    - Depth of excavation,
    - Location of engineered components of the facility that may be adversely affected by settlement,
    - The amount of settlement calculated at each point chosen along the cross section,
- The detailed settlement calculations of the engineering components,
- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility, and
- The detailed tensile strain analysis.
- If vertical sump risers are included in the facility design, then include:
    - A narrative and tabular summary of the results of the bearing capacity analysis,

> Ohio EPA discourages the use of vertical sump risers in solid *waste containment units* and hazardous *waste containment units*. This is due to the inherent difficulties they present during filling operations, and the potential they create for damaging liner systems.

- A summary and a detailed discussion of the results of the subsurface investigation that apply to the bearing capacity and how they were used in the analyses,

- A summary of the approach, methodologies, and equations used to model the bearing capacity of the facility.

DESIGN CRITERIA

Ohio EPA does not specify or recommend a factor of safety to use during settlement analysis. Instead, facilities must be designed so they satisfy applicable minimum regulatory design requirements at the time they are ready to receive waste and continue to satisfy applicable minimum design requirements after settlement is complete (at least 100% of *primary settlement* plus the *secondary settlement* expected using a time-frame of 100 years or another time-frame acceptable to Ohio EPA). This also applies to any increases in weight of the facility (e.g., vertical or horizontal expansions, increases in containment berm height). Therefore, it is important for responsible parties and designers to consider the possibility for increasing the weight of the facility and account for the additional settlement during the initial design. Failure to do so is likely to result in a facility being prevented from vertically expanding because to do so would cause the *waste containment system* or the liquid removal systems to become compromised. Applicable minimum regulatory design requirements, include, but are not limited to:

- Maintaining the minimum slopes of liners and pipes,

- Maintaining the integrity of soil berms, liners, barrier layers, and other engineered components,

- Maintaining the integrity of geosynthetics,

- Ensuring that all piping will be in working order, and

- Showing that liquids in the liquid control and collection systems will be below maximum levels allowed and otherwise meet performance standards.

> One notable exception to the requirement for designing geosynthetic systems without accounting for tensile strength of the materials is when a slip layer of geosynthetic above an FML is purposefully included in a design (see Chapter 9 for more information).

Ohio EPA requires that the tensile strength of geosynthetics are ignored when evaluating the slope stability of a facility design. This is because plastic materials creep under stress, and over time, the thickness of the geosynthetics will decrease under constant stress. Geosynthetics may crack under constant stress, and for geonets, constant stress may cause the collapse of the drainage pathways rendering the material useless. Tensile strain may occur in geosynthetics when placing the materials with too little slack, dragging subsequent layers of geosynthetic across previously placed layers during installation, placing materials such as soil, drainage material, waste, or waste water on top of the geosynthetics, and during settlement.

When tensile strain is unavoidable, the facility should be designed to minimize tensile strain in geotextiles, geomembranes, geosynthetic clay liners, geocomposite drainage layers, leachate collection piping, and waste water piping. Generally, it is recommended that strain not exceed the manufacturer's recommendations for the aforementioned components. Any design that results in geosynthetics being in strain must be accompanied with documentation and test results showing that the proposed materials will maintain the integrity of the systems of which they are a part under the calculated strain. The testing will need to represent the stress history that will be caused by the loading conditions experienced by the materials at the time of installation through final loading with waste or waste water.

The above criteria to be applied during settlement analysis are appropriate if the design assumptions are conservative; site-specific, *higher quality data* are used; and the calculation methods chosen are demonstrated to be valid and appropriate for the facility. The use of a design that is more robust than regulatory requirements 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.

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 settlement analyses 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 settlement analyses 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, administrative consent agreements).

From time to time, changes to the facility design may be needed that will alter the assumptions and specifications used in the settlement 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 settlement analysis that uses assumptions and specifications appropriate for the change request or contain a justification for why a new analysis is not necessary.

> In most cases, immediate settlement will not be a concern because the immediate settlement will occur during construction. However, immediate settlement must be taken into account at some facilities. This is especially true for facilities where construction is staged to build several phases. For example, one year, three berms and a liner system are constructed. Then the following year a large berm is constructed along the remaining upslope edge of the liner. In this instance, immediate settlement from the placement of the last berm may cause a portion of the liner to settle into a grade that does not meet design criteria. This could result in improper leachate flow or improper drainage of lagoons and ponds. Methods for analyzing immediate settlement can be found in most geotechnical and foundation textbooks (e.g., McCarthy, 2002; Holtz and Kovacs, 1981, etc).

SETTLEMENT ANALYSIS

A settlement analysis includes the *overall settlement* of a facility to ensure that pipes will remain intact and any liquid drainage flow paths for leachate, surface water, or waste water will satisfy design requirements after settlement is complete. Settlement analyses also include any *differential settlement* across a facility to ensure that engineered components will not be damaged, liquid drainage paths will be maintained, and the facility will satisfy design requirements, not only at the time of construction, but also after *differential settlement* is complete. At least two components of settlement are required to be evaluated: *primary settlement* and *secondary settlement.* The strain on engineered components created by *differential settlement* should also be calculated. Settlement is considered completed when at least 100% of *primary settlement* and the *secondary settlement* expected using a time-frame of 100 years or another time-frame acceptable to Ohio EPA is taken into account.

Due to the natural variability in soils and changes in the vertical stress across a facility, settlement characteristics and the amount of settlement are likely to be different from one point to another both vertically and laterally across a site. The variability of settlement characteristics and the changes in vertical stress due to the geometry of the *waste containment unit(s)* across a site should be discussed in detail in the summary of the subsurface investigation submitted with the settlement calculations. This discussion should describe each compressible layer found at the site, indicate if these layers exist under all or just part of the site, and discuss the extent of the variability of these layers throughout their distribution.

The vertical and lateral variability of settlement characteristics across a site and the significant damage that settlement can cause to engineered components emphasize the need for thorough and careful subsurface investigation. To facilitate a settlement analysis, it is recommended that several points be chosen along the critical cross sections of the facility and that the location of these points be spaced at a distance that would best characterize the facility depending on its size, geometry, and the variability of the soil materials at the site.

Responsible parties of *waste containment facilities* often want to expand existing facilities. This requires that a settlement analysis take into account the settlement of such things as natural foundation materials, structural fill, and waste. Estimating the settlement of structural fill, waste, and some *soil units* that are extremely variable can be difficult. This is especially true of municipal solid waste (MSW) because of the diverse mechanics occurring in the waste such as biodegradation, mechanical compression (bending, crushing, reorientation of waste caused by applied stress), and raveling (movement of fine materials into waste voids by seepage, vibration, or decomposition) (Sowers 1968, 1973). Settlement of MSW requires specialized analysis, is not well understood, and is beyond the scope of this manual. Some publications (e.g., Ling et al, 1998; Spikula, 1996; Wall and Zeiss, 1995) discuss the estimation of MSW settlement. They have been referenced at the end of this chapter.

For greenfield sites, the area within the entire footprint of each proposed *waste containment unit* must be adequately sampled (see Chapter 3). The characterization of each *compressible layer*, both vertically and laterally, is then used to calculate the expected settlement at points along any flow line or for any portion of the facility.

When a settlement analysis is being conducted for an existing *waste containment facility* where borings cannot be placed within the limits of waste placement, the variability in the soil profile of the *compressible layers* under the existing facility can be estimated by using the settlement characteristics from adjacent *borings* outside the limits of waste placement or borings performed prior to the existing waste placement.

For MSW landfills, when a separatory liner system is placed between existing waste and new waste, it must be placed at a minimum ten percent slope in all areas except along flow lines augmented by

> Many engineered components of modern *waste containment facilities* may fail if subjected to *differential settlement* which increases strain on piping and liner system components. Because of this, *responsible parties* may want to consider using additional sampling methods such as cone penetrometers or seismic refraction to gather as much detailed data as possible to accurately delineate the subsurface characteristics of each type of soil material.

leachate collection pipes or at some other slope based on a design acceptable to the director. Other types of facilities may wish to incorporate this into their designs. Nevertheless, it is recommended that all facilities with separatory liner systems not only analyze the foundation soils underlying the *waste containment facility* for settlement, but also analyze the settlement of the waste underlying the separatory liner. The analysis should verify that the leachate collection and management system portion of the separatory liner maintains drainage and the separatory liner system components maintain integrity throughout the life and post closure of the facility or longer as determined by Ohio EPA regulations.

When doing this type of analysis, the variability of the settlement parameters for the existing waste and the foundation under the waste needs to be taken into account. A method that can be used to determine settlement is to assign randomly varied values of settlement characteristics to the waste and the soil materials underlying the existing *waste containment facility.* The settlement characteristics should be varied both vertically and laterally for the waste. The variation of the *compressible layers* can be considered by varying the values of the compression index (C_c) and the initial void ratio (e_0) in a reasonable range. The range of values representing soil materials can be based on the results from the *higher quality data* retrieved from *borings* that surround the existing facility. Book values and/or *higher quality data* retrieved from waste samples or test fills can be used for the values representing waste.

Settlement should be calculated along as many cross sections as are necessary to ensure that the expected amount of *overall* and *differential settlement* that will be experienced by the engineered components of the facility has been adequately estimated. If it is discovered that *overall* and *differential settlement* along any cross section will likely cause damage to an engineered component, or cause the engineered component to be unable to meet the minimum design criteria, then the facility must be redesigned to eliminate the adverse effects of *overall* and *differential settlement* through methods such as overbuilding, surcharging, removal of the material causing the problem, or engineered reinforcement.

Overall Settlement Analysis

When calculating the overall settlement for a facility, points of settlement should be located along the length of critical liquid drainage flow paths and especially at points where increased settlement may occur. Points chosen along the pathways should be evaluated for each compressible layer below the bottom of the facility and the vertical stress being applied above these points. One approach may be to select a range for each settlement parameter for each compressible layer using the sampling and testing procedures outlined in Chapters 3 and 4. The range of the parameters should then be utilized in such a manner as to create the worst-case scenario for primary and secondary settlement of the chosen flow path. Less settlement occurs at a point when the values for C_c, C_r (recompressive index), and C_a (secondary compression index) are at the lowest end of their respective ranges and σ_p' and e_0 are at the highest end. The opposite is true of the reverse, and a combination will yield a value between these two extremes. These aspects of the calculations should be considered when determining the settlement along the flow path. The input parameters used in these calculations should be conservative and based on site specific concerns. Once the expected settlement is determined for each point, the slope between the points on the flow paths can be determined. The resulting slopes must meet any regulatory minimums for drainage slopes and/or maintain drainage in the proper direction.

It is important to clearly understand the assumptions and limits of any given method for determining the increase in vertical stress and expected settlement because many methods will not be applicable to *waste containment facilities*. For example, according to Civil Engineering Reference Manual by Lindeburg, Boussinesq's equation applies only to small footings compared to the depth of interest.

Differential Settlement Analysis

Differential settlement can occur due to many factors. Typically, *differential settlement* is a result of variable materials underlying the facility, such as areas of highly compressible material adjacent to less compressible material. These transitional areas should be thoroughly investigated and sampled during the geologic investigation (see Chapter 3 for more information). Then, a critical cross section should be determined across the transition of the two materials. *Differential settlement* may also occur where abrupt changes in loading have been applied to the facility. Cross sections should be analyzed across the loading transition. *Differential settlement* also occurs at locations of mine highwalls or where vertical risers have been incorporated into the liquids management system design. It is recommended in the area of mine highwalls that the settlement analysis incorporate two-dimensional stress distribution theory to verify that the waste containment facility and liquid drainage pathways will not be compromised by the differential settlement. In the case of vertical risers, a bearing capacity analysis is the appropriate calculation to be performed.

Strain

After *overall* and *differential settlement* analyses have been performed, the engineered components of the *waste containment facility*, such as geotextiles, geomembranes, geosynthetic clay liners, geocomposite drainage layers, leachate collection piping, and waste water piping, should be analyzed for tensile strain. The analysis should verify that the engineering components can maintain their integrity when subjected to the induced strain due to the settlement determined in the *overall* and *differential settlement* analyses. The analysis should also include a discussion of the predicted strain compared to the manufacturers' specifications for allowable strain in the products proposed for use at the facility.

Determining Settlement and Strain

The first step of calculating expected settlement (*overall* and *differential*) is to calculate the initial effective vertical stress (σ_o' = total vertical stress - pore water pressure) and the change in the effective vertical stress ($\Delta\sigma_o'$) caused by the facility on a point of interest in the underlying materials. The values added together are the effective vertical stress ($\sigma_o' + \Delta\sigma_o'$) exerted upon the materials that will cause settlement. When calculating effective vertical stress in situations where no differential settlement will occur, a one-dimensional approximation of the settlement may be used. This can be accomplished by calculating the weight of the material directly above the point of interest. When calculating the effective vertical stress where strain may be developed due to differential settlement, a two-dimensional stress distribution theory should be used. Once σ_o' and $\Delta\sigma_o'$ have been calculated, a typical settlement analysis would be performed using the following:

Primary Settlement (Sc)

Primary settlement, also known as *primary consolidation* settlement (Sc), is the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to a squeezing out of water from the void spaces of the mass and accompanied by a transfer of the load from the soil water to the soil solids (ASTM D 653). The rate of settlement is controlled by the permeability of the soil. As a result, in higher permeability cohesionless soils, the settlement occurs rapidly, and in lower permeability cohesive soils, the process is gradual.

The following equation is used to estimate the *primary settlement* in normally consolidated clays or loose granular materials:

$$S_c = \left(\frac{C_c}{1+e_0}\right)\cdot H\cdot\log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_0'}\right) \qquad (6.1)$$

where H = thickness of the layer after excavation to be evaluated,
C_c = primary compression index,
e_0 = initial void ratio,
σ_0' = effective vertical stress at the middle of the layer after excavation, but before loading, and
$\Delta\sigma_0'$ = increase or change in effective vertical stress due to loading.

The following equation is used to estimate the consolidation settlement in overconsolidated clays. Dense cohesionless materials do not settle significantly and thus, do not have to be evaluated using this equation.

$$S_c = \left(\frac{C_r}{1+e_0}\right)\cdot H\cdot\log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_0'}\right) \qquad (6.2)$$

where C_r = recompressive index.

If the increase in vertical stress at the middle of the consolidation layer is such that ($\sigma_0' + \Delta\sigma_0'$) exceeds the preconsolidation pressure (σ_p') of the consolidating layer, the following equation should be used:

$$S_c = \left[\left(\frac{C_r}{1+e_0}\right)\cdot H\cdot\log\left(\frac{\sigma_p'}{\sigma_0'}\right)\right] + \left[\left(\frac{C_c}{1+e_0}\right)\cdot H\cdot\log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_p'}\right)\right] \qquad (6.3)$$

The increase in vertical stress is caused by the application of a surcharge to the consolidating layer. Usually the engineered components and waste of a facility will be the surcharge. The entire vertical stress that will be induced at the middle of each consolidating layer should be used in the calculations. This vertical stress typically corresponds to the maximum weight of the facility (e.g., when a solid waste facility is at its maximum waste height, or a waste water lagoon is operating at minimum freeboard).

Ohio EPA stresses the use of laboratory data to determine the various inputs for the settlement equations. ASTM D 2435-03 describes methods to determine σ_p' and e_0 from laboratory data. Although not directly indicated in the standard, C_c can also be obtained from the same diagram that σ_p' is obtained. C_c is the slope of the virgin compression curve (i.e., the line that ends with "F" from Fig. 4 of the ASTM standard). C_r is obtained from a diagram for overconsolidated soils, where C_r is the slope of the recompression curve (see **Figure 6-1** on page 6-9).

<u>Secondary Settlement (S_s)</u>

Secondary settlement, also known as creep, is the reduction in volume of a soil mass caused by the application of a sustained load to the mass and due principally to the adjustment of the internal structure of the soil mass after most of the load has been transferred from the soil water to the soil solids (ASTM D 653). Due to the absence of pore water pressure, the solid particles are being rearranged and further compressed as point-to-point contact is experienced.

Secondary settlement can be calculated using the following equation:

$$S_s = \frac{C_\alpha}{1 + e_p} \cdot H \cdot \log\left(\frac{t_s}{t_{pf}}\right) \qquad (6.4)$$

where C_a = *secondary compression* index of the *compressible layer*,
H = thickness of the layer to be evaluated after excavation, but before loading
t_s = time over which *secondary compression* is to be calculated (use 100 years plus the maximum time it will take to complete *primary consolidation* under the facility unless some other time frame is acceptable to Ohio EPA for a specific facility), and
t_{pf} = time to complete *primary consolidation* in the consolidating layer in the field, and
e_p = the void ratio at the time of complete *primary consolidation* in the test specimen of the *compressible layer*.

Both t_s and t_{pf} must be expressed in the same units (e.g., days, months, years).

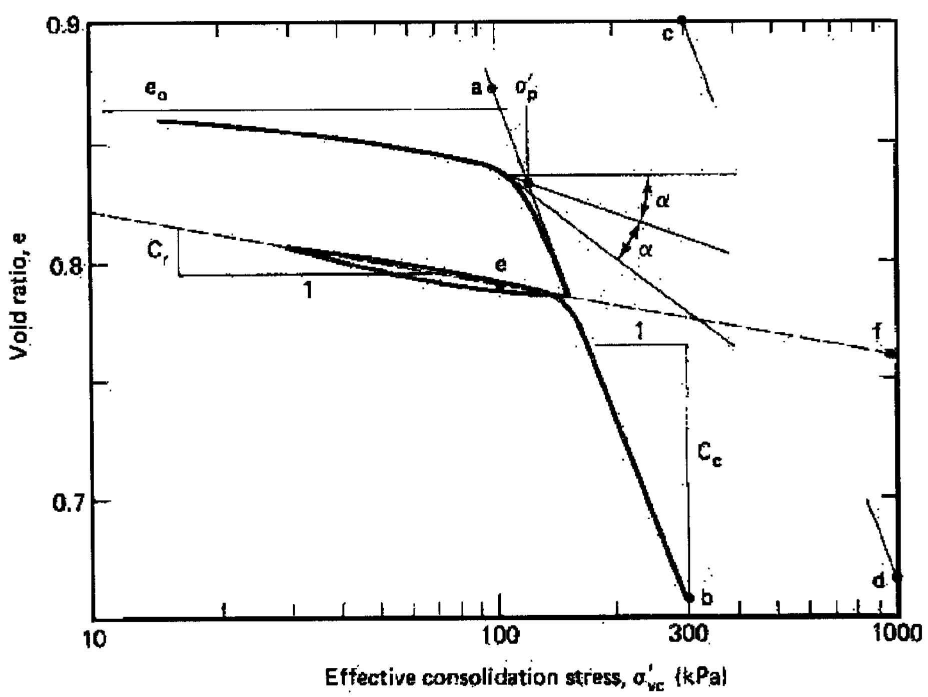

Figure 6-1 Overconsolidated stress diagram. From Ex. Figure 8.9, Holts and Kovacs, pp. 316

The values for e_p and C_a are determined graphically, such as from a void ratio - log time curve as shown in **Figure 6-2**. The value of C_a is the slope of the plot exceeding 100 percent primary consolidation or t_p in **Figure 6-2**.

The value for t_p shown in **Figure 6-2** is the time to complete primary consolidation for the *specimen.* The value of t_p which is needed in equation 6.4, is the field value for t_p. Therefore, t_p (referred to as t_{pf}) should be determined from the following equation to best represent a field value for t_p.

$$t_{pf} = \frac{T_v \cdot H_t^2}{C_v} \qquad (6.5)$$

where H_t = maximum length of drainage in the consolidating layer so that H_t is the full thickness of the consolidating layer if it is drained on one side (top or bottom), and H_t is one-half of the thickness of the consolidating layer if it is drained on both sides (top and bottom),

t_{pf} = time to complete *primary consolidation* in the consolidating layer in the field (years),

C_v = coefficient of consolidation (converted to ft^2/year or m^2/year as appropriate), and

T_v = a dimensionless time factor associated with the time it takes for primary settlement to be completed (see discussion below for more information).

C_v can be determined from one of the methods described in ASTM D 2435-03.

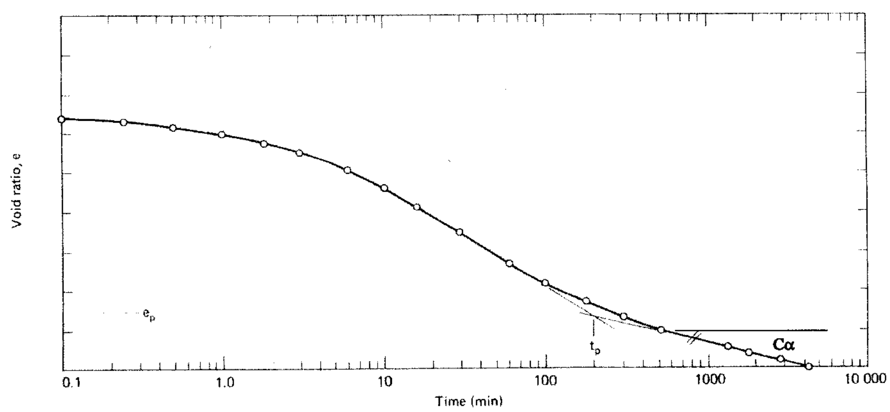

Figure 6-2 Graphical determination of e_p and Cα. Adapted from Figure Ex. 9.10b, Holtz and Kovacs, pp 412.

The dimensionless time factor (T_v) has a theoretical relationship with the percent of primary consolidation (U%) that can be expressed by the following two equations:

For U<60% $$T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \qquad (6.6)$$

For U>60% $$T_v = 1.781 - 0.933\log(100 - U\%) \qquad (6.7)$$

Plotting these two equations produces the chart solution of Terzaghi's theory of consolidation. Because the equation produces an asymptotic line, Ohio EPA recommends deriving Tv using U% = 99.999 for most facilities. This results in a Tv = 4.58.

Although Ohio EPA recommends a laboratory determination of the above inputs, many can be derived from various charts found in engineering textbooks and manuals used across the country such as the U.S. Army Corps of Engineers manual 1110-1-1904 (September 30, 1990). Some of these charts use a correlation between other inputs or field/lab data, such as blow counts and liquid limits. If charts are used in the settlement analysis, their applicability should be validated with correlations to laboratory data, and the analysis should include a description of how the use of the information from the charts is appropriate with respect to the material represented.

Strain

Once settlement has been calculated for each settlement point, the strain that will occur between each adjacent point can be calculated. The strain can be estimated by using the following equation:

$$E_T(\%) = \frac{L_f - L_0}{L_0} \bullet 100 \qquad (6.8)$$

where E_T = tensile strain,
L_0 = original distance separating two location points, and
L_f = the final distance separating the same two points after settlement is complete.

Primary Consolidation - Example Calculations

An example of calculating the primary settlement for clay is illustrated using a landfill that has a maximum excavation of 30 ft and a maximum waste depth of 210 feet over a 50-foot thick overconsolidated clay layer underlain by a 40-foot thick dense gravel layer. The settlement of the dense gravel layer would not be calculated because significant settlement is not likely due to its density. To be conservative, all the clay is assumed to be saturated. Any amount of immediate settlement is likely to be compensated for during construction. Oedometer tests on undisturbed specimens from three borings provided the following range of values: preconsolidation pressure (σ_p') = 3,900 psf - 4,000 psf, C_c = 0.152 - 0.158, C_r = 0.023 - 0.026, e_0 = 0.4797 - 0.4832, C_v = 0.0240 - 0.0250, $C\alpha$ = 0.0129 - 0.0134, and e_p = 0.0866 - 0.0867. The field saturated unit weight of the clay is typically 135 pcf. Because this clay layer will be recompacted for bottom liner, we will assume that the liner will have the same settlement parameters. Six of the points of concern for settlement in this example are shown in **Figure 6-3**:

For this example, settlement will be analyzed for only points #1 through #6. The average initial effective overburden pressure at the center of the clay layer σ_o'=3,375 psf. Because σ_p' > σ_o', the in-situ clay is overconsolidated. Since σo' + $\Delta\sigma_o$' > σ_p', equation 6.3 will be used. The increase in vertical stress ($\Delta\sigma_0$') at points #1 through #6 will be determined using a one-dimensional stress distribution analysis.

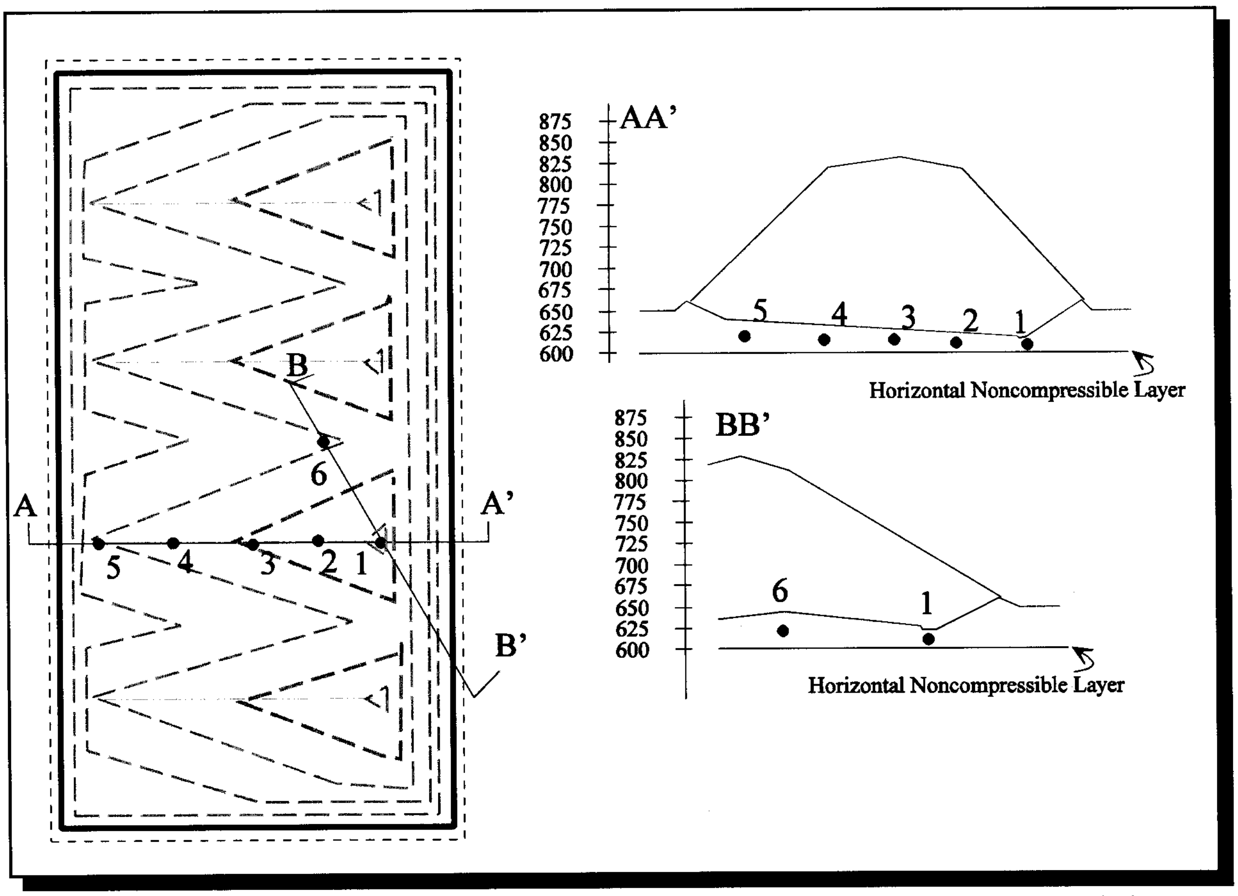

Figure 6-3 Example plan view and cross sections showing some of the locations selected for settlement analysis.

Primary Consolidation - Example Calculations (cont.)

$$S_c = \left[\left(\frac{C_r}{1+e_0}\right)\cdot H \cdot \log\left(\frac{\sigma_p'}{\sigma_0'}\right)\right] + \left[\left(\frac{C_c}{1+e_0}\right)\cdot H \cdot \log\left(\frac{\sigma_0' + \Delta\sigma_0'}{\sigma_p'}\right)\right]$$

Point	Top of Gravel	Top of Liner	Top of LDF	H (ft)	σ_o' (psf)	Load Height (ft)	$\Delta\sigma_o'$	C_c	C_r	e_o	σ_p'	S_c (ft)
1	600	619	732	19	1283	113	8475	0.152	0.023	0.4832	4000	0.8996
2	600	624	820	24	1620	196	14700	0.158	0.026	0.4797	3900	1.7540
3	600	629	830	29	1958	201	15075	0.158	0.026	0.4797	3900	2.1350
4	600	635	820	35	2363	185	13875	0.158	0.026	0.4797	3900	2.4489
5	600	640	725	40	2700	85	6375	0.158	0.026	0.4797	3900	1.6788
6	600	641	820	41	2768	179	13425	0.158	0.026	0.4797	3900	2.8140

$$t_{pf} = \frac{T_v \cdot H_t^2}{C_v} \qquad S_s = \frac{C_\alpha}{1+e_p}\cdot H \cdot \log\left(\frac{t_s}{t_{pf}}\right)$$

Point	H_t (ft)	t_s (yr)	C_v @T_{90} (in²/min)	t_{pl} (yr)	e_p	C_α	S_s (ft)
1	19	559.3	0.0250	459.2722	0.0867	0.0129	0.019
2	24	863.3	0.0240	763.3333	0.0866	0.0134	0.016
3	29	1215.0	0.0240	1114.5197	0.0866	0.0134	0.013
4	35	1723.0	0.0240	1623.4086	0.0866	0.0134	0.011
5	40	2220.4	0.0240	2120.3704	0.0866	0.0134	0.010
6	41	2328.0	0.0240	2227.7141	0.0866	0.0134	0.010

Primary Consolidation - Example Calculations (cont.)

	Top of Liner	Primary Settlement S_c (ft)	Secondary Settlement S_s (ft)	Top of Liner after Settlement	Length (ft)	Initial Slope (%)	Final Slope (%)
1	619	0.8996357	0.019296	618.08107			
					500	1.0%	0.8%
2	624	1.7498684	0.015824	622.23431			
					500	1.0%	0.9%
3	629	2.1350133	0.013346	626.85164			
					600	1.0%	0.9%
4	635	2.4489316	0.011205	632.53986			
					500	1.0%	1.2%
5	640	1.6788209	0.00987	638.31131			
1	619	0.8996357	0.019296	618.08107	1000	2.2%	2.0%
6	641	2.813968	0.00964	638.17639			

The resulting strain between the points can be estimated using Equation 6.8.

$$E_T(\%) = \frac{L_f - L_0}{L_0} \bullet 100$$

Point	Top of Liner	X Coordinate	Top of Liner after Settlement	Original Length (ft)	Length after Settlement	E_T (%)
1	619	0	618.1			
				500.025	500.017	0.00%
2	624	500	622.2			
				600.0208	600.018	0.00%
3	629	1000	626.9			
				500.036	500.032	0.00%
4	635	1600	632.5			
				500.025	500.033	0.00%
5	640	2100	638.3			
1	619	0	618.1	1000.242	1000.202	0.00%
6	641	950	638.2			

Considerations for Mine Spoil

> Although this section is specifically tailored to address mine spoil, the techniques described herein may be applicable to other types of foundation materials susceptible to *differential settlement.*

The potential damage caused by settlement of engineered components by constructing across an existing highwall/mine spoil interface (see **Figure 6-4**) or a buried valley can be considerable. A highwall is the edge of the quarry and the transition point from existing *bedrock* to the mine spoil used to fill the quarry area. This transition point presents a sharp contrast between the compressible mine spoil and rigid highwall that can result in severe tensile stress from *differential settlement.* The increase in tensile stress in the engineered components installed across the highwall/mine spoil interface is determined by estimating the mine spoil settlement and assuming that the highwall will not settle. This creates a conservative estimate of the *differential settlement* across the highwall/mine spoil interface that can then be used to determine the strain on engineered components.

Several alternatives can be considered to reduce the tensile stress created by *differential settlement* upon engineered components at the highwall/mine spoil interface. One alternative is cutting back the highwall to increase the length over which the *differential settlement* will occur. This will reduce the tensile strain because the *differential settlement* is occurring over a longer length rather than at the vertical highwall/mine spoil interface. This could involve excavating the *bedrock* of the highwall to create a grade sloping away from the mine spoil and placing fill in the excavation to reduce the effects of the difference in compressibility of the two materials.

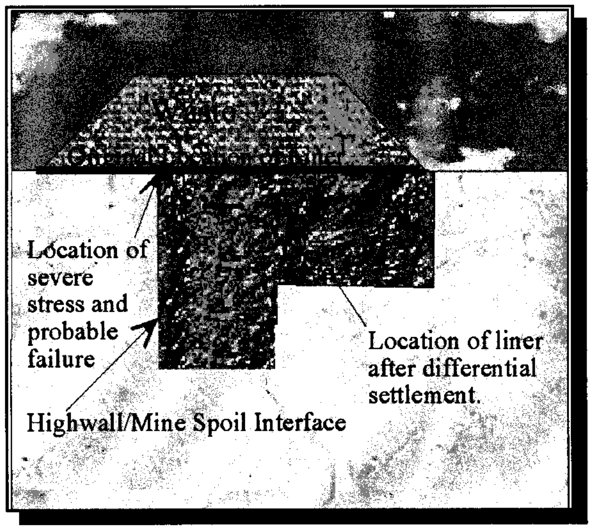

Figure 6-4 Example of failure point at a highwall/mine spoil interface.

A second alternative is to surcharge the mine spoil to cause a large portion of the settlement of the mine spoil to occur before constructing any engineered components across the high tensile stress area. The surcharge should be applied using a significant percentage of the proposed weight to be placed over the highwall. Thus, when the surcharge is removed, less settlement will occur when the facility is constructed, which should reduce the tensile strains in the engineered components. This alternative can be undertaken in conjunction with cutting back the highwall.

A third alternative, tensile reinforcement using geogrids or geotextiles, might be suitable in some rare cases for bridging the highwall/mine spoil transition. However, the use of tensile reinforcements will require sufficient anchorage on both ends to generate the tensile forces necessary to resist settlement.

Whenever an engineered solution is proposed for use to eliminate or mitigate *differential settlement,* detailed calculations and a design proposal must be submitted to Ohio EPA for approval. This usually occurs as part of a permit application or other request for authorization. The submittal must demonstrate the long-term effectiveness of the engineered solution and include a proposed plan for monitoring the effectiveness of the solution or provide a justification that long-term monitoring is not warranted.

The orientation of engineered components (e.g., geomembrane seams) should also be considered. Engineered components in the mine spoil area should be oriented so that the tensile strain that develops because of *differential settlement* will be directed away from stress sensitive engineered components. For example, the seams of geosynthetics should be installed perpendicular to a mine spoil/highwall interface, rather than parallel to it.

BEARING CAPACITY

Although the design of a *waste containment facility* is governed mostly by the results of the slope stability and settlement analyses, bearing capacity should be addressed. The analyses of bearing capacity and settlement are interrelated because they rely upon the same subsurface investigation data, use similar calculations for determining the increase in vertical stress created upon the foundation materials by the facility, and are similarly affected by the geometry of the facility. Designing a facility to account for induced settlements usually addresses all concerns except when the entire *waste containment facility* is underlain by a nonrigid foundation such as soft clays; has vertical leachate sump risers in the design; or contains stabilized waste. After a successful settlement analysis of the facility has been performed, a bearing capacity analysis of the facility over the nonrigid foundation; vertical riser; or stabilized waste relative to equipment travel during operations and after closure should be conducted.

> Stabilized waste is defined as any waste, such as sludge or pickle liquors, that must be blended with another material to generate the strength necessary to bear the weight of objects or other materials. *Responsible parties* may need to stabilize the waste and/or contaminated soils being disposed to provide support for a cap and equipment. It is recommended that the unconfined compressive strength of the stabilized waste and/or contaminated soil be at least 15 psi. If this amount of compressive strength cannot be made available at the time of construction, it is important that the *responsible party* ensure that the waste will increase in strength over time and has adequate strength to support construction and maintenance activities. For the short-term, the waste should be capable of supporting the combined weight of the cap with a heaviest piece of construction equipment. This can be demonstrated by having a factor of safety against bearing capacity failure of at least 2.0 or greater using the heaviest piece of construction equipment. For the long term, the waste should be able to support the weight of the cap and the heaviest piece of maintenance equipment once construction is complete. This can be demonstrated by having a factor of safety against bearing capacity failure of at least 3.0 using the heaviest piece of maintenance equipment.

Reporting of the bearing capacity analysis would include the same elements as the settlement analysis with the addition of a description of any downdrag forces and the assumptions associated with those forces used in the bearing capacity analysis.

Three modes of bearing capacity failures exist that may occur under any foundation. They are general shear, punching shear, and local shear (see **Figure 6-5** on Page 6-17). Designers should evaluate all potential bearing failure types for applicability to their facility design, especially if vertical sump risers are included in the design. Ohio EPA discourages the use of vertical sump risers in solid waste and hazardous *waste containment units* due to the inherent difficulties they present during filling operations, and the potential they create for damaging underlying liner system. They also pose a risk to the integrity of the *waste containment system* if they are not designed properly. The size and stiffness of the foundation slab are critical. If the slab is not large enough in area, and is not stiff enough to prevent deflection under the expected load, then excessive settlement or a bearing capacity failure could occur. This would likely breach the *waste containment system* at one of its most critical points. Also, it is not recommended that geosynthetic clay liners be installed beneath vertical sump risers due to the likelihood of the bentonite squeezing out from beneath the foundation slab.

The following factor of safety should be used, unless superseded by rule, when demonstrating that a facility is designed to be safe against bearing capacity failures.

Bearing Capacity Analysis: FS $\geq$ 3.0

> The number of digits after the decimal point indicates that rounding can only occur to establish the last digit. For example, 1.57 can be rounded to 1.6, but not 2.0.

Using a factor of safety less than 3.0 against bearing capacity failure for long-term loading situations is not considered a sound engineering practice in most circumstances. This is due to the many large uncertainties involved when calculating bearing capacity. The factor of safety is also high, because any failure of the *waste containment facility* due to a bearing capacity failure is likely to increase the potential for harm to human health and the environment. If a vertical sump riser has a factor of safety against bearing capacity failure less than 3.0, the following alternatives can be considered: elimination of the vertical sump riser in favor of a side slope sump riser, removal of soil layers susceptible to a bearing capacity failure, or redesigning the vertical sump riser to be within the bearing capacity of the soils. In the case of stabilized waste, if the factor of safety is less than 3.0, the waste must be reprocessed to meet the stability requirement. If a bearing capacity analysis of a facility over soft clays is less than 3.0, then the facility will need to be redesigned or the soil layers susceptible to a bearing capacity failure removed.

> 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.

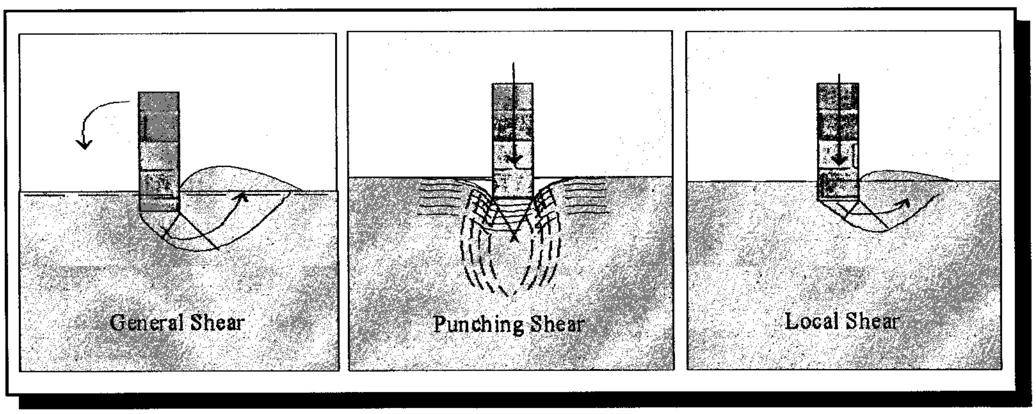

Figure 6-5 The three modes of bearing failures.

State and local building departments require permits before constructing and using any structure, such as storage tanks, scale houses, or office buildings. The building departments require bearing capacity analysis and settlement analysis as part of the permit process for these types of structures. Ohio EPA expects that the *responsible party* will comply with all building and occupancy requirements for these types of peripheral structures. Therefore, although these types of structures are often defined as being a part of a *waste containment facility*, Ohio EPA will not review the bearing capacity or settlement calculations for these types of structures.

The factor of safety against bearing capacity failure is calculated as follows:

$$FS_b = \frac{q_{ult}}{p_{total}} \geq 3.0 \qquad (6.9)$$

where FS_b = factor of safety against bearing failure,
q_{ult} = ultimate bearing capacity of the foundation soils, and
p_{total} = the total pressure applied to the base of a foundation by an overlying mass.

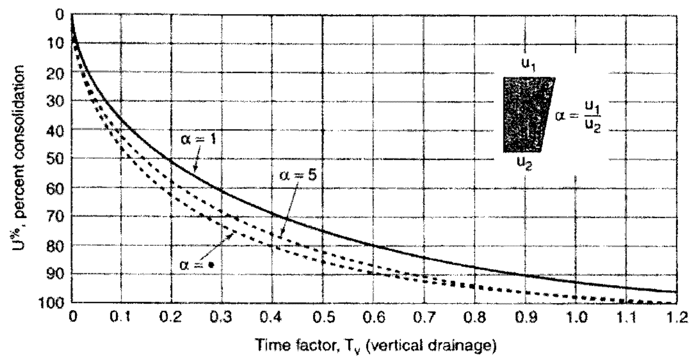

Figure 10-23 Variation of time factor T_v with percentage of consolidation U.

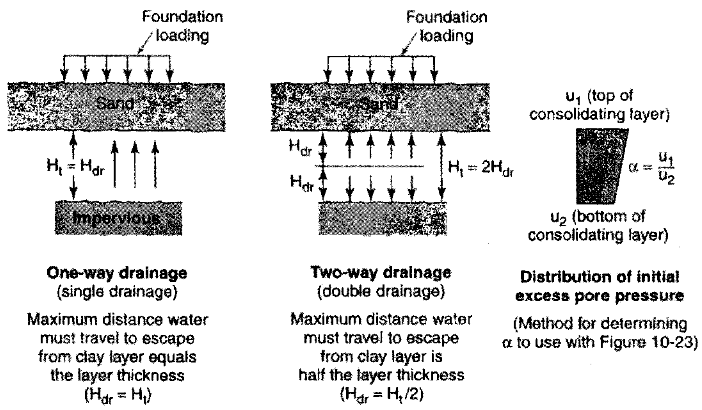

Figure 10-24 Vertical drainage conditions in consolidation theory.

Figure 6-6 This set of figures and the chart can be used for determining the time factor (T_v) for settlement and identifying the drainage path length (H_{dr}). Determining T_v for U%>95 can be calculated using: T_v=1.781-0.933log(100-U%) Source: McCarthy, 2002, Page 383.

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CHAPTER 7

HYDROSTATIC UPLIFT ANALYSIS

This chapter provides information to use when analyzing the hydrostatic uplift potential at a *waste containment facility* in Ohio. Hydrostatic uplift may affect the subbase or engineered components of a *waste containment facility* anytime ground water exists at a facility. When an excavation or a portion of a *waste containment facility* will be constructed at a depth where a *phreatic surface* of ground water is present or piezometric pressures are present, the potential adverse effects upon the *waste containment facility* will need to be taken into account.

> When the ground water head is sufficiently high, pressure may cause soil layers affected by the pressure to lose strength and fail. It is widely accepted that the effective stress created by a soil mass is the main factor that determines the engineering behavior of that soil. According to Terzaghi et al, 1996, total stress in soil is a sum of an effective stress (or intergranular stress as a result of particle-to-particle contact pressure) and a neutral stress (pore water pressure). At the instance of failure, total stress in the soil is equal to the pore water pressure, and the effective stress is equal to zero. In other words, when particle-to-particle contact disappears, so does the soil's strength.

The discussion in this chapter assumes that hydrostatic uplift occurs when enough water pressure builds to simply lift a soil layer or flexible membrane liner (FML). Although this may be a common case, other possible mechanisms of soil disruption exist under hydrostatic uplift forces. Some of them are roofing, boiling, or even a uniform heave throughout the soil mass without formation of a large blister. The mechanism that develops is controlled mainly by soil characteristics and construction practices. Details on these mechanisms are given in literature and are beyond the scope of this policy.

REPORTING

This section describes the information that should be submitted to demonstrate that a facility is not susceptible to hydrostatic uplift. 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 and tabular summary of the results of the hydrostatic uplift analysis,

- A summary and discussion of the results of the subsurface investigation that apply to hydrostatic uplift analysis and how they were used in the analysis,

- A summary of the worst-case scenarios used to analyze the hydrostatic uplift potential of the facility,

- Isopach maps comparing the excavation and construction grades, depicting the temporal high *phreatic* and *piezometric surfaces* and showing the limits of the *waste containment unit(s)*,

Figure 7-1 Hydrostatic pressure can cause in situ materials to fracture and allow the passage of the underlying ground water into an excavation, causing flooding of the excavation and weakening the in situ materials. Note the two delta formations in the above picture that are obvious evidence of flow through the in situ materials, which at this Ohio landfill, are over 20 feet thick.

- Drawings showing the cross sections analyzed. The cross sections should include:

    1. the engineered components and excavation limits of the facility
    2. the *soil stratigraphy*,
    3. the temporal high *phreatic* and *piezometric surfaces*, and
    4. the field densities of each layer.

- The detailed hydrostatic uplift calculations, and

- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

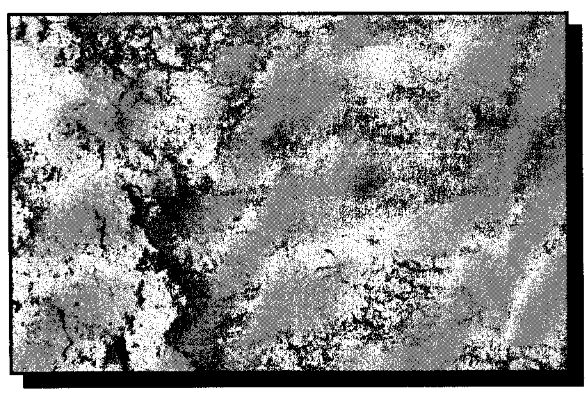

Figure 7-2 Hydrostatic pressures are causing ground water to pipe into an excavation of an Ohio landfill. This may have been caused by fracturing of the in situ materials, piping, or from an improperly abandoned *boring*.

FACTOR OF SAFETY

The following factor of safety should be used, unless superseded by rule, when demonstrating that a facility will resist hydrostatic uplift.

Hydrostatic Uplift Analysis: FS $\geq$ 1.40

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 a higher factor of safety against hydrostatic uplift 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 *phreatic* or *piezometric surfaces*, 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 failures due to hydrostatic uplift. A factor of safety against hydrostatic uplift lower than 1.40 is not considered a sound engineering practice in most circumstances. This is due to the uncertainties in calculating a factor of safety against hydrostatic uplift, and any failure of the *waste containment facility* due to hydrostatic uplift is likely to increase the potential for harm to human health and the environment. If a facility has a factor of safety against hydrostatic uplift less than 1.40, mitigation of the hydrostatic uplift pressures, redesigning the facility to achieve the required factor of safety, or using another site not at risk of a failure due to hydrostatic uplift will be necessary.

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 geometry of the worst-case location for hydraulic uplift is unique to one phase, it is a small portion of the phase, pumping of water out of the *saturated soil unit* or *bedrock* can be done to alleviate hydrostatic uplift pressure, and the area can be excavated, constructed and buried by sufficient waste or fill material during the same construction season so that failure of the engineered components will be prevented, then the *responsible party* may propose (this does not imply approval will be granted) to use a lower factor of safety against hydrostatic uplift in the range of 1.4 to 1.2. The proposal should include any pertinent information necessary for demonstrating the appropriateness of the lower factor of safety to the facility.

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 hydrostatic uplift 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 hydrostatic uplift 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).

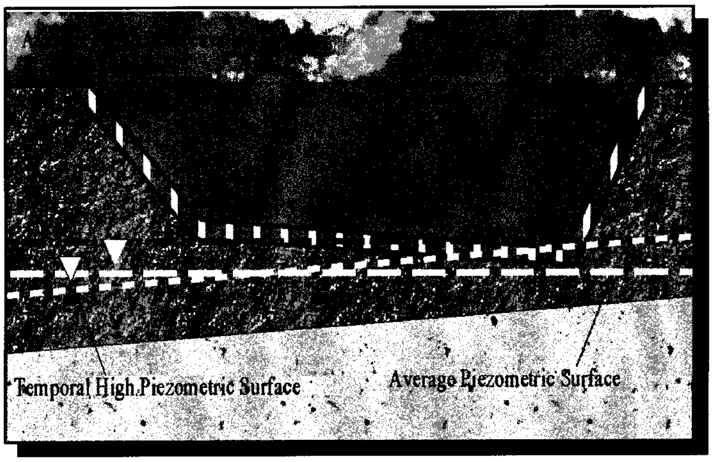

Figure 7-3 Example of how using the average depth of excavation (double-dot dashed line) and the average elevation of the *piezometric surface* (large dashed line) result in the conclusion that hydrostatic uplift will not occur, which is incorrect. Note that the temporal high *piezometric surface* (small dashed line) does intersect the liner system (hashed area) creating the potential for hydrostatic uplift that must be analyzed.

From time to time, changes to the facility design may be needed that will alter the assumptions and specifications used in the hydrostatic uplift 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 hydrostatic uplift analysis that uses assumptions and specifications appropriate for the change request.

ANALYSIS

When selecting the scenarios for analysis of hydrostatic uplift, it must be ensured that the worst-case interactions of the excavation and of the construction grades with the *phreatic* and *piezometric surfaces* are selected. Temporal changes in *phreatic* and *piezometric surfaces* must be taken into account. The highest temporal *phreatic* and *piezometric surfaces* must be used in the analysis. Using average depth of excavation or average elevation for the *phreatic* and *piezometric surfaces* is not acceptable (see **Figure 7-3**). The purpose of the analysis is to find all areas of the facility, if any, that have a factor of safety less than 1.40 for hydrostatic uplift.

Figure 7-4 This is another example of hydrostatic pressures at an Ohio landfill creating flow through more than 20 feet of heavy in situ clay materials causing flooding of the excavation.

Figure 7-5 illustrates a situation where a clay liner (or another soil layer) is constructed above a *saturated* layer. The piezometric head (H_P) is applying upward pressure on the liner.

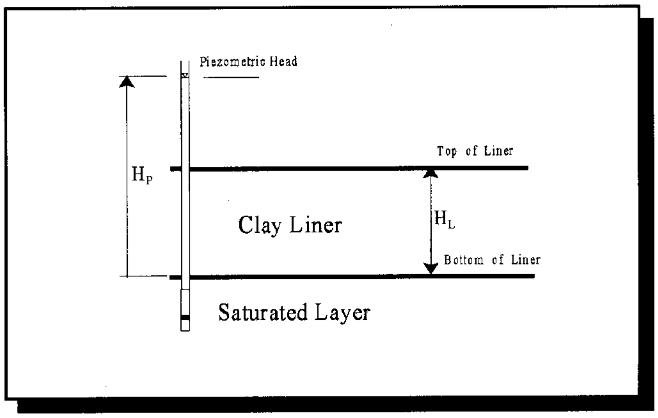

Figure 7-5 An example of piezometric head from ground water exceeding the top of an engineered component or soil layer creating a potential for hydrostatic uplift .

If γ_L = field density of clay liner,
γ_W = density of water,
H_L = clay liner thickness, and
H_P = piezometric level (head),

then, at some depth (for instance at the interface between the liner and the *saturated* layer)

$\gamma_L \cdot H_L$ would represent the total stress (σ), and

$\gamma_W \cdot H_P$ would represent the pore water pressure (u).

An unstable (or point of failure) situation could then be described as: $\sigma = u$

$$\text{i.e., } \gamma_L \cdot H_L = \gamma_W \cdot H_P \qquad (7.1)$$

or as a stress ratio: $$\frac{\gamma_L \cdot H_L}{\gamma_W \cdot H_P} = 1 \qquad (7.2)$$

Conversely, the total stress required to achieve a factor of safety of 1.4 is:

$$\gamma_L \cdot H_L > 1.4(\gamma_W \cdot H_P) \qquad (7.3)$$

An unstable condition caused by hydrostatic uplift may develop when the hydrostatic uplift force overcomes the downward force created by the weight of the soil layer(s). If an area acted upon by the hydrostatic force is sufficiently great, excess water pressure may cause overlying soil to rise, creating a failure known as "heave." Although heave can take place in any soil, it will most likely occur at an interface between a relatively impervious layer (such as a clay liner) and a *saturated*, relatively pervious base.

Rather than assigning specific values, the terms "relatively impervious" and "relatively pervious" are used here only to indicate a difference in permeabilities between the two respective layers. In simple terms, the bigger this difference is, the higher the uplift force on the "relatively impervious" layer will be.

Water percolation through a soil layer affects hydrostatic uplift force. As a result, considering seepage may theoretically be a more accurate approach. The shear resistance of the soil could also be theoretically taken into account. However, for practical purposes, a conservative evaluation of the resistance created by a soil layer against hydrostatic uplift can be accomplished by calculating a maximum uplift force based on a maximum measured piezometric head and comparing it to the normal stress created by the overlying soil layers. This is especially true when checking an interface between a subbase and a clay (or plastic) liner, where any significant seepage through the liner material is not anticipated nor wanted.

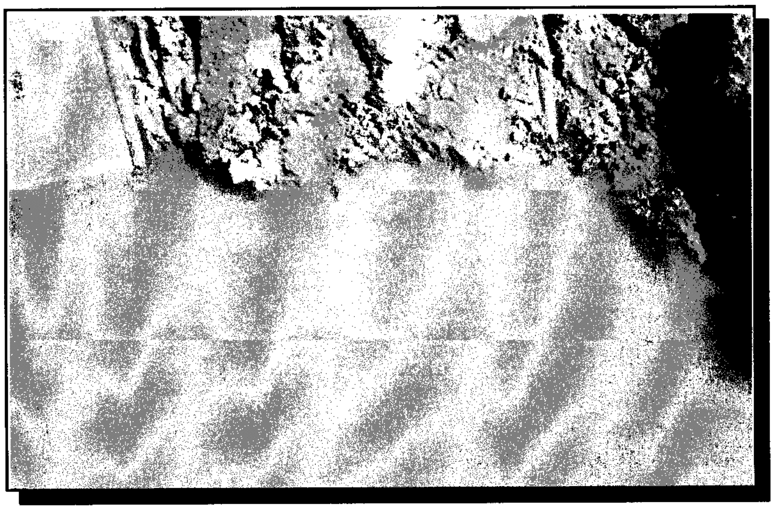

Figure 7-6 This is another example of hydrostatic pressures at an Ohio landfill causing flow through more than 20 feet of heavy in situ clay materials resulting in flooding of the excavation. Note that in this case, the presence of water cannot be taken into account due to precipitation. The flow of uplift water is evidenced only by a cloudy disturbance in the flooded excavation.

Hydrostatic Uplift - Example Methodology

A factor of safety is commonly calculated as a ratio between a resisting (available or stabilizing) force and a driving (attacking or destabilizing) force. The factor of safety against hydrostatic uplift can be expressed as:

$$FS = \frac{F_{GL}}{F_{HW}} \geq 1.40 \qquad (7.4)$$

where F_{GL} = downward force resulting from the weight of soil,
F_{HW} = hydrostatic uplift force, and
FS = factor of safety against hydrostatic uplift.

> A rough rule of thumb can be drawn from this example, such that potential for heaving of a soil layer exists whenever a piezometric level (head) extends to an elevation more than 1.3 times the thickness of the layer that is above the plane of potential failure (usually the contact plane between two layers with different permeabilities).

The forces in Equation 7.4 can be defined as:

$$F_{GL} = \gamma_L \cdot H_L \cdot A$$

and

$$F_{HW} = \gamma_w \cdot H_p \cdot A$$

where A = unit area.

When the forces in Equation 7.4 are substituted with above definitions, unit areas cancel. The expression now takes the form of Terzaghi's equation (Equation 7.2), with exception that number 1, previously indicating an unstable condition, is replaced with a FS:

$$FS = \frac{\gamma_L \cdot H_L}{\gamma_W \cdot H_P} \geq 1.40 \qquad (7.5)$$

For example, if γ_L= 112 pcf and γ_W= 62.4 pcf then the critical piezometric level can be calculated by using Equation 7.5 as follows:

$$H_P \leq \frac{\gamma_L \cdot H_L}{\gamma_W \cdot FS} \leq \frac{112 \cdot H_L}{62.4 \cdot 1.4} \leq 1.282 \cdot H_L \quad (\approx 1.3 \cdot H_L)$$

The piezometric level in the *saturated* layer can be measured with piezometers, water levels in *borings*, or other techniques, and compared to $1.3 \cdot H_L$ to very roughly assess the likelihood of hydrostatic uplift. However, for permit applications or other authorization requests submitted to Ohio EPA, accurate calculations using facility specific values must be included.

Hydrostatic Uplift - Example Calculation

If a sump (or another hole) is being excavated in a soil layer subjected to hydrostatic pressure (H_P, see **Figure 7-7**), the maximum depth of the sump can be calculated that would still allow for the required factor of safety. This can be determined by substituting H_L in Equation 7.5 with H_{Lsump} and calculating its value.

For example, determine if a three-foot deep sump can be constructed under the following conditions (see **Figure 7-7**): H_L = 5 ft,
H_P = 8 ft,
γ_L = 112 pcf,
γ_w = 62.4 pcf, and
D_{SB} = depth from top of liner to sump bottom (8 ft).

Using Equation 7.5 the factor of safety is: $FS = \dfrac{\gamma_L \cdot H_L}{\gamma_W \cdot H_P} = \dfrac{112 \cdot 5}{62.4 \cdot 8} = 1.12$, which is unacceptable.

As a result, a thicker liner will be needed in the sump. The thickness of liner in the sump necessary to provide a factor of safety of 1.40 can be calculated as follows:

$$H_{L\,sump} = \frac{FS \cdot \gamma_W \cdot H_P}{\gamma_L} = \frac{1.4 \cdot 62.4 \cdot 8}{112} = 6.24\ ft$$

Therefore, the maximum depth of the sump should not exceed:

$$H_{sump} = D_{SB} - H_{Lsump} = 8\,\text{ft} - 6.24\,\text{ft} = 1.76\,\text{ft}$$

To avoid water infiltrating into the excavation and damaging the liner, some form of reduction to the piezometric head (e.g., using dewatering wells) will be necessary during excavation and construction of the liner system and sump used in this example.

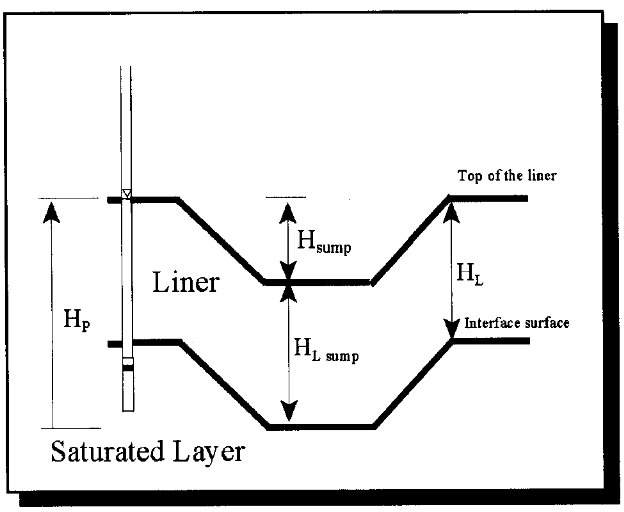

Figure 7-7 An example of piezometric head on a soil liner with a sump.

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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.

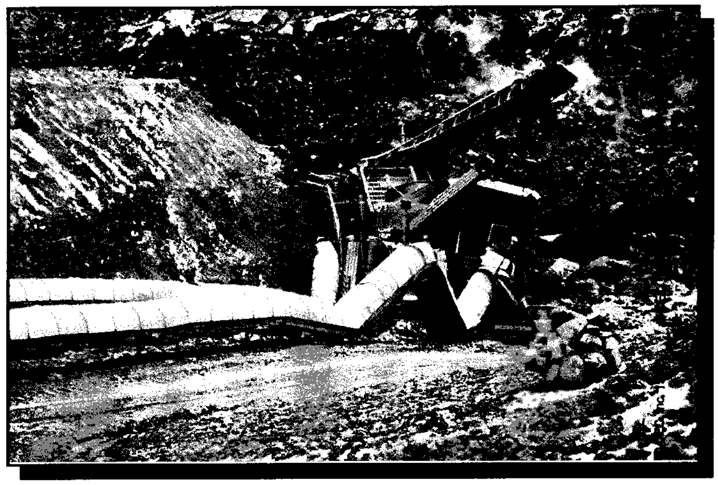


Figure 8-1 A sliding mass of waste is capable of producing enormous force as is demonstrated in this picture of mining and earthmoving equipment that were crushed by a large waste failure at an Ohio landfill. Photo courtesy of CEC, Inc.

- 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,

> 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.

- 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

- 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.

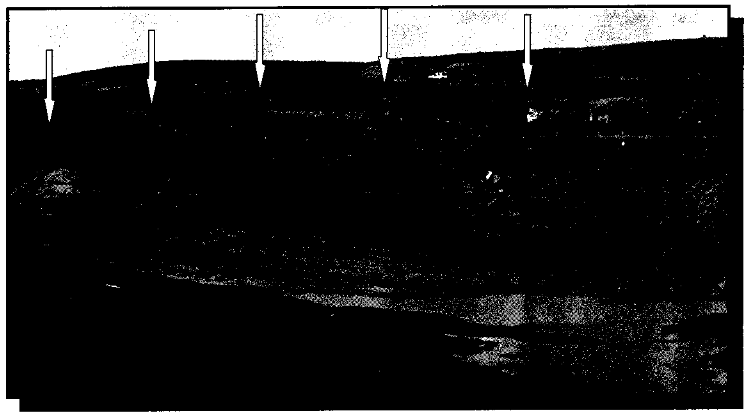


Figure 8-2 A complex rotational failure at a Texas landfill. White arrows identify the failure escarpment. For scale, note the pickup truck above the failure escarpment. Photograph courtesy of Dr. Timothy D. Stark, PE, University of Illinois, Urbana.

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:

> 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 $\phi = 35°$. 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).

- *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*.

- 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.

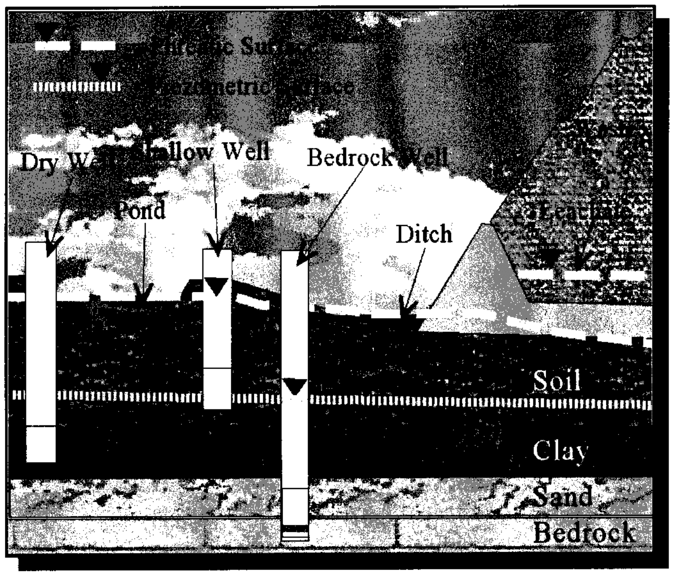


Figure 8-3 Examples of *phreatic* and *piezometric surfaces*.

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.

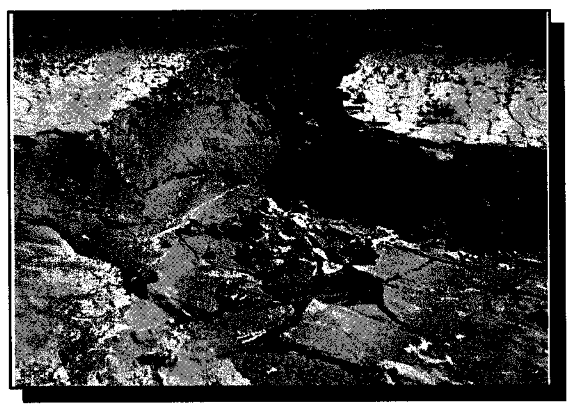


Figure 8-4 Looking through the failed containment berm of a storm water retention basin that was located in Cuyahoga County. The outlet was plugged, causing the *phreatic surface* in the basin to become unexpectedly high. As a result, it overwhelmed the shear strength of the soil materials used to construct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

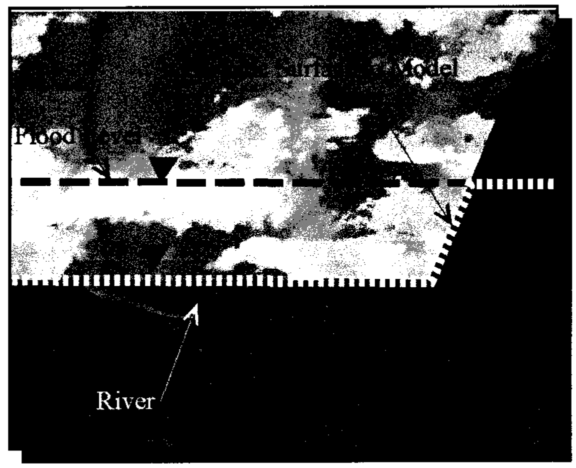


Figure 8-5 Example *phreatic surface* to model to account for pore water pressure created by flooding and then flood subsidence.

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.

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:

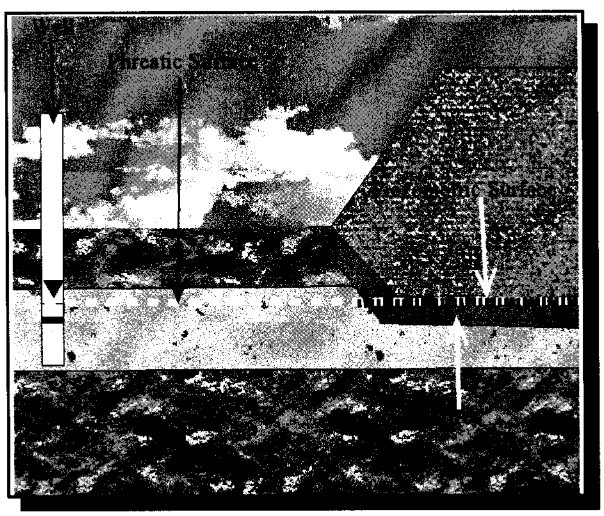


Figure 8-6 Example of a *piezometric surface* created by engineered components of a *waste containment facility*.

- 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

> 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.

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.

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.

> 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.

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.

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 strengths* 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° 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.

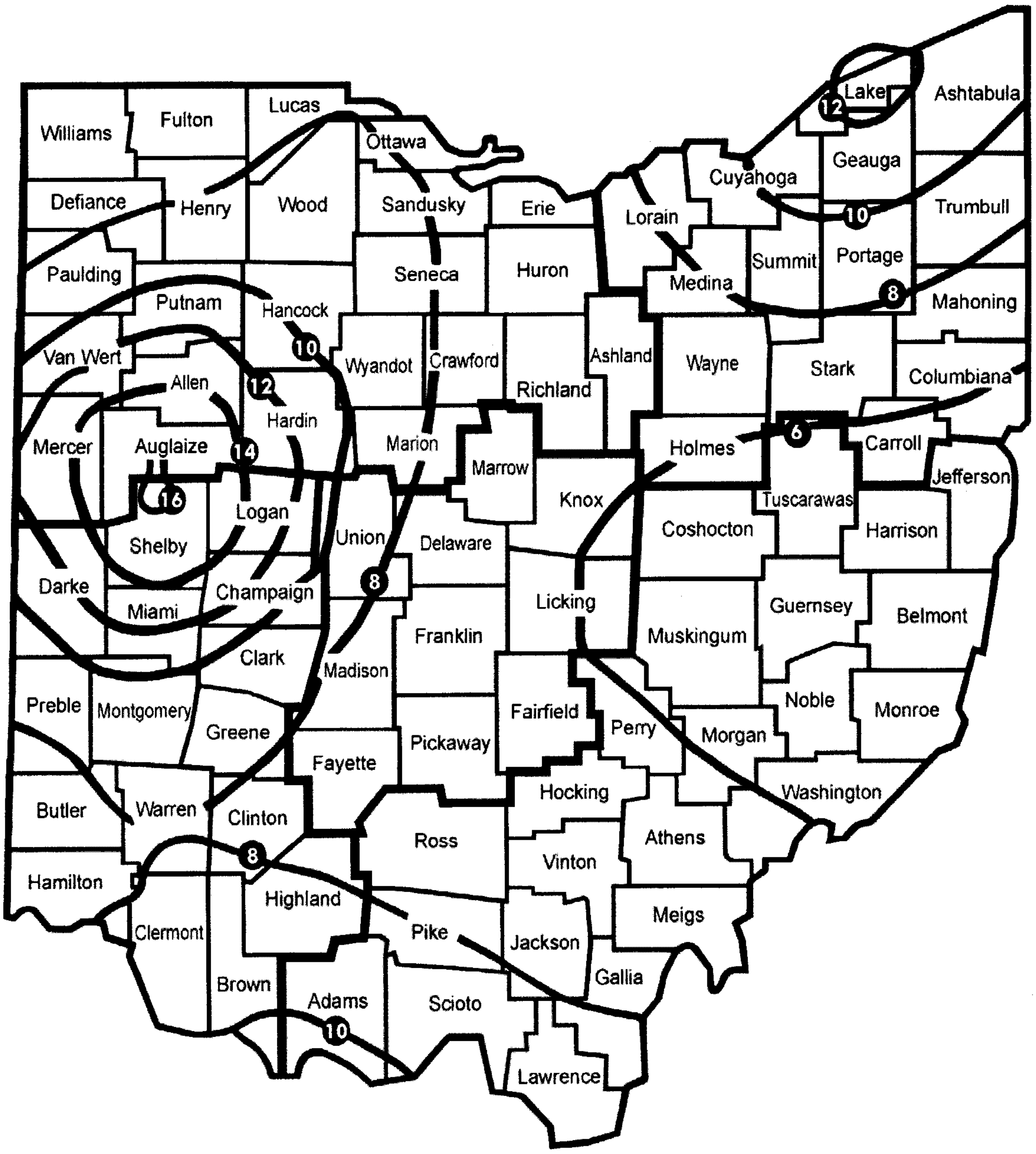


Figure 8-9 The peak horizontal ground acceleration (%g) with 2% probability of exceedance in 50 years. U.S. Geological Survey, October 2002, National Seismic Hazard Mapping Project, "Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary)."

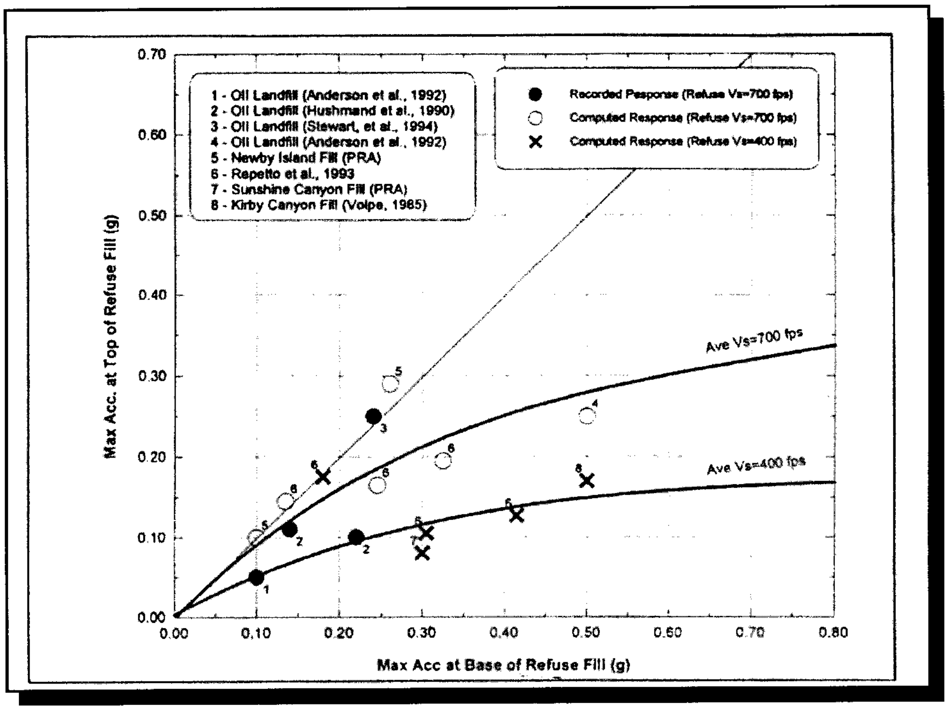


Figure 8-10 Approximate relationship between maximum accelerations at the base and crest of 200 feet of refuse. Singh and Sun, 1995, Figure 1.

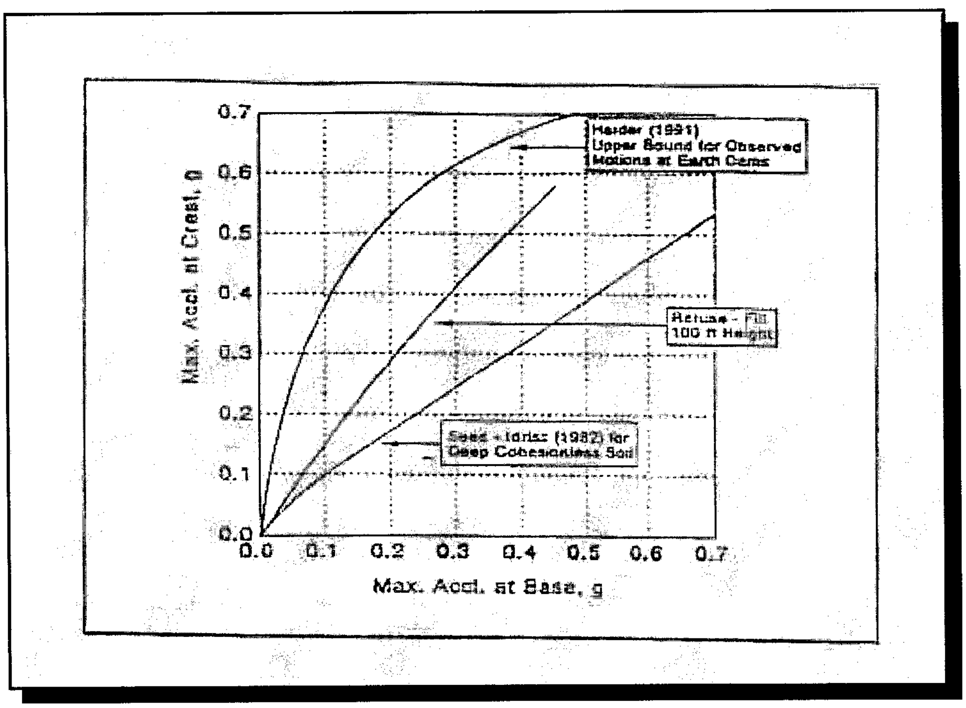


Figure 8-11 Approximate relationship between maximum accelerations at the base and crest for various ground conditions. Singh and Sun, 1995, Figure 3.

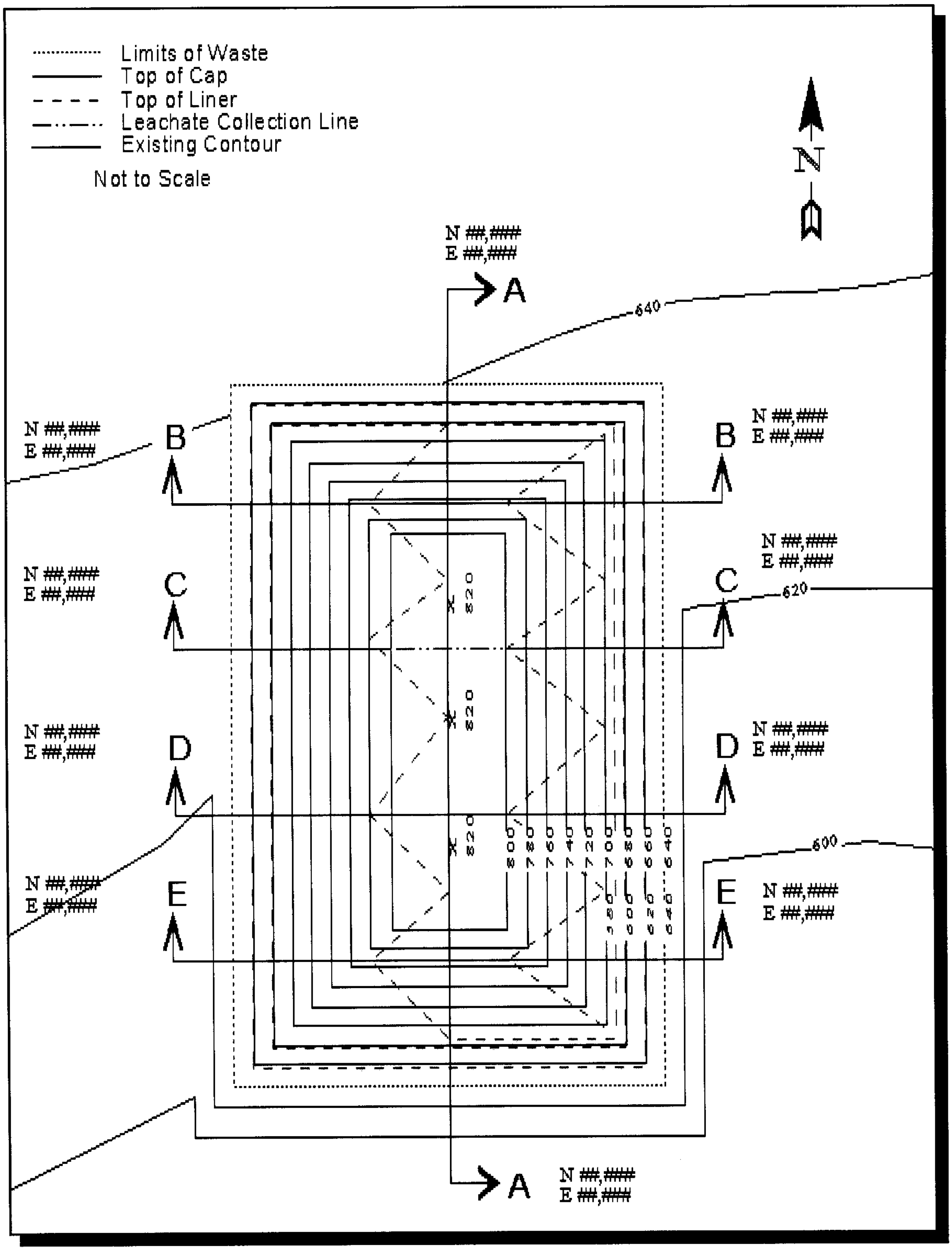


Figure 8-12 Example plan view showing top and bottom elevations and the location of cross sections that were analyzed for stability.

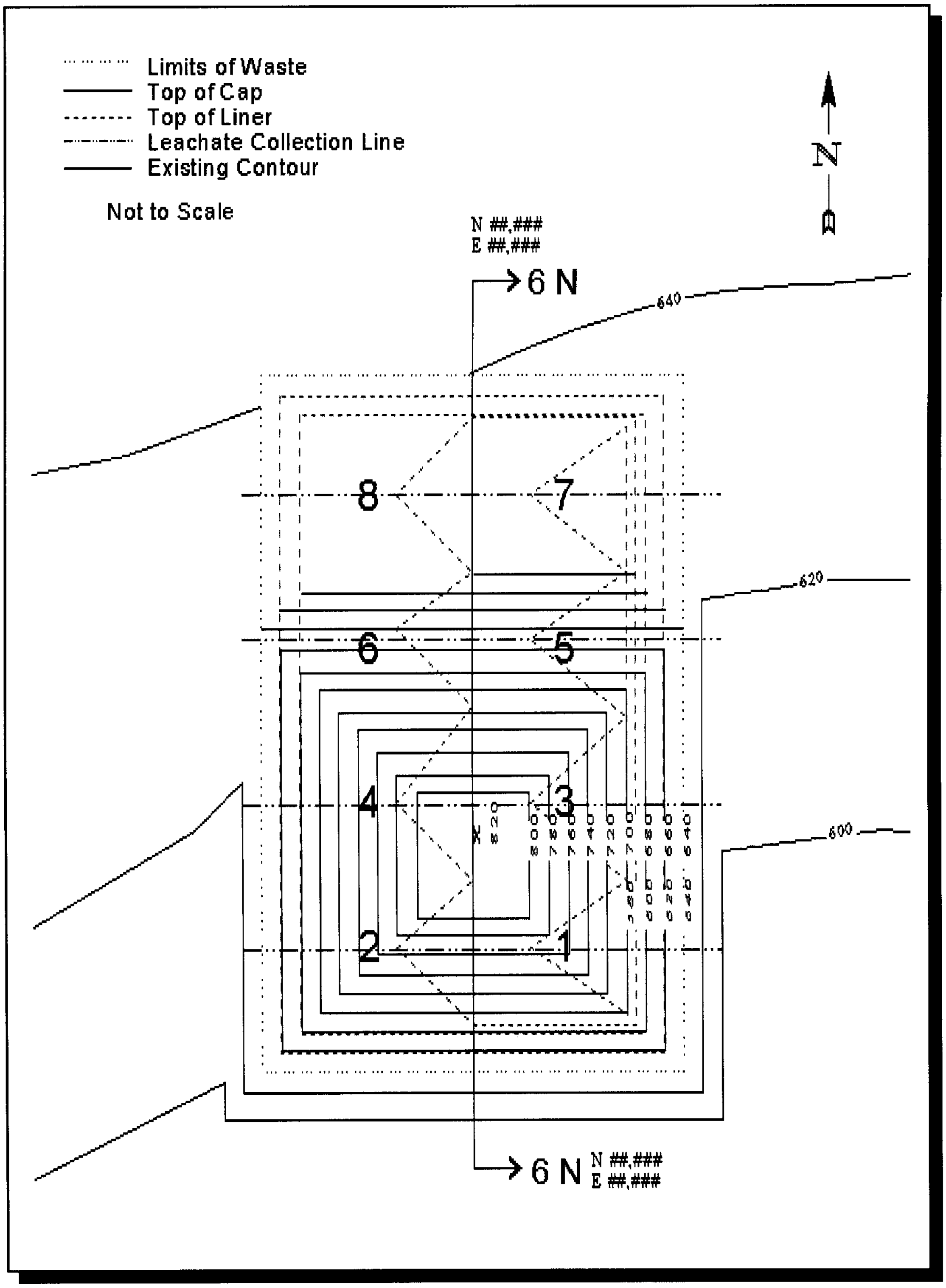


Figure 8-13 Example plan view showing the location of one of the *interim slope* cross sections that were analyzed for stability.

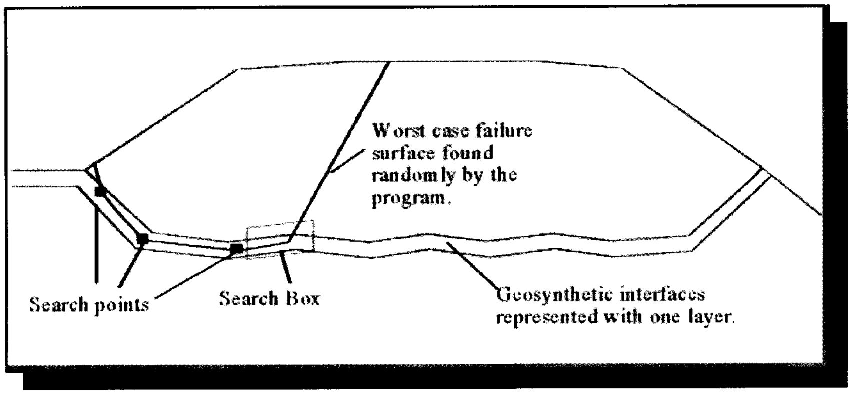


Figure 8-14 Cross Section A-A'. Example translational failure surface found by directing modeling software to a specific interface.

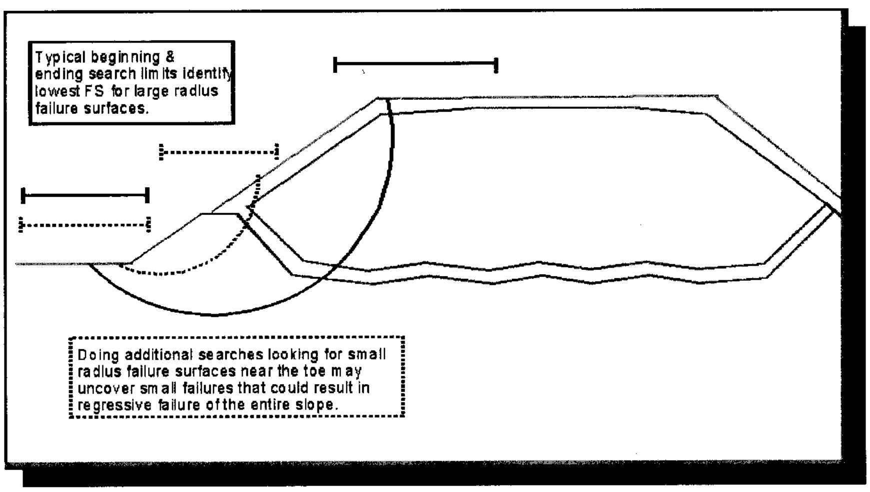


Figure 8-15 Example of using different search limits to look for different size failure surfaces.

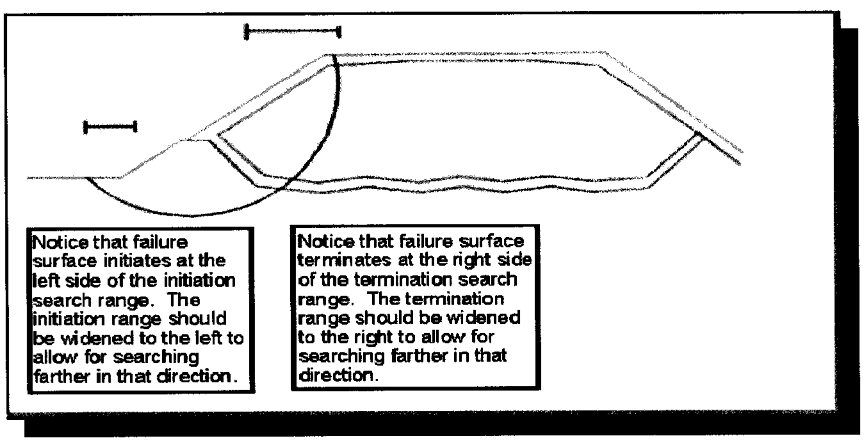


Figure 8-16 Example of search limits inappropriately restricting the search engine in both directions. Even if the search limits inappropriately restrict searching in only one direction, the search range should be adjusted and the analysis run again.

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

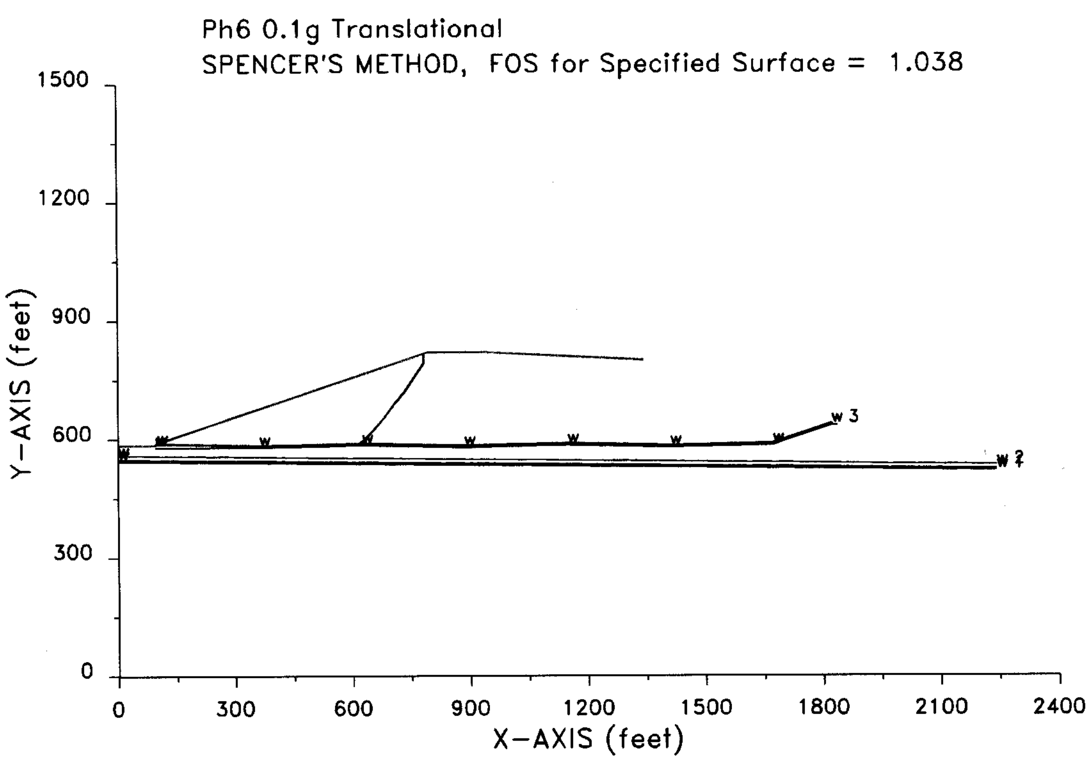
XSTABL

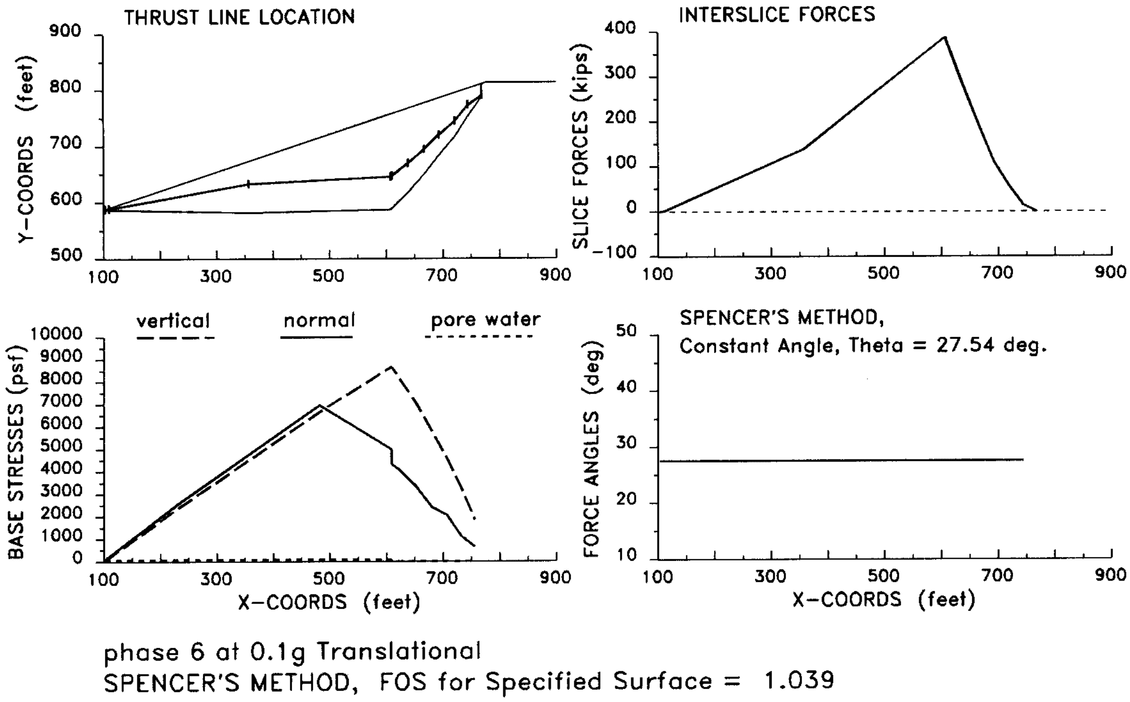
Slope Stability Analysis
using the
Method of Slices

Ver.5.202 96) 1697

Problem Description : Ph6 0.1g Translational

PH6TBQSS 12-17-** 12:38





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)

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

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 # 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

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

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.

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.

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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.

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

- 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*,
- 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:

> 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.

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 use of higher factors of safety against shallow failures may be warranted whenever:

> 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 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.

> 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.

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.

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 ϕ, 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.

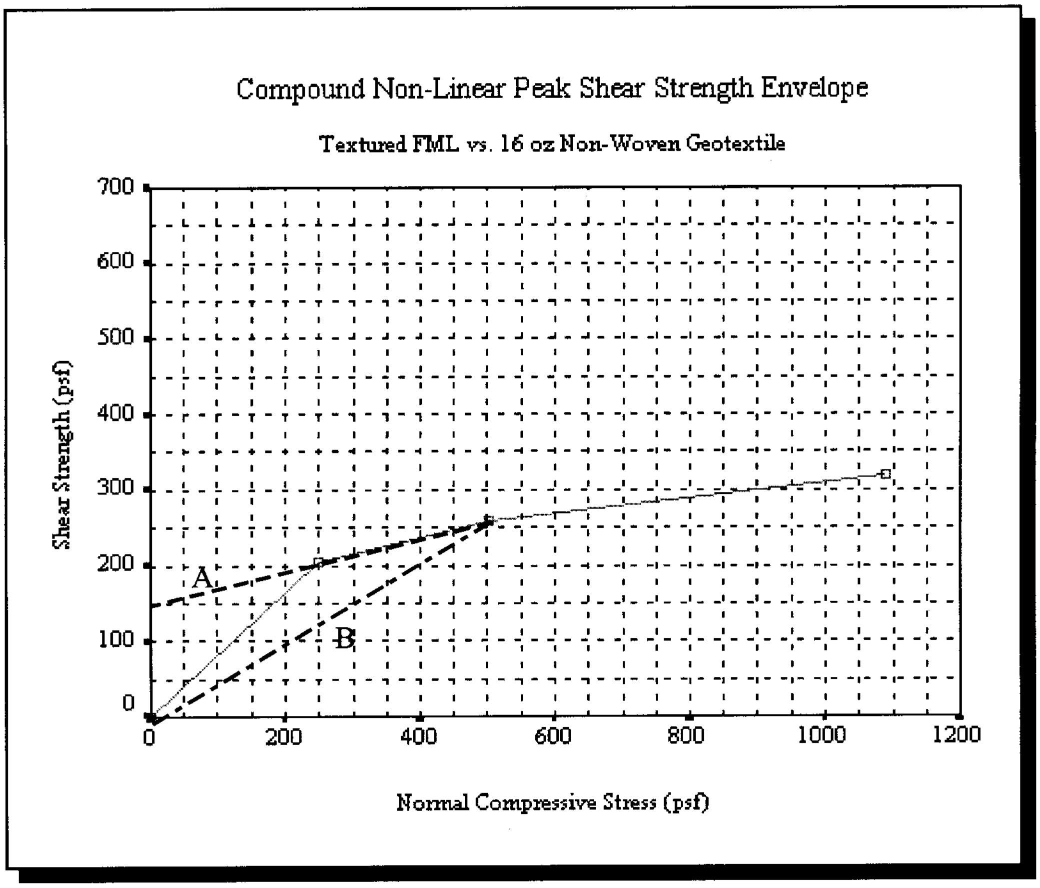


Figure 9-2 An example of a compound nonlinear *peak shear strength* envelope from test results of textured FML/GT interfaces that will be used on a final slope with no tack-on benches or roads, having a 1-ft leachate drainage layer covered by a 2-ft thick protective layer. For this facility, the sand 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 ϕ, 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 $\phi = 0°$, a c and ϕ derived from line A, or a ϕ 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.

> 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 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).

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 1×10^{-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**).

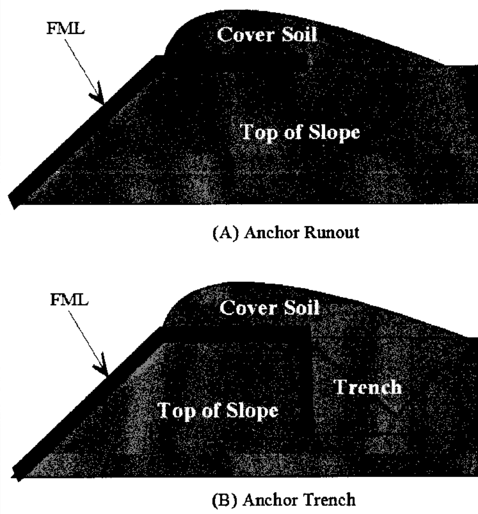


Figure 9-4 Example detail of (A) anchor runout and (B) anchor trench. Anchor trenches can also be "V" shaped (dashed line).

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.

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.

> 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.

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.

$$FS = \frac{\dfrac{c}{\gamma_c z_c \cos^2\beta} + \tan\phi\left[1 - \dfrac{\gamma_w(z_c - d_w)}{\gamma_c z_c}\right] - n_g(\tan\beta)(\tan\phi)}{n_g + \tan\beta} \qquad (9.1)$$

$$\phi = \tan^{-1}\left[\frac{FS(n_g + \tan\beta) - \left(\dfrac{c}{\gamma_c z_c \cos^2\beta}\right)}{\left[1 - \dfrac{\gamma_w(z_c - d_w)}{\gamma_c z_c}\right] - n_g \tan\beta}\right] \qquad (9.1.1)$$

where FS = factor of safety against shallow failure,

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

γ_c = field density of cover materials,

γ_w = density of water,

c = cohesion of failure surface,

ϕ = internal angle of friction,

β = 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,
β = 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 1×10^{-4} cm/sec, use USDA Soil Survey estimates, or do in-field testing of a long-term vegetated area adjacent to the facility).

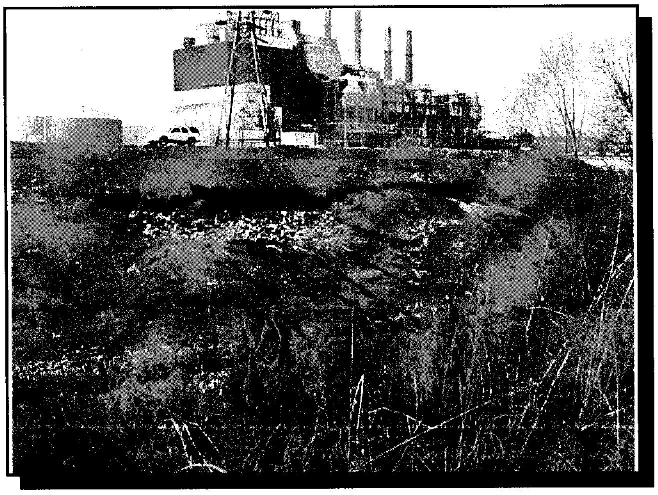


Figure 9-5 A shallow rotational failure in a containment berm at an ash settling pond in Ohio.

Shallow Failure - Example Calculations

A 200-ft high landfill in Ohio has 3(h):1(v) (18.43°) *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 2.0×10^{-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 1.0×10^{-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
$$\phi_{required} = \tan^{-1}\left[\frac{1.5(0g + \tan 18.43) - \left(\frac{0\,psf}{120\,pcf \cdot 1\,ft \cdot \cos^2 18.43}\right)}{\left[1 - \frac{62.4\,psf\,(1\,ft - 1\,ft)}{120\,pcf \cdot 1\,ft}\right] - 0g(\tan 18.43)}\right] = 26.56^\circ$$

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

The minimum interface and internal *peak shear strength* of all materials required for both *internal slopes* and *final slopes* at this facility is 26.56° 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
$$\phi = \tan^{-1}\left[\frac{1.0(0.14g + \tan 18.43) - \left(\frac{0\,psf}{120\,pcf \cdot 2.5\,ft \cdot \cos^2 18.43}\right)}{\left[1 - \frac{62.4\,psf\,(2.5\,ft - 2.5\,ft)}{120\,pcf \cdot 2.5\,ft}\right] - 0.14g(\tan 18.43)}\right] = 26.40^\circ$$

The minimum interface and internal *peak shear strength* for all materials in the composite cap system at this facility is 26.40° 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 $$\phi_{required} = \tan^{-1}\left[\frac{1.1(0g + \tan 18.43) - \dfrac{0\,psf}{120\,pcf\cdot 1\,ft\cdot\cos^2 18.43}}{\left[1 - \dfrac{62.4\,psf(1\,ft - 0.564\,ft)}{120\,pcf\cdot 1\,ft}\right] - 0g(\tan 18.43)}\right] = 25.37^\circ$$

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 25.37° 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 ($2.12\text{x}10^{-3}$ cm/sec) and Equation 9.3 because P(1-RC) > k_c (e.g., $2.12\text{x}10^{-3}$ cm/sec (1 - 0.9) > $1.0\text{ x}10^{-4}$ 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.

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

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:

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

From Equation 9.3 $$h_{avg} = \frac{1.0\times10^{-4}\,cm/\text{sec}\cdot 3962.4\,cm(\cos 18.43)}{1.094\,cm/\text{sec}(\sin 18.43)} = 1.0869\ cm = 0.03565 ft$$

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

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

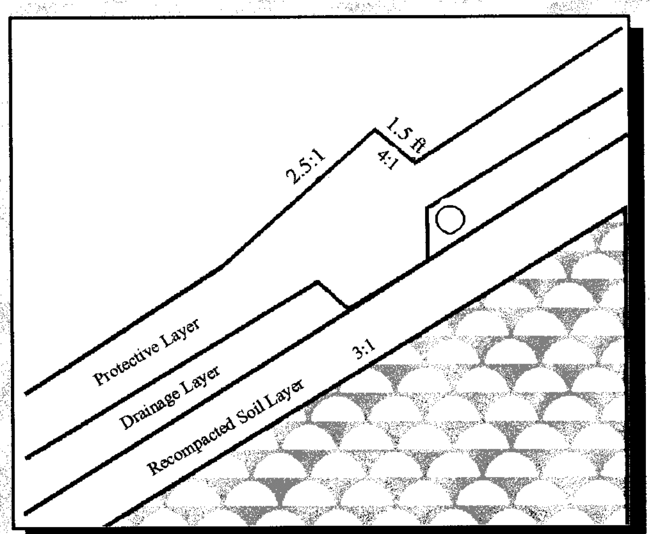
The minimum *peak shear strength* required for all interfaces and materials to have a static factor of safety equal to 1.10 for *final slopes* under *saturated* conditions at this facility is 37.37°.

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 37.37•.

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

<table>
<tr><td rowspan="1">Component Being Evaluated</td><td>Method Used</td><td colspan="2">Back Calculated Result</td></tr>
<tr><td>Protective layer permeability</td><td>Formula 9.3</td><td colspan="2">2.54x10^{-5}cm/sec[1]</td></tr>
<tr><td>Protective layer shear strength</td><td>Shallow rotational XSTABL modeling[2]</td><td colspan="2">c = 0 ϕ = 31•[1]</td></tr>
<tr><td rowspan="3">RSL shear strength to provide an FS• 1.50 under drained static conditions</td><td rowspan="3">Shallow rotational XSTABL modeling</td><td colspan="2">Shear Strength[1] Envelope</td></tr>
<tr><td>Normal Stress</td><td>Shear Stress</td></tr>
<tr><td>0
288
576
1440</td><td>0
275
300
350</td></tr>
<tr><td rowspan="3">FML vs. DL or RSL shear strength to provide an FS• 1.50 under drained static conditions</td><td rowspan="3">Shallow translational XSTABL[2] modeling</td><td colspan="2">Shear Strength[1] Envelope</td></tr>
<tr><td>Normal Stress</td><td>Shear Stress</td></tr>
<tr><td>0
288
576
1440</td><td>0
215
275
350</td></tr>
</table>

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.

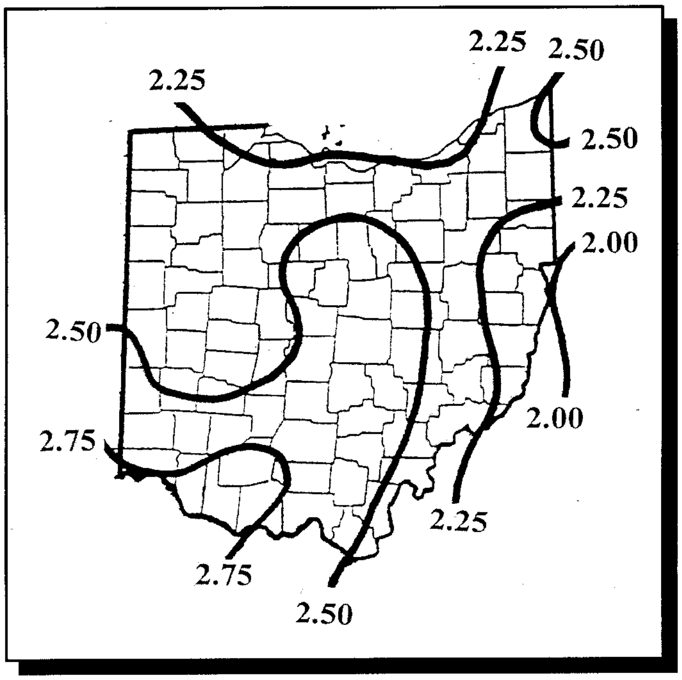


Figure 9-6 The 50-year 1-hour storm. Spatial distribution of 1-hour rainfall (inches/hour). Huff, Floyd A., and Angle, James R., "Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

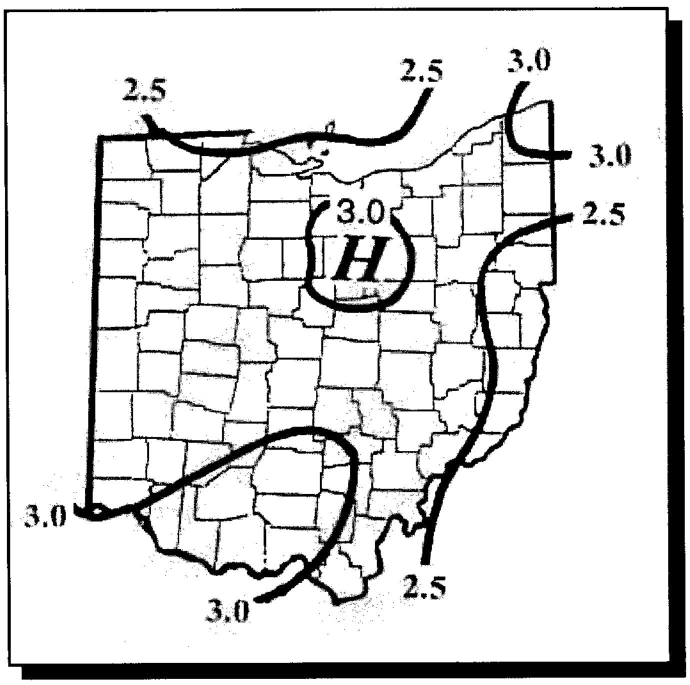


Figure 9-7 The 100-year 1-hour storm. Spatial distribution of 1-hour rainfall (inches/hour). Huff, Floyd A., and Angle, James R,. "Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

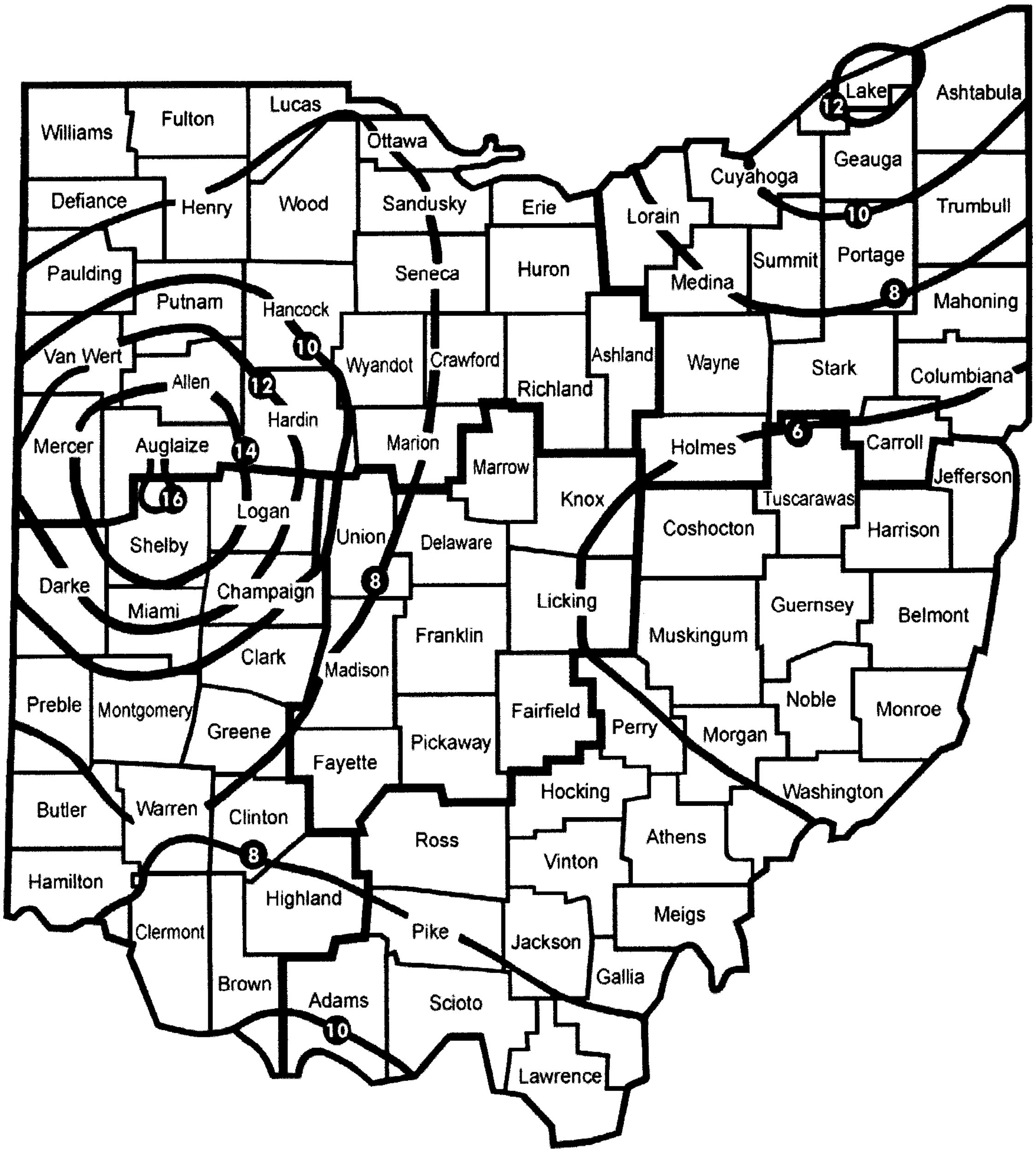


Figure 9-8 The peak acceleration (%g) with 2% probability of exceedance in 50 years. U.S. Geological Survey (October 2002) National Seismic Hazard Mapping Project, "Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary)."

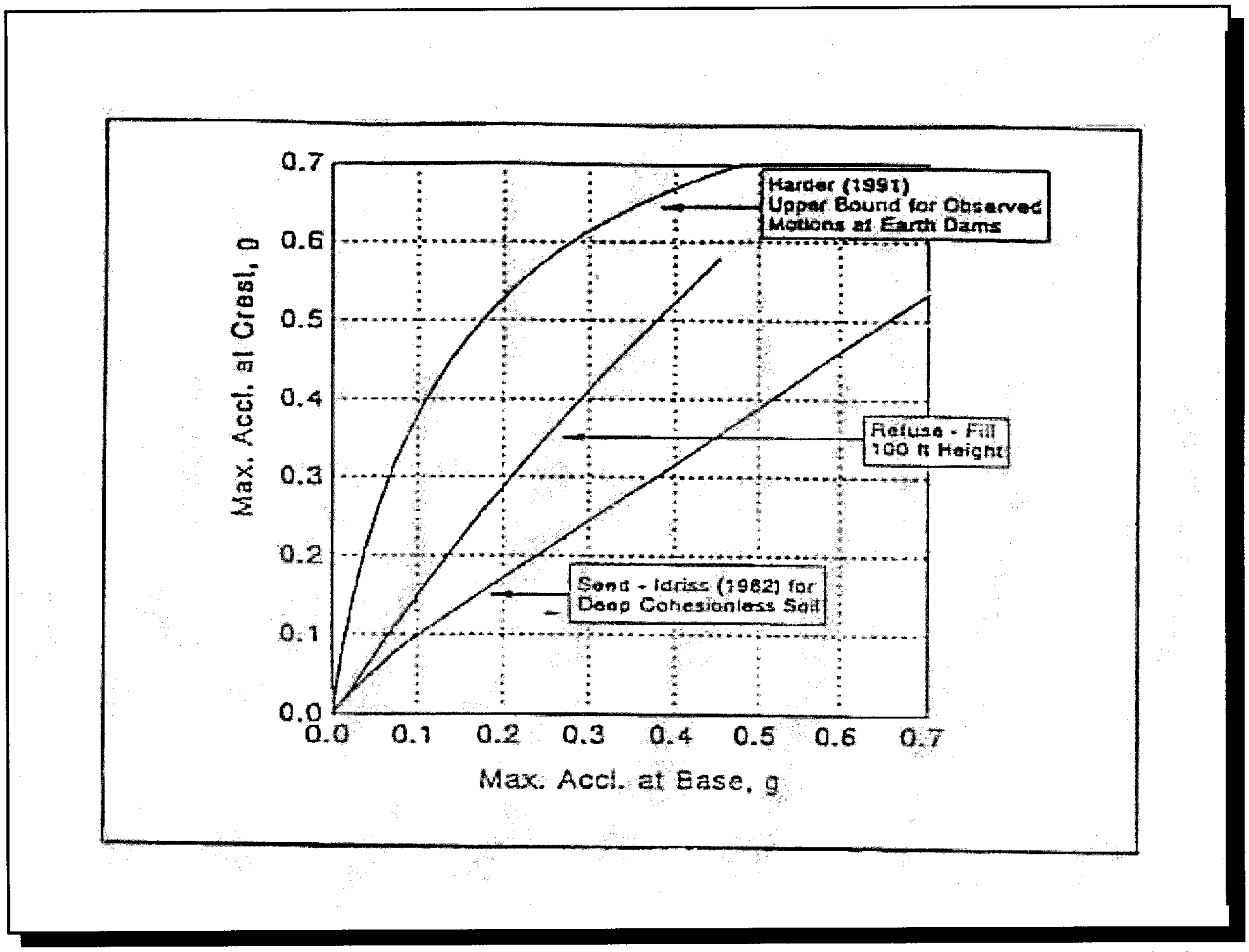


Figure 9-9 The relationship between maximum horizontal seismic acceleration at the base and crest of 100 feet of refuse, on top of deep cohesionless soils, and on top of earth dams. Singh and Sun, 1995, Figure 3.

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

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

                 All Rights Reserved

                 Ver. 5.20296 ) 1697
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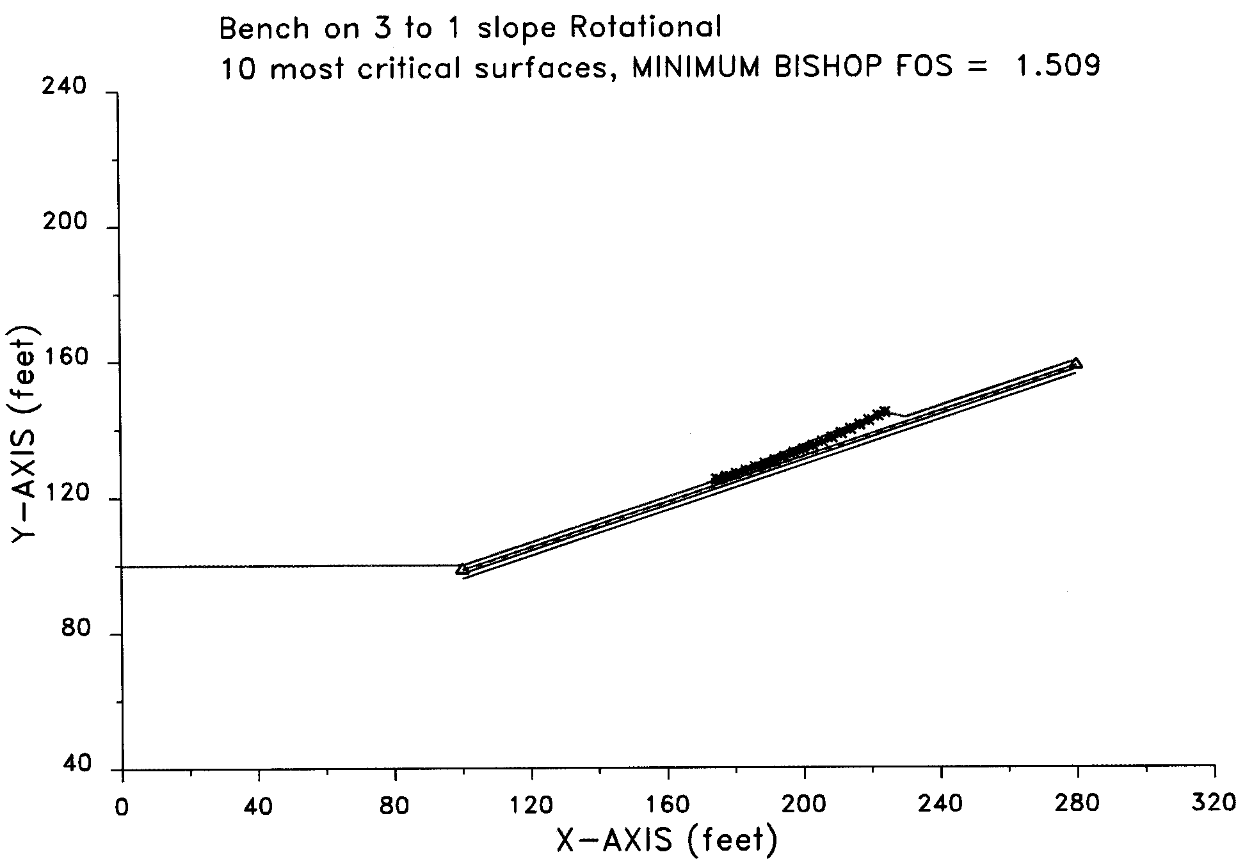
Problem Description : Bench on 3 to 1 slope Rotational

BEN3PTLD 6-01-** 12:34



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 No.	Normal Stress (psf)	Shear Stress (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 No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)
1	100.0	98.5	280.0	158.5

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

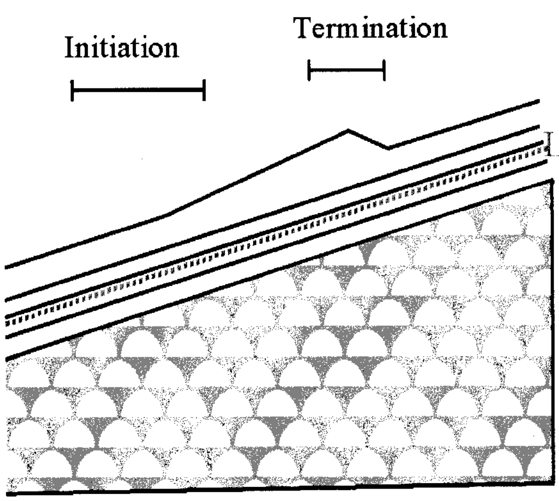
A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

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.

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ANGULAR RESTRICTIONS
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```

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

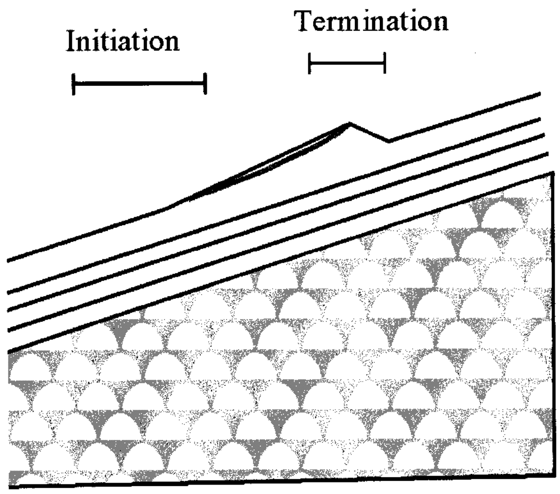
Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

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

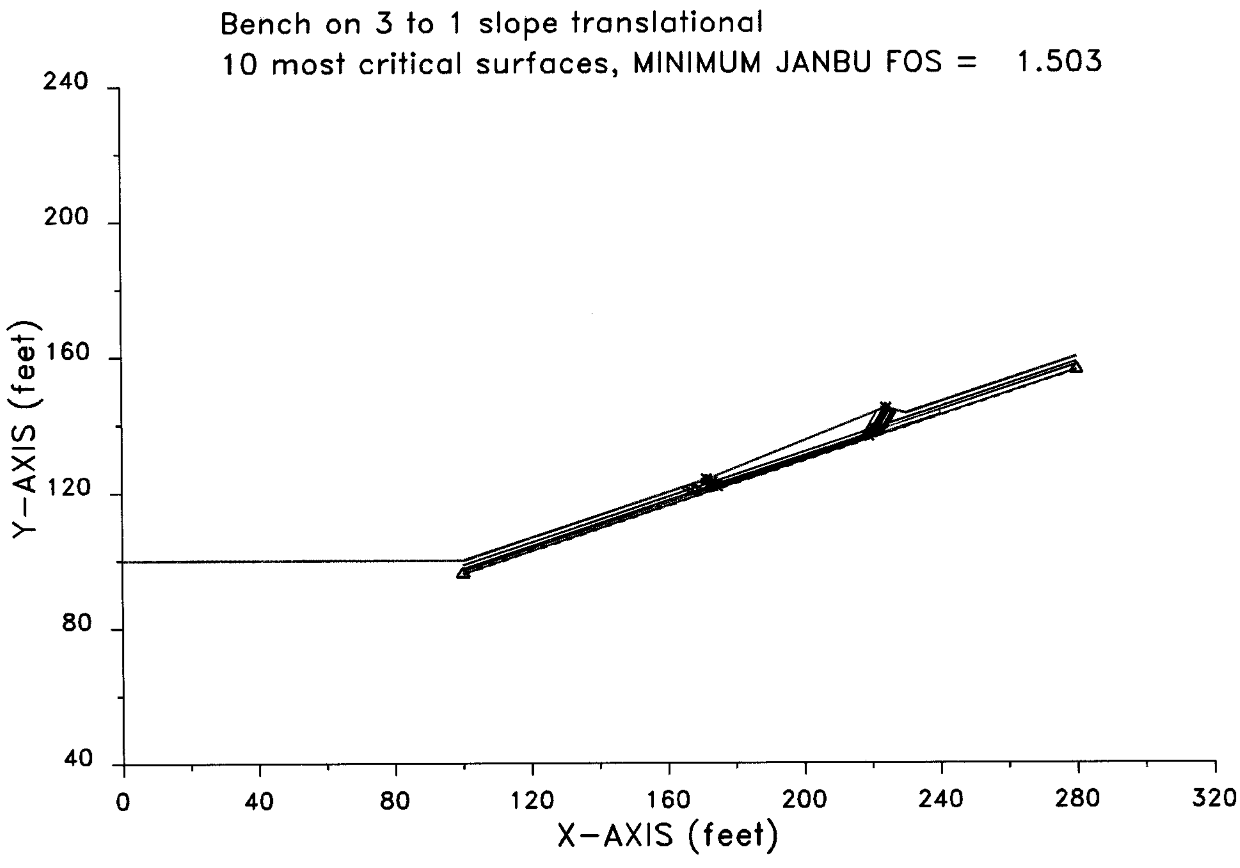
XSTABL File: BEN3RSLT 6-01-** 11:48

X S T A B L

Slope Stability Analysis
using the
Method of Slices

Ver. 5.20296) 1697

BEN3RSLT 6-01-** 11:48



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.

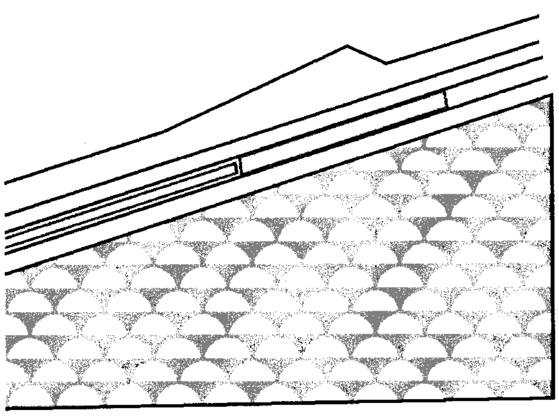
2 boxes specified for generation of central block base

* * * * * 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.



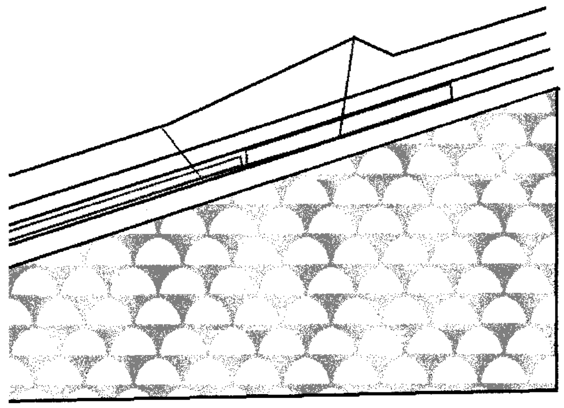
Factors of safety have been calculated by the :

* * * * * 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  x-surf  y-surf
  No.    (ft)    (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  x-surf  y-surf
  No.    (ft)    (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  x-surf  y-surf
  No.    (ft)    (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

* * * END OF FILE * * *

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APPENDIX 1

EARTHQUAKES AND SEISMIC RISK IN OHIO

OHIO GEOLOGY

GEOFACTS No. 3

OHIO DEPARTMENT OF NATURAL RESOURCES • DIVISION OF GEOLOGICAL SURVEY

Although most people do not think of Ohio as an earthquake-prone state, at least 120 earthquakes with epicenters in Ohio have been felt since 1776. In addition, a number of earthquakes with origins outside Ohio have been felt in the state. Most of these earthquakes have been felt only locally and have caused no damage or injuries.

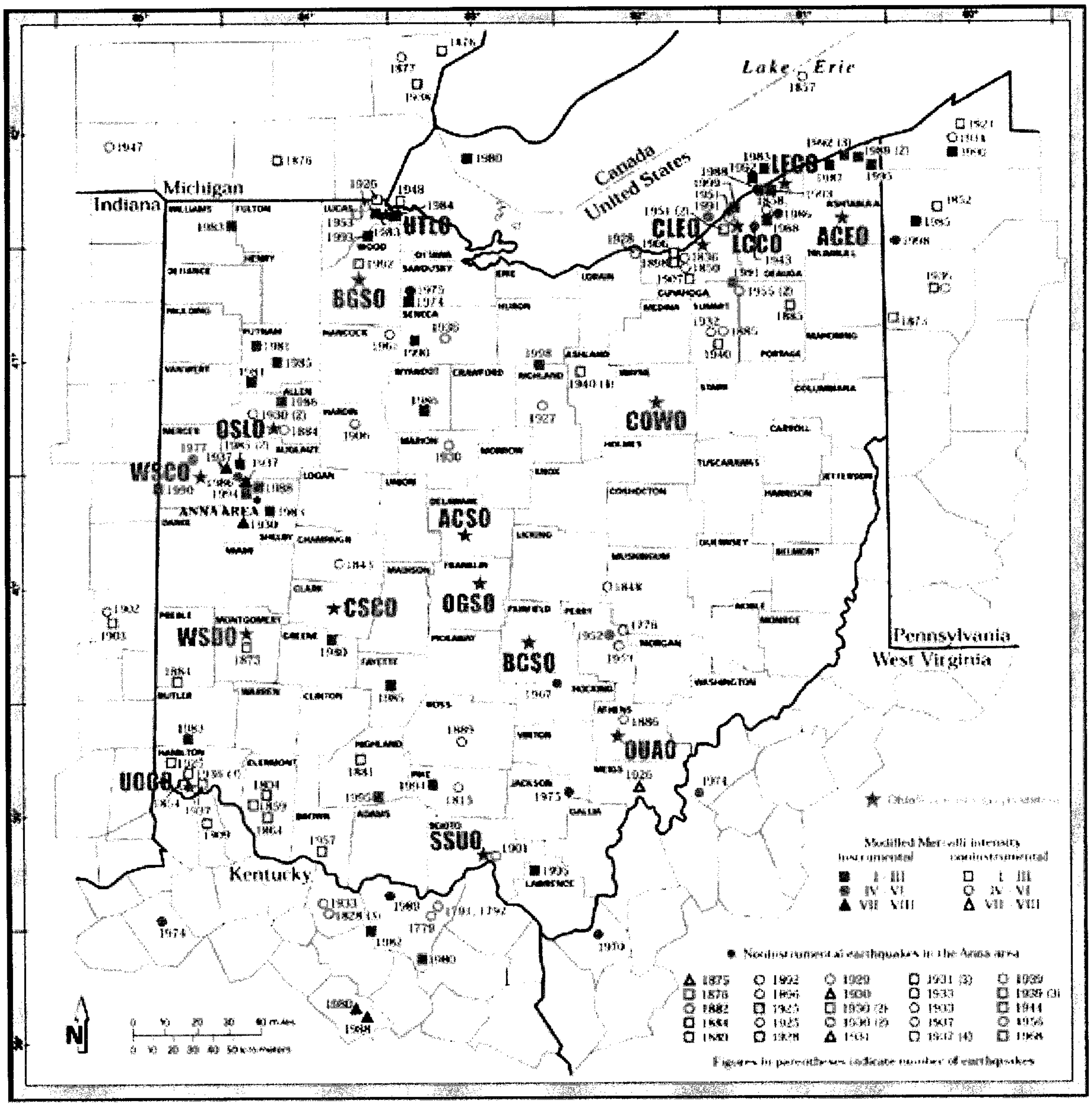
However, at least 14 moderate-size earthquakes have caused minor to moderate damage in Ohio. Fortunately, no deaths and only a few minor injuries have been recorded for these events.

Ohio is on the periphery of the New Madrid Seismic Zone, an area in Missouri and adjacent states that was the site of the largest earthquake sequence to occur in historical times in the continental United States. Four great earthquakes were part of a series at New Madrid in 1811 and 1812. These events were felt throughout the eastern United States and were of sufficient intensity to topple chimneys in Cincinnati. Some estimates suggest that these earthquakes were in the range of 8.0 on the Richter scale.

A major earthquake centered near Charleston, South Carolina, in 1886 was strongly felt in Ohio. More recently, an earthquake with a Richter magnitude of 5.3 centered at Sharpsburg, Kentucky, in 1980 was strongly felt throughout Ohio and caused minor to moderate damage in communities near the Ohio River in southwestern Ohio. In 1998 a 5.2-magnitude earthquake occurred in western Pennsylvania, just east of Ohio, and caused some damage in the epicentral area.

EARTHQUAKE REGIONS

Three areas of the state appear to be particularly susceptible to seismic activity (see map below). Shelby County and surrounding counties in western Ohio have experienced more earthquakes than any other area of the state. At least 40 felt earthquakes have occurred in this area since 1875. Although most of these events have caused little or no damage, earthquakes in 1875, 1930, 1931, and 1937 caused minor to moderate damage. Two earthquakes in 1937, on March 2 and March 9, caused significant damage in the Shelby County community of Anna. The damage included toppled chimneys, cracked plaster, broken windows, and structural damage to buildings. The community school, of brick construction, was razed because of structural damage.



ACSO Alum Creek (Ohio Earthquake Information Center)

OGSO Ohio Geological Survey
ACEO Ashtabula EMA
BCSO Bloom-Carroll Schools
BGSO Bowling Green State University
CSCO Clark State Community College
CLEO Cleveland Museum of Natural History
COWO College of Wooster
LECO Lake Erie College
LCCO Lakeland Community College
OSLO Ohio State University-Lima
OUAO Ohio University
SSUO Shawnee State University
UOCO University of Cincinnati
UTLO University of Toledo
WSCO Wright State-Celina
WSDO Wright State-Dayton (pending)

Northeastern Ohio has experienced at least 20 felt earthquakes since 1836. Most of these events were small and caused little or no damage. However, an earthquake on January 31, 1986, strongly shook Ohio and was felt in 10 other states and southern Canada. This event had a Richter magnitude of 5.0 and caused minor to moderate damage, including broken windows and cracked plaster, in the epicentral area of Lake and Geauga Counties.

Southeastern Ohio has been the site of at least 10 felt earthquakes with epicenters in the state since 1776. The 1776 event, recorded by a Moravian missionary, has a very uncertain location. Earthquakes in 1901 near Portsmouth (Scioto County), in 1926 near Pomeroy (Meigs County), and in 1952 near Crooksville (Perry County) caused minor to moderate damage.

CAUSES OF OHIO EARTHQUAKES

The origins of Ohio earthquakes, as with earthquakes throughout the eastern United States, are poorly understood at this time. Those in Ohio appear to be associated with ancient zones of weakness in the Earth's crust that formed during continental collision and mountain-building events about a billion years ago. These zones are characterized by deeply buried and poorly known faults, some of which serve as the sites for periodic release of strain that is constantly building up in the North American continental plate due to continuous movement of the tectonic plates that make up the Earth's crust.

SEISMIC RISK

Seismic risk in Ohio, and the eastern United States in general, is difficult to evaluate because earthquakes are generally infrequent in comparison to plate-margin areas such as California. Also, active faults do not reach the surface in Ohio and therefore cannot be mapped without the aid of expensive subsurface techniques.

A great difficulty in predicting large earthquakes in the eastern United States is that the recurrence interval--the time between large earthquakes--is commonly very long, on the order of hundreds or even thousands of years. As the historic record in most areas, including Ohio, is only on the order of about 200 years--an instant, geologically speaking--it is nearly impossible to estimate either the maximum magnitude or the frequency of earthquakes at any particular site.

Earthquake risk in the eastern United States is further compounded by the fact that seismic waves tend to travel for very long distances. The relatively brittle and flat-lying sedimentary rocks of this region tend to carry these waves throughout an area of thousands of square miles for even a moderate-size earthquake. Damaging ground motion would occur in an area about 10 times larger than for a California earthquake of comparable intensity.

An additional factor in earthquake risk is the nature of the geologic materials upon which a structure is built. Ground motion from seismic waves tends to be magnified by *unconsolidated* sediments such as thick deposits of clay or sand and gravel. Such deposits are extensive in Ohio. Buildings constructed on *bedrock* tend to experience much less ground motion, and therefore less damage. Geologic maps, such as those prepared by the Ohio Division of Geological Survey, delineate and characterize these deposits. Geologic mapping programs in the state geological surveys and the U.S. Geological Survey are therefore critical to public health and safety.

Modified Mercalli Scale		Magnitude Scale
I	Detected only by sensitive instruments	1.5
II	Felt by few persons at rest, especially on upper floors; delicately suspended objects may swing	2
III	Felt noticeably indoors, but not always recognized as earthquake; standing autos rock slightly, vibrations like passing truck	2.5
IV	Felt indoors by many, outdoors by few, at night some awaken; dishes, windows, doors disturbed; standing autos rock noticeably	3
V	Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects	3.5, 4
VI	Felt by all, many frightened and run outdoors; falling plaster and chimneys, damage small	4.5
VII	Everybody runs outdoors; damage to buildings varies depending on quality of construction; noticed by drivers of autos	5
VIII	Panel walls thrown out of frames; walls, monuments, chimneys fall; sand and mud ejected; drivers of autos disturbed	5.5
IX	Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked; underground pipes broken	6, 6.5
X	Most masonry and frame structures destroyed; ground cracked, rails bent, landslides	7
XI	Few structures remain standing; bridges destroyed, fissures in ground, pipes broken, landslides, rails bent	7.5
XII	Damage total; waves seen on ground surface, lines of sight and level distorted, objects thrown up into air	8

General relationship between epicentral Modified Mercalli intensities and magnitude. Intensities can be highly variable, depending on local geologic conditions (modified from D.W. Steeples, 1978, Earthquakes: Kansas Geological Survey pamphlet).

The brief historic record of Ohio earthquakes suggests a risk of moderately damaging earthquakes in the western, northeastern, and southeastern parts of the state. Whether these areas might produce larger, more damaging earthquakes is currently unknown, but detailed geologic mapping, subsurface investigations, and seismic monitoring will greatly help in assessing the risk.

EARTHQUAKE PREPAREDNESS

Large earthquakes are so infrequent in the eastern United States that most people do not perceive a risk and are therefore unprepared for a damaging event. Simple precautions such as bolting bookcases to the wall, strapping water heaters to the wall, putting latches or bolts on cabinet doors, and maintaining an emergency supply of canned food, drinking water, and other essentials can prevent both loss and hardship. Brochures on earthquake preparedness are available from disaster services agencies and the American Red Cross. Earthquake insurance is commonly available in Ohio for a nominal additional fee on most homeowner policies. Such a policy might be a consideration, particularly for individuals who live in areas of Ohio that have previously experienced damaging earthquakes.

THE OHIO SEISMIC NETWORK

In early 1999, the first statewide cooperative seismic network, OhioSeis, became operational. This network uses broadband seismometers to digitally record earthquakes in Ohio and from throughout the world. The network was established with the primary purpose of detecting, locating, and determining magnitude for earthquakes in the state. These data not only provide information to the public after an earthquake but, after a long period of

monitoring, will more clearly define zones of highest seismic risk in the state and help to identify deeply buried faults and other earthquake-generating structures. The OhioSeis network was funded in part by the Federal Emergency Management Agency (FEMA) through the Ohio Emergency Management Agency as part of the National Earthquake Hazards Reduction Program (NEHRP). The stations are operated independently by volunteers as part of a cooperative agreement.

For additional information concerning earthquakes, contact:

Ohio Department of Natural Resources
Division of Geological Survey
4383 Fountain Square Drive
Columbus, OH 43224-1362
Telephone: 614-265-6988

This GeoFacts compiled by Michael C. Hansen - January 2000

REFERENCES

Downloaded from http://www.dnr.state.oh.us/geosurvey/geo_fact/geo_f03.htm

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APPENDIX 2

LANDSLIDES IN OHIO

OHIO GEOLOGY

GEOFACTS No. 8

OHIO DEPARTMENT OF NATURAL RESOURCES • DIVISION OF GEOLOGICAL SURVEY

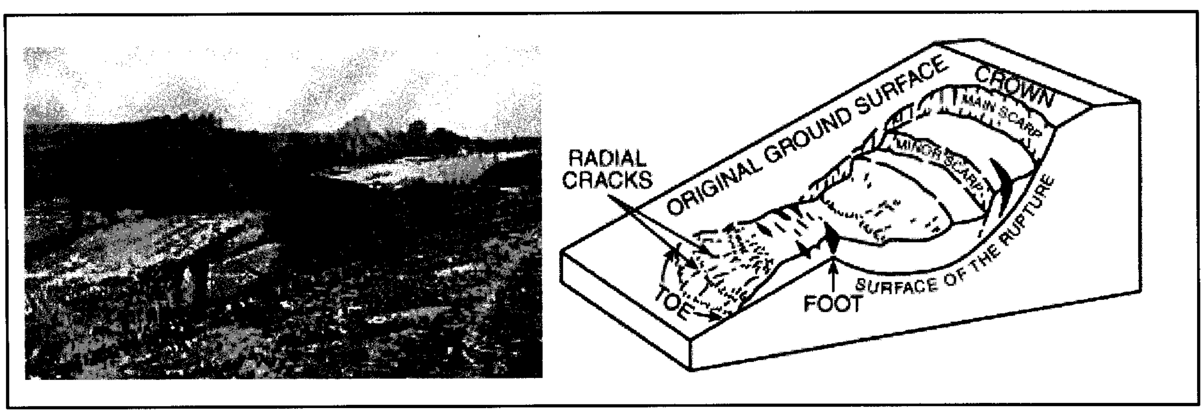
Landslides are a significant problem in several areas of Ohio. The Cincinnati area has one of the highest per-capita costs due to landslide damage of any city in the United States. Many landslides in Ohio damage or destroy homes, businesses, and highways, resulting in annual costs of millions of dollars. Upon occasion, they can be a serious threat to personal safety. On Christmas Eve 1986, an individual traveling in an automobile was killed by falling rock along U.S. Route 52 in Lawrence County in southern Ohio. Although this is Ohio's only recorded landslide fatality, there have been numerous near misses.

TYPES OF LANDSLIDES

The term landslide is a general term for a variety of downslope movements of earth materials. Some slides are rapid, occurring in seconds, whereas others may take hours, weeks, or even longer to develop.

Rotational Slump

A rotational slump is characterized by the movement of a mass of weak rock or sediment as a block unit along a curved slip plane. These slumps are the largest type of landslide in Ohio, commonly involving hundreds of thousands of cubic yards of material and extending for hundreds of feet.



Major Components of a Rotational Slump.

Rotational slumps have an easily recognized, characteristic form. The upper part (crown or head) consists of one or more transversely oriented zones of rupture (scarps) that form a stair-step pattern of displaced blocks. The upper surface of these blocks commonly is rotated backward (reverse slope), forming depressions along which water may accumulate to create small ponds or swampy areas. Trees on these rotated blocks may be inclined upslope, toward the top of the hill. The lower, downslope end (toe) of a rotational slump is a fan-shaped, bulging mass of material characterized by radial ridges and cracks. Trees on this portion of the landslide may be inclined at strange angles, giving rise to the descriptive terms "drunken" or "staggering" forest. Rotational slumps may develop comparatively slowly and commonly require several months or even years to reach stability; however, on occasion, they may move rapidly, achieving stability in only a few hours.

Earthflow

Earthflows are perhaps the most common form of downslope movement in Ohio; many of them are comparatively small in size. Characteristically, an earthflow involves a weathered mass of rock or sediment that flows downslope as a jumbled mass, forming a hummocky topography of ridges and swales. Trees may be inclined at odd angles throughout the length of an earthflow. Earthflows are most common in weathered surface materials and do not necessarily indicate weak rock. They are also common in *unconsolidated* glacial sediments. The rate of movement of an earthflow is generally quite slow.

Rockfall

A rockfall is an extremely rapid, and potentially dangerous, downslope movement of earth materials. Large blocks of massive *bedrock* may suddenly become detached from a cliff or steep hillside and travel downslope in free fall and/or a rolling, bounding, or sliding manner until a position of stability is achieved.

Most rockfalls in Ohio involve massive beds of sandstone or limestone. Surface water seeps into joints or cracks in the rock, increasing the weight of the rock and causing expansion of joints when it freezes, thus prying blocks of rock away from the main cliff. Weak and easily eroded clay or shale beneath the massive bed is an important contributing factor to a rockfall; undercutting in this layer removes basal support.

CAUSES OF LANDSLIDES

Landslides are not random, totally unpredictable phenomena. Certain inherent geologic conditions are a prerequisite to the occurrence of a landslide in a particular area. The presence of one or more of the following conditions can serve as an alert to potential landslide problems.

Steep slopes. All landslides move downslope under the influence of gravity. Therefore, steep slopes, cliffs, or bluffs are required for development of a landslide, especially in conjunction with one or more of the conditions listed below.

Jointed rocks. Vertical joints (fractures) in rocks allow surface moisture to penetrate the rock and weaken it. During periods of cold weather, this moisture freezes and causes the rock masses to be pried apart along the joint.

Fine-grained, permeable rock or sediment. These materials are particularly susceptible to landslides because large amounts of moisture can easily enter them, causing an increase in weight, reduction of the bonding strength of individual grains, and dissolution of grain-cementing materials.

Clay or shale units subject to lubrication. Ground water penetrating these materials can lead to loss of binding strength between individual mineral grains and subsequent failure. Excess ground water in the area of contact between susceptible units and underlying materials can lubricate this contact and thus promote failure.

Large amounts of water. Periods of heavy rainfall or excess snowmelt can saturate the zone above the normal water table and cause a landslide.

Although many areas of the state possess one or more of the above conditions, a landslide requires a triggering mechanism to initiate downslope movement. Events or circumstances that commonly trigger landslides in Ohio include:

Vibrations. Human-induced vibrations such as those from blasting, or even the passing of a heavy truck, in some circumstances, can trigger a landslide. Vibrations from earthquakes can trigger landslides, although no such occurrence has been documented in Ohio.

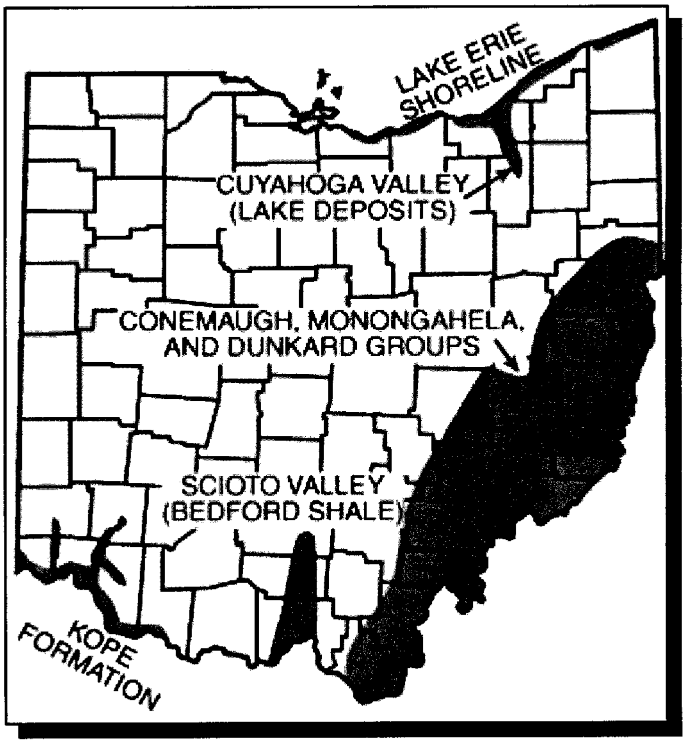
Oversteepened slope. Undercutting of a slope by stream or wave erosion or by human construction activities can disturb the equilibrium of a stable slope and cause it to fail. Addition of fill material to the upper portion of a slope can cause the angle of stability to be exceeded.

Increased weight on a slope. Addition of large amounts of fill, the construction of a building or other structure, or an unusual increase in precipitation, either from heavy rains or from artificial alteration of drainage patterns, can trigger a landslide.

Removal of vegetation. Cutting of trees and other vegetation on a landslide-prone slope can trigger failure. The roots tend to hold the rock or sediment in place and soak up excess moisture.

LANDSLIDE-PRONE AREAS OF OHIO

Landslides are rare or nonexistent throughout much of Ohio because of a lack of steep slopes and/or lack of geologic units prone to failure. Several areas of the state, however, experience frequent and costly landslides.



Areas of Ohio subject to severe slope failure.

Portions of eastern and southern Ohio are characterized by steep slopes and local relief of several hundred feet. In addition, *bedrock* of Mississippian, Pennsylvanian, and Permian ages, thick colluvium (deposits of broken and weathered *bedrock* fragments), and thick lake silts and outwash formed in association with Pleistocene glaciers make this area particularly prone to slope failures. The most slide-prone rocks in eastern Ohio are red mudstones ("red beds") of Pennsylvanian and Permian age. These rocks tend to lose strength when they become wet, forming rotational slumps or earthflows. About 85 percent of slope failures in this region are in red beds of the Pennsylvanian-age Conemaugh and Monongahela Groups.

Eastern Ohio also is subject to rockfalls. Thick, massive sandstones form steep cliffs in many areas of the region and, periodically, large blocks may suddenly fall or tumble downslope.

In the lower part of the Scioto River valley, thick colluvium developed on shales of Mississippian age, particularly the Bedford Shale, is prone to failure. Also prone to failure are lake clays and silts that accumulated in some valleys in this area when Pleistocene glaciers dammed the north-flowing preglacial Teays River system.

Portions of Cincinnati (Hamilton County) and surrounding counties where rocks of Ordovician age are exposed are prone to numerous and costly landslides in the form of rotational slumps and earthflows. The majority of *bedrock* slope failures are in the shale-dominated Kope Formation and to a lesser degree in the Miamitown Shale. Landslides tend to occur in the thick colluvium developed on these units when excessive hydrostatic pressure builds up in this zone.

The valley of the Cuyahoga River between Cleveland and Akron, in Cuyahoga and Summit Counties, is well known for rotational slumps in clays and silts deposited in lakes formed when glaciers of the Pleistocene Ice Age blocked various segments of the valley. The modern Cuyahoga River has cut through these deposits, leaving steep bluffs of unstable sediments along the valley walls. Many of these landslides tend to be concentrated on north-facing slopes where moisture retention is higher.

The eastern half of the Ohio portion of the Lake Erie shoreline, from Cleveland to Ashtabula, is characterized by *unconsolidated* glacial sediments such as till and lake clays and silts that are highly susceptible to wave erosion at the base of the bluff. Such erosion is accentuated during periods of high lake levels accompanied by large storms. The continual removal of slumped sediment by waves prevents natural achievement of stability of the slope. Many lakeshore homes, roads, and other structures have been destroyed in these areas, where bluff recession is as rapid as 7 feet per year.

HOW TO AVOID LANDSLIDES

Site selection for a home or other structure in a landslide-prone area of the state should include a determination of the underlying geologic materials and their susceptibility to failure. Geologic maps are a key resource for this. The presence of hummocky topography, steplike scarps, unusually inclined trees or fence posts, and seeps of water are all signs that the slope has undergone failure at some time in the past.

Precautions against slope failure include avoiding the following practices: excavating at the base of the slope, placing large quantities of fill on the upper part of the slope, removing vegetation, disrupting natural drainage patterns, and allowing water from downspouts or septic tanks to discharge onto a slope. In questionable areas, the services of a consulting geologist familiar with the problems of slope failure may be well worth the expense.

FURTHER READING

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This GeoFacts compiled by Michael C. Hansen - September 1995

REFERENCES

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APPENDIX 3

UNSTABLE SLOPES ADVISORY FOR SOLID WASTE LANDFILL FACILITIES

OhioEPA

State of Ohio Environmental Protection Agency

STREET ADDRESS: MAILING ADDRESS:

Lazarus Government Center
122 S. Front St.
Columbus, Ohio 43215

TELE: (614) 644-3020 FAX: (614) 644-3184
www.epa.state.oh.us

P.O. Box 1049
Columbus, Ohio 43216-1049

DSIWM Guidance 0586

Unstable Slope Advisory for Solid Waste Landfill Facilities

May 29, 2004

APPLICABLE RULES

MSW: OAC 3745-27-19(E)(1)(c)
ISW: OAC 3745-29-19(E)(1)(c)
RSW: OAC 3745-30-14(E)(1)(c)
Tires: OAC 3745-27-75(E)(19)
C&DD: OAC 3745-400-11(E)(1)

Cross-References:
#0660 *Geotechnical and Stability Analyses for Ohio Waste Containment Facilities*

PURPOSE

This document outlines the operational and construction practices of material placement for maintaining stable waste slopes and the structural integrity of engineered components.

APPLICABILITY

This document applies to operating municipal (MSW), industrial (ISW) and residual (RSW) solid waste landfills, scrap tire monofills, and construction and demolition debris (C&DD) landfills.

BACKGROUND

Operational and construction practices have a profound impact upon the stability of waste slopes and in maintaining the integrity of the engineered components. Excavated and constructed slopes (including waste slopes) can fail if sound operating and construction practices are not followed.

Several incidents involving failure of slopes and damage to engineered components have occurred at solid waste landfills around the state. Each incident can, in part, be attributed to construction and operational errors, specifically over-steep waste slopes. The operators at the facilities where these failures occurred placed waste at a grade that exceeded the shear resistance of the affected material, or the shear forces induced by waste placement exceeded the shear resistance of one of the geosynthetic and/or soil interfaces. Additionally, each of these facility operators incurred significant cost to assess and repair damage to the engineered components of the facility.

Slope stability analyses on final, interim and internal slopes are a requirement in the solid waste rules. All the landfill rules also require the owner or operator to maintain the integrity of the engineered components of the landfill facility and repair any damage to or failure of the components.

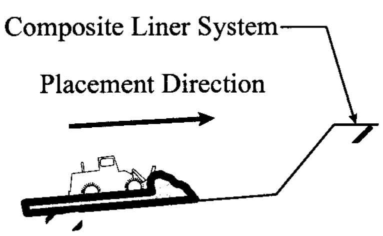
The following suggestions are not regulatory requirements but, adherence is highly recommended to help avoid slope failures, the resulting costly repairs to engineered components of the facility, violations for failing to maintain the integrity of the engineered components, and operational violations which could occur as a result of a failed engineered component.

PROCEDURE

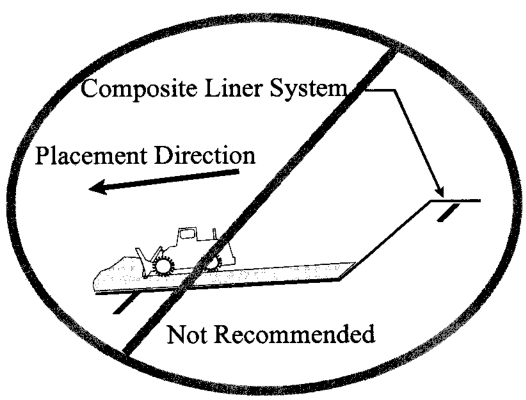
Construction

Drainage layer sand, frost protection material and the select waste layer should only be placed while advancing up slope relative to the bottom composite liner grade similar to that shown in Figure 1. This is especially true on perimeter containment berms. At Ohio facilities, placing drainage material from the top down or laterally across a containment berm has caused anchor trench pullout, ripped flexible membrane liners, and failure through the recompacted soil liner.

Figure 1



Recommended Placement Method



Waste Placement

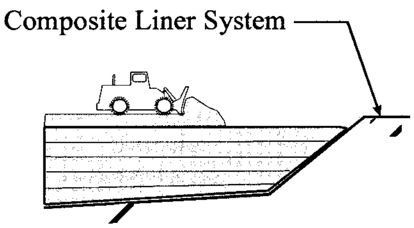
In cells where geosynthetics have not been installed (e.g. C&DD, RSW, scrap tire monfills) the maximum grade of waste placement should be determined from a slope stability analysis that incorporates appropriate shear strength values of the waste and the natural underlying materials. The shear strength of the natural materials should be obtained from testing site-specific natural material at site-specific normal stresses. For C&DD and RSW facilities, the maximum slope for the cap is 25%, DSIWM recommends waste placement does not exceed this slope.

In cells where geosynthetics have been installed, the geosynthetics are usually the weakest component (with the exception of some industrial wastes) and will dictate the maximum grade of waste placement. As with drainage layer sand, frost protection material and placement of the select waste layer, waste should initially be placed in thin nearly horizontal layers starting from the lowest area of the phase or cell and advanced up slope relative to the bottom composite liner grade (see Figure 1). Pushing waste in a direction that is down slope with the bottom liner grade can cause stresses in the geosynthetics or result in an interface failure that can compromise the composite liner system.

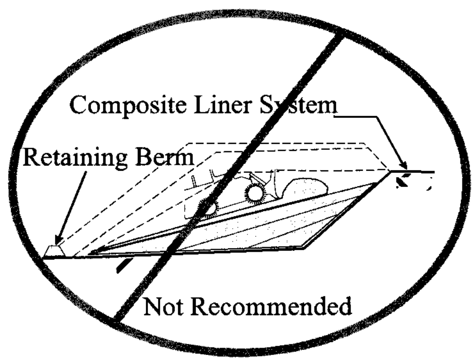
Waste should continue to be placed in thin nearly horizontal layers (see Figure 2) until sufficient normal stresses can be developed that will maintain the structural integrity of the liner system for waste placement at a steeper grade. This steeper slope can only be determined through a stability analysis which incorporates both the appropriate shear strength values of the waste and natural underlying materials as stated previously (for unlined cells), and the interface frictional values obtained from testing site-specific geosynthetics and soils at site-specific normal stresses. Waste placement at a steeper grade can also create failure planes through waste and where intermediate cover is placed.

The recommended placement method may require changes in phasing and construction of a haul road into the bottom of the cell, which in turn may require an alteration or modification to the PTI (or C&DD license), depending upon the extent of the changes. It should be pointed out that construction of a haul road into the bottom of a cell has its own attendant concerns for maintaining the integrity of engineered components, consequently its design and construction should be thoroughly evaluated.

Figure 2



Recommended Placement Method



Steep waste slopes have also been a cause of slope failure and destruction of composite bottom liner systems, resulting in significant remediation costs. The heterogeneous nature of MSW and the materials disposed in MSW landfills (such as ISW and RSW wastes), makes it very difficult to determine accurate and plausible shear strength values. ISW and RSW typically exhibit shear strength characteristics significantly less than that of MSW. One failure occurred in Ohio at a residual waste landfill with slopes of 5 horizontal to1 vertical (5:1) and resulted in waste material sliding into an adjacent uncertified cell. A slope of 3:1 is about the maximum feasible grade for MSW and about the maximum feasible final grade of a landfill given the limitations of the interface strengths with cap systems, equipment limitations, and difficulties with increased erosion and cover and cap maintenance. For detailed information on designing stable slopes see #0660 *Geotechnical and Stability Analyses for Ohio Waste Containment Facilities.*

Saturation

Saturation can dramatically affect shear strength. Failures have occurred through waste, intermediate covers on a steep slope, and in drainage layers on the side slope.

Slope stability analyses should evaluate saturated conditions. Selection of intermediate cover materials and placement should take into consideration the creation of failure planes. In another state, a slope failure occurred because a thick layer of wood chips was used as a cover material over a steep slope. The wood chips were eventually covered by subsequent layers of waste, but they had become saturated and eventually failed, resulting in a large waste slide. Granular drainage layer on the side slopes, left exposed during a long period of time, can become saturated and fail. The designer can account for the effects of exposure and saturation by designing the drainage layer to accommodate the maximum head predicted for the fifty year, one hour storm event. To mitigate saturation, the owner or operator can place the select waste layer (or a four foot thick lift of waste) up the exposed drainage layer on side slopes, if the slope stability analysis indicates waste placement will be stable.

Summary:

Operational and construction practices have significant impact on the stability of waste slopes and in maintaining the integrity of engineered components. Additionally, interim waste slopes are often the most critical slopes at landfills. Therefore, DSIWM recommends implementing the following practices at all landfills, as appropriate.

Drainage sand, frost protection material, select waste and initial lifts of waste should only be placed while advancing up slope relative to the grade of the bottom composite liner system.

In cells where geosynthetics have been installed, waste should be placed in thin nearly horizontal lifts (exclusive of the select waste layer).

The maximum grade of waste placement for interim and final slopes of waste should be determined from a stability analysis.

In general, waste slopes should not exceed 4:1 for C&DD and RSW, or 3:1 for MSW or ISW. However, given material limitations, the maximum allowable slope may need to be flatter.

Industrial and residual solid wastes should be evaluated on an individual basis to determine maximum waste placement grades for that particular waste and should not exceed 3:1.

The effects of saturation should be evaluated and measures taken to address the loss of shear strength that occurs.

Changes to the facility (e.g. a change in phasing or haul road construction) may require a permit alteration or modification or a license modification. Consult with the appropriate district office or license authority (for C&DD facilities) for additional information on modifications, alterations and license requirements.

POINT OF CONTACT

If you have questions regarding this document or would like additional information, please contact:

Central District Office DSIWM Supervisor
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