

Geotechnical and Stability Analyses for Ohio Waste Containment Facilities



Cover photos:

(Left) Division of Environmental Response and Revitalization - CERCLA closure site failure.

(Top Right) Division of Materials and Waste Management - commercial municipal solid waste landfill failure.

(Bottom Right) Division of Surface Water - captive wastewater ash impoundment berm failure.

ACKNOWLEDGEMENTS

This policy was revised by the following Ohio EPA, Geotechnical Resource Group members:

Annette DeHavilland, CO-DMWM

Michael Harris, SWDO-DMWM

Habib Kaake, NWDO-DMWM

Ned O'Loughlin, NWDO-DMWM

Jerry Parker, NEDO-DMWM

Brian Queen, SEDO-DMWM

This policy was originally written by the following Ohio EPA, Geotechnical Resource Group members: Annette DeHavilland, MS ENV E, PE, CO-DSIWM, Scott Heidenreich, MPH, Editor, NWDO-DSIWM, Dan Lukovic, MS Mining, CO-DHWM, Jerry Parker, NEDO-DSIWM, Laura Parkinson, CDO-DSIWM, Brian Queen, SEDO-DSIWM, and Virginia Wilson, PE, NEDO-DSIWM.

The writers wish to acknowledge contributions and advice from: Dr. Braja M. Das, Dean of the School of Engineering and Computer Science, California State University, Sacramento; Doug Evans, PE, formerly CO-DSIWM; Jessica Hirashima, formerly CO-DMWM; Jeff Meyer, PE, formerly SWDO-DSIWM; James Moseley, MSCE, PE, formerly NWDO-DSIWM; Dr. Patrick J. Fox, PE, Professor, Department of Civil & Environmental Engineering, Pennsylvania State University, University Park; David F. McCarthy, PE, Author of Essentials of Soil Mechanics and Foundations, Clinton, NY; Dr. Timothy D. Stark, PE, Professor of Civil Engineering, University of Illinois, Urbana; Dr. Edward Kavazanjian, PE, Professor of Civil and Environmental Engineering, Arizona State University, Tempe; Dr. Neven Matasovic, Principal, Geo-Logic Associates, Inc., Costa Mesa, California.

Geotechnical Resource Group (GeoRG)
Ohio Environmental Protection Agency
Columbus, Ohio 43216-1049

This page intentionally left blank.

TABLE OF CONTENTS

Acknowledgements	3
Forward (READ THIS FIRST, THE USE OF REQUIREMENTS VS. RECOMMENDATIONS, DEFINITIONS AND ACRONYMS, BASIC CONCEPTS OF SLOPE STABILITY)	ix
Chapter 1: The Need for Geotechnical Stability Analysis	
Why Are Geotechnical and Stability Analyses Needed	1-1
When Are Geotechnical and Stability Analyses Needed	1-2
What are Geotechnical and Stability Analyses Needed For	1-3
References	1-5
Chapter 2: Content of Geotechnical and Stability Analyses	
Introduction	2-1
Report Content (Organization , Subsurface Investigation , Materials Testing , Liquefaction Potential , Settlement and Bearing Capacity , Hydrostatic Uplift and Seepage , Deep-Seated Failures , Shallow Failures , Geotechnical Analyses)	2-1
Chapter 3: Subsurface Investigation	
Introduction	3-1
Reporting Subsurface Investigation Results	3-2
Conducting the Investigation (Preliminary Investigation , Site Characterization and Screening , Investigating Critical Layers and Compressible Layers , Example Table of Contents , Boring Log Checklist)	3-4
References	3-13
Chapter 4: Materials Testing Program Methods and Assumptions	
Introduction (Factors Affecting Validity and Accuracy)	4-1
Reporting	4-5
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)	4-6
Consolidated-Undrained Triaxial Compression Test (ASTM D 4767)	4-8
Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (ASTM D 2850)	4-10
Unconfirmed Compressive Strength of Cohesive Soil (ASTM D 2166)	4-12
Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method (ASTM D 5321)	4-13
Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method (ASTM D 6243).	4-15

Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D 3080)	
Conformance Testing	4-17
Shear Strength Envelopes - Example	4-22
References	4-25
Chapter 5: Liquefaction Potential Evaluation and Analysis	
Introduction	5-1
Reporting	5-2
Design Criteria / Factors of Safety	5-3
Liquefaction Evaluation (Soil Unit Screening Criteria , Factor of Safety , Liquefaction Impact Assessment)	5-5
Appendix 5-A Example	5-7
References	5-11
Chapter 6: Settlement Analyses	
Introduction	6-1
Reporting	6-1
Design Criteria	6-3
Total Settlement Analysis (Primary , Secondary , Differential)	6-4
Bearing Capacity	6-14
Appendix 6-A Examples (Total Settlement , Differential Settlement)	6-18
References	6-26
Chapter 7: Hydrostatic Uplift and Seepage Analyses	
Introduction	7-1
Reporting	7-1
Factor of Safety	7-3
Analysis (hydrostatic uplift , heaving , seepage potential/piping , erosion rate index)	7-5
Appendix 7-A Examples (hydrostatic uplift of a pump station , hydrostatic uplift at the sump , seepage analysis using FEM , estimation of soil erosion rate)	7-17
Appendix 7-B Determination of Total Head and the Concept of Seepage Forces	7-24
References	7-26
Chapter 8: Deep-Seated Failure Analysis	
Introduction	8-1
Reporting	8-3
Factors of Safety	8-5
Assigning Shear Strengths (drained , undrained , failed slope , waste , high moisture wastes , interface , seismic , specifying)	8-6
Accounting for the Effects of Water (phreatic , piezometric)	8-13

Analysis (Static [long term , short term , staged loading], Seismic [screening analysis])	8-15
Appendix 8-A Examples (static , seismic long term , short term)	8-21
References	8-53

Chapter 9: Shallow Failure Analysis

Introduction	9-1
Reporting	9-2
Factors of Safety	9-3
Assigning Shear Strengths	9-5
Assigning Transmissivities and Permeabilities	9-6
Analysis (Unsaturated , Saturated , Seismic , Equipment Loading , Gas Pressure, Anchoring)	9-10
Appendix 9-A Examples (Unsaturated , Saturated , Seismic , Equipment loading)	9-19
References	9-43

FOREWORD

READ THIS FIRST

This document is designed to assist owners of regulated *waste containment facilities* in determining stability. The information provided in this document may be applied to all plans, applications, or requests submitted to any division of Ohio EPA for approval, concurrence, or comment. This document 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 pressure or gravitational influence.

The information in this document 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 determining stability. However, the applicable statutes and rules should also be consulted directly. 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 document as applicable.

This document 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 document 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 document are intended to demonstrate how performance standards 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 document have been evaluated by Ohio EPA and have been shown to be useful for demonstrating that a *waste containment facility* will meet requirements for stability. Alternative methods or procedures may be used if they are fully documented as being valid and appropriate for demonstrating stability requirements.

THE USE OF REQUIREMENTS VS. RECOMMENDATIONS

This document 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.

This document 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."

DEFINITIONS AND ACRONYMS

Throughout this document 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 document.

AASHTO	American Association of State Highway and Transportation Officials.
ASCE	American Society of Civil Engineers.
ASTM	American Society for Testing and Materials.
<i>Bearing capacity</i>	The capacity of a soil or rock to support the applied load.
<i>Bedrock</i>	Solid rock underlying <i>unconsolidated soil units</i> . Syn: <i>consolidated stratigraphic unit</i> .
<i>Book values</i>	Values derived from charts, tables, or other generalized presentations of data found in textbooks, periodicals, and manuals. <i>Book values</i> 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 <i>book values</i> to estimate the shear strength of competent <i>bedrock</i> is likely to be appropriate.
<i>Borings</i>	Any means of mechanical penetration into the subsurface for the purposes of characterizing material properties or collecting material <i>samples</i> .
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
<i>Compressible layer</i>	Soil or fill materials that may settle after establishing a facility and may continue to settle after a facility has closed.
<i>Conformance testing</i>	Testing conducted before construction on <i>samples</i> 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

<i>Consolidated stratigraphic unit</i>	See: <i>Bedrock</i> .
CPT	Cone Penetrometer Test; a common in-situ test to measure the geotechnical properties of the soil.
<i>Critical layer</i>	A potentially liquefiable layer, or a thickness of soil or waste material that has a <i>drained</i> or <i>undrained shear strength</i> that may cause a failure if all or part of the mass of a facility were suddenly put in place. <i>Critical layers</i> may be only a few inches thick to tens of feet thick. <i>Critical layers</i> may include parts of one or more stratigraphic <i>soil units</i> .
CU	Consolidated-Undrained.
DERR	Division of Environmental Response and Revitalization
<i>Differential settlement</i>	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.
<i>Downdrag;</i>	The side force created along the length a buried element due to <i>differential settlement</i> . This force is also referred to as negative skin friction load.
<i>Drained conditions</i>	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.
<i>Drained shear strength</i>	The shear strength exhibited by a soil layer when no excess pore water pressure is present. <i>Drained shear strength</i> is used for conducting an effective stress analysis.
EPA	Environmental Protection Agency.
<i>Facility bottom</i>	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 “ <i>facility bottom</i> ” excludes <i>internal slopes</i> or <i>interim slopes</i> (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/ft² (psf) may be assigned *peak shear strength* during stability analyses, if appropriate.

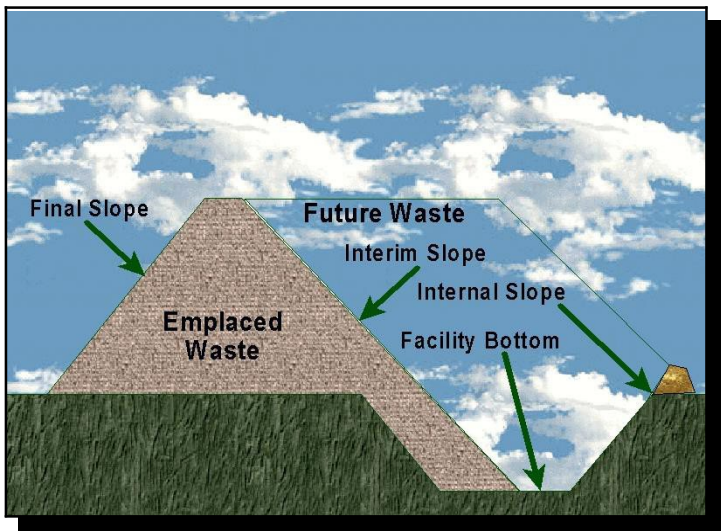


Figure f-1 An example of a typical landfill progression showing *internal, interim, and final slopes*, and the *facility bottom*. These types of slopes may also be present at other types of *waste containment facilities*.

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*.

<i>Interim slopes</i>	Slopes that exist at a <i>waste containment facility</i> because of daily filling or because a phase or unit has reached its limits, including cover soils. An <i>interim slope</i> will exist for only part of the facility life and is not part of the engineered components of the facility (see Figure f-1).
<i>Internal slopes</i>	Slopes excavated below grade and/or constructed using berms, including, as applicable, the liner/leachate collection system, <i>protective layers</i> , and other engineered components (see Figure f-1). Interfaces on <i>internal slopes</i> that exceed a grade of five (5) percent must be assigned <i>residual shear strength</i> during deep-seated failure analysis and may be assigned <i>peak shear strength</i> during shallow failure analysis, if appropriate.
<i>Large displacement shear strength</i>	The stable, post-peak shear stress measured in the same interface direct shear test corresponding to measured displacements usually on the order of 2 to 3 inches (depending on the capability of the testing device). Whereas residual shear strength is beyond the <i>large displacement shear strength</i> and is the steady state shear stress recorded after the strain is increased beyond the <i>peak shear strength</i> of a <i>specimen</i> (see Figure f-2).
<i>Lower quality data</i>	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 <i>lower quality data</i> include standard penetration tests (SPTs) and pocket penetrometers. Data obtained from <i>samples</i> that have been highly disturbed are also lower quality and nonreproducible.
MSW	Municipal solid waste.
OCR	Overconsolidation ratio; ratio of the maximum past effective stress to the current effective stress.
ODNR	Ohio Department of Natural Resources.
ODOT	Ohio Department of Transportation.
<i>Overall settlement</i>	The settlement of an entire <i>waste containment facility</i> , 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 below).

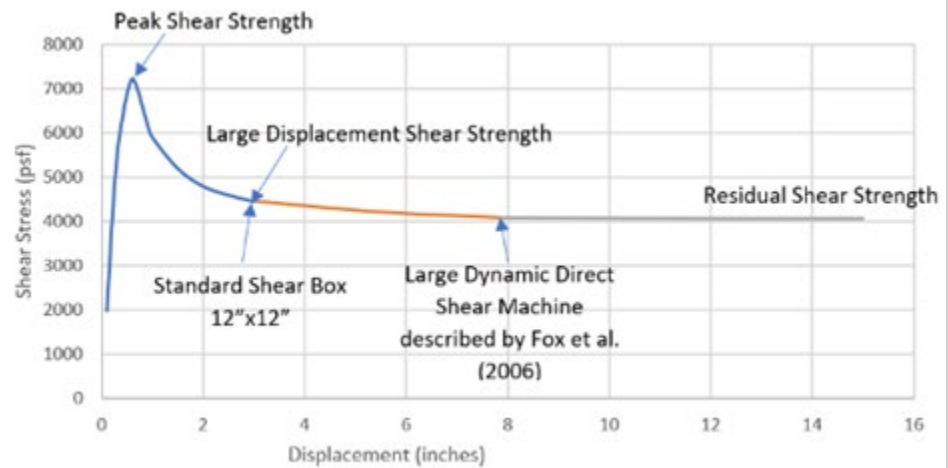


Figure f-2 Typical Shear Strength of Soil-Geosynthetic Interface by Direct Shear

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.

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).

<i>Protective layer</i>	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 <i>protective layers</i> 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.
<i>Responsible party</i>	The persons in control of the property, facilities, and activities that occur at a <i>waste containment facility</i> , including, but not limited to, applicant, permittee, owner, operator, or potentially <i>responsible party</i> (PRP).
RSL	Recompacted soil layer (liner or barrier layer depending upon context).
<i>Sample</i>	noun: Used in this manual to describe a volume of material from which <i>specimens</i> are prepared for testing. One <i>sample</i> may provide one or more <i>specimens</i> for testing. verb: Used in this manual to refer to the activities necessary to collect <i>samples</i> of materials.
<i>Saturated</i>	a: for shallow failure analysis: the <i>protective layers</i> 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 <i>protective layer</i> . When a <i>protective layer</i> is a leachate collection layer prior to waste placement, then <i>saturated</i> means the state when head exists due to the occurrence of a design storm. b: for laboratory methods: a <i>specimen</i> has, to the extent possible, all voids full of water. c: for subsurface conditions, one or more <i>soil units</i> , or part of a <i>soil unit</i> has most of the voids filled with water.
<i>Secondary consolidation</i>	See: <i>Secondary settlement</i> .
<i>Secondary settlement</i>	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).

<i>Soil stratigraphy</i>	The vertical and lateral or spatial arrangement of <i>soil units</i> at a facility.
<i>Soil unit</i>	a: A discrete layer or body of <i>unconsolidated</i> 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 <i>soil units</i> may be appropriate. Individual <i>soil units</i> might not be laterally continuous across a facility. b: a stratum of soil within the <i>soil stratigraphy</i> of the facility. Syn: <i>Unconsolidated stratigraphic unit</i>.
<i>Specimen</i>	A specific volume of material subjected to testing. For example, a volume of soil material trimmed out of a <i>sample</i> and placed into a triaxial compression apparatus to be tested for shear strength.
SPT geotechnical	Standard Penetration Test; a common in-situ test to measure the properties of the soil.
<i>Strain incompatibility</i>	The condition that exists when the displacement necessary to mobilize the <i>peak</i> internal or <i>peak</i> interface <i>shear strength</i> 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 <i>strain incompatibility</i> is not taken into account, it may cause computer modeling software to overlook the critical failure surface.
<i>Tensile strain</i>	The fractional change of an object's length due to tensile stress.
<i>Total settlement</i>	The settlement at any given point caused by the sum of immediate, <i>primary</i> , and <i>secondary settlement</i> .
<i>Unconsolidated</i>	In geology, used to differentiate between <i>bedrock</i> (<i>consolidated material</i>) and other materials such as weathered <i>bedrock</i> and soils (<i>unconsolidated material</i>). This is different from the geotechnical terms

of “*unconsolidated*,” “normally consolidated,” and “overconsolidated” used to describe the stress history of a *soil unit*.

*Unconsolidated
Stratigraphic Unit*

See: *Soil unit*.

Undrained conditions

The state that exists when a *soil unit* 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.

Coduto, Donald P., 2001, Foundation Design – Principles and Practices, Prentice Hall, Inc., 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:

$$\tau_f = c + \sigma \cdot \tan \phi \quad (\text{see [Figure f-3](#)})$$

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:

$$\alpha = 45^\circ + \frac{\phi}{2} \quad (\text{see [Figure f-3](#)})$$

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

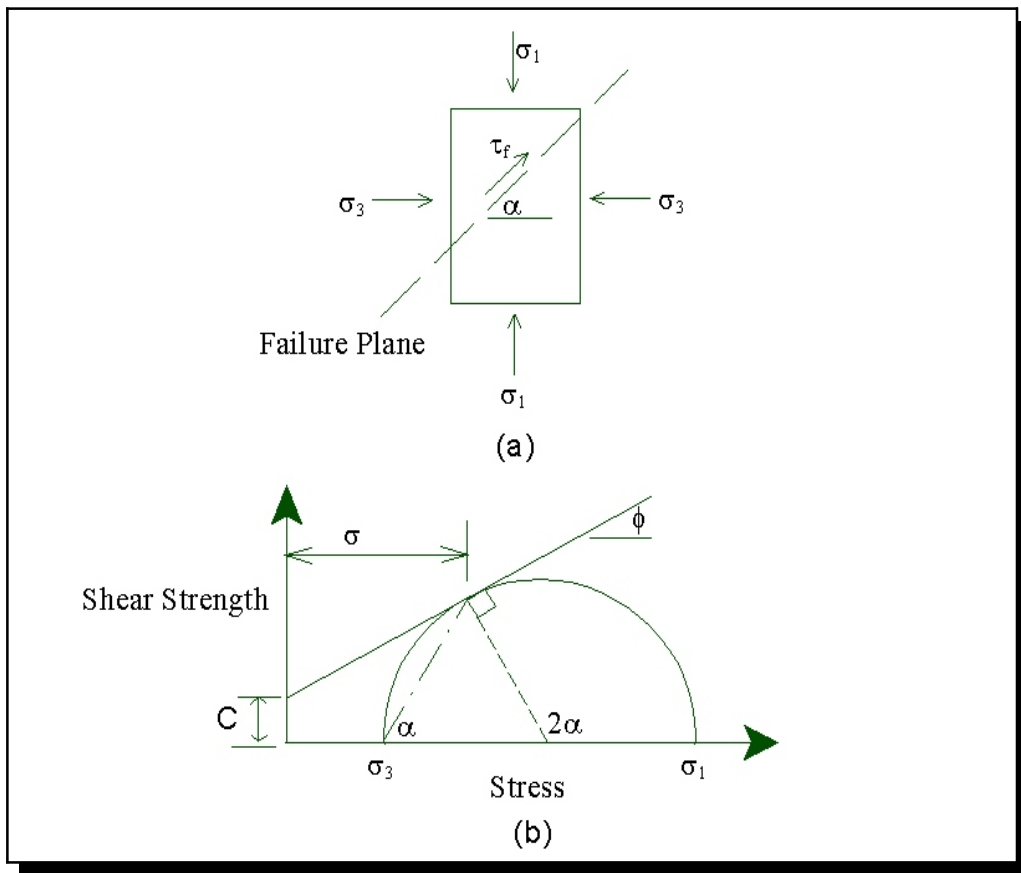


Figure f-3 Mohr-Coulomb envelope. (a) Soil element. (b) Shear strength envelope. Adapted from Abramson, et al, 1996, Slope Stability and Stabilization Methods, Figure 1.20, pg 37, and Holtz and Kovacs, 1981, An Introduction to Geotechnical Engineering, Figure 10.7a.

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.
- Fox, P. J., Nye, C. J., Morrison, T. C., Hunter, J. G., and Olsta, J. T. (2006). "Large dynamic direct shear machine for geosynthetic clay liners," *Geotechnical Testing Journal*, 29(5), 392-400.
- Holtz, R. D., and Kovacs, W. D., 1981, *An Introduction to Geotechnical Engineering*, Prentice Hall, Englewood Cliffs, New Jersey.

Chapter 1

THE NEED FOR GEOTECHNICAL STABILITY ANALYSIS

Table of Chapter Contents

Why Are Geotechnical and Stability Analyses Needed	1-1
When Are Geotechnical and Stability Analyses Needed	1-2
What are Geotechnical and Stability Analyses Needed For	1-3
References	1-5

Why Are Geotechnical and Stability Analyses Needed

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.

Groundwater 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.



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.

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 *waste containment unit*, and they 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.

When Are Geotechnical and Stability Analyses 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 *waste containment unit* 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 *waste containment unit*.
- It becomes apparent to the *responsible party* that the design in the authorizing document must be changed while construction is occurring.

When a *waste containment 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 should be promptly submitted to Ohio EPA for review.

What are Geotechnical and Stability Analyses Needed For

It is important for the designer to identify and assess 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 1-1). 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.

Note that construction and operational activities are common triggers for failures at *waste containment facilities*. These activities are often planned or performed independently of the design process and subsequently cause circumstances that were unforeseen during the design of the *waste containment unit*. An example is using heavy construction equipment to place drainage layer on top of geomembrane causing slope movements between interfaces resulting in residual shear strength at these interfaces.



Figure 1-2, Failed slope at an Ohio landfill between textured geomembrane and GCL covered with 1 ft. of pea gravel on 3 to 1 slope after being placed with a Caterpillar D8 (14.7 psi) pushing up slope. Some movement was seen during gravel placement but by the next day the slope had completely failed.

Table 1-1, Some natural and man-made factors that can adversely affect stability of a waste containment facility.

Type	Common Causes
Removal of toe support	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.
	Cuts and excavations (especially for an extended period of time or length or if unplanned); removal of retaining walls or sheet piles; excavation of waste; quarrying; borrowing soil.
	Draw-down or filling of bodies of water (e.g., ponds, lagoons); reduction in water levels after flooding.
Removal of underlying materials that provide support	Weathering ; underground erosion due to seepage (piping); solution of foundation materials from groundwater.
	Mining.
	Excavating.
	Degradation of waste; subsurface fire
	Differential settlement.
Decreasing the shear resistance of materials	Gas pressure underlying impermeable layers, thus decreasing or removing contact.
	Water infiltration into cracks, fissures, and interfaces of engineered components; freeze/thaw cycles ; leachate recirculation or injection of liquids into the waste mass; hydrostatic uplift.
	Stress release.
	Structural degradation.
	Post-peak strength
Increasing shear stresses from addition of mass/weight	Changing a material with a different interface shear strength
	Weight of water from precipitation (e.g., rains, snow, ice); water leakage from culverts, water pipes, and sewers; increase in water levels in lagoons and ponds due to flooding or increasing water levels; leachate recirculation or injection of liquids into the waste mass.
	Accumulation of debris from natural movements (falls and slides), stockpiling or overfilling; equipment staging; construction of haul roads; increasing the density or loading rate of waste; relocating or regarding of waste.
	Pushing downwards soil or waste from the top of a slope.
Increasing shear stresses from additional forces	Using heavy construction equipment to place drainage layer/ <i>protective layer</i> on top of geomembrane causing slope movements between interfaces resulting in residual shear strength at these interfaces.
	Earthquakes ; blasting; vibrations from long trains passing by a location, dynamic loading from equipment travel.
	Freeze/thaw cycles. Expansion of clays.

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

Table of Chapter Contents

Introduction	2-1
Report Content (Organization , Subsurface Investigation , Materials Testing , Liquefaction Potential , Settlement and Bearing Capacity , Hydrostatic Uplift and Seepage , Deep-Seated Failures , Shallow Failures , Geotechnical Analyses)	2-1

INTRODUCTION

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 *waste containment facility*.

REPORT CONTENT

Organization of the Report

A report which is organized in a logical manner and well referenced will enhance Ohio EPA's review of the information. Elements of a well-organized report include the following:

- Drawings and cross-sections are referenced to the *waste containment facility* coordinate system, and northing and easting lines are shown.
- Use of tabs and a consistent format for the data will make it easier to find information when needed.
- If drawings, cross-sections, or other items are not duplicated throughout the report, the referenced items are easy to locate and marked to show the appropriate information.
- Present the geotechnical and stability analyses in the same order as in the rules, if any, or this document.

Any drawings or cross-sections referred to in this document 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 narrative summary describing the results of the different phases of the subsurface investigation including identification of the *critical layers* and *compressible layers*, elevations of the temporal high *phreatic surfaces*, and the elevations of the temporal high *piezometric surfaces*, cross-sections, plan view of *boring* and *sample* locations, tables of field and laboratory results.
- A justification of the assumptions used to plan the site characterization field investigation.
- A justification for the selection of *samples* for laboratory testing and test methods used.
- Supporting information such as figures, drawings, or references used, *boring* logs, and laboratory test results.

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 description of collection and transport procedures for *samples*.

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.

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

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 including a list of *borings* and whether it passed or did not pass the liquefaction screening evaluation.
- The results of the liquefaction potential analysis for any soils that do not pass the liquefaction screening evaluation.
- The results of the liquefaction impact assessment for any *soil unit* with a factor of safety against liquefaction < 1.0 .
- Electronic copies of the input files or the spreadsheets used to perform the liquefaction calculations.

Settlement Analyses and Bearing Capacity

The results of the settlement analysis for the *waste containment 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.
- Information used from the subsurface investigation and a description of the model used to evaluate settlement.
- Plan view drawings and critical cross-sections.
- Calculations of the amount of settlement and tensile strain on the geomembrane.

- Electronic copies of the input files or the spreadsheets used to perform the settlement calculations.
- If vertical sump risers are present, a *bearing capacity* analysis.

Hydrostatic Uplift and Seepage Analysis

Ohio EPA recommends the results of the hydrostatic uplift and seepage analyses 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 tabular summary of the results of the hydrostatic uplift analysis and the seepage analysis.
- A summary of the worst-case scenarios used and a justification for selection of the critical cross-sections used.
- Information used from the subsurface investigation.
- A plan view showing the location of the critical cross-sections with the limits of excavation, top of liner/added geologic material, and an isopach of temporal high *phreatic surfaces* and temporal high *piezometric surfaces*.
- Cross-sections and calculations for hydrostatic uplift, heave analysis, piping/internal erosion analysis, and erosion rate index analysis.
- Electronic copies of the input files or the spreadsheets used to perform the calculations.

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 and table of the results of the deep-seated failure analysis on all the analyzed cross-sections.
- Justification for the internal and interface shear strengths used for the various components, selection of the critical cross-sections, and the

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 units* at the site will not become *saturated* during construction and filling of a *waste containment unit*.

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.

slip surface.

- Tables summarizing the input parameters.
- Information used from the subsurface investigation.
- A plan view showing the location of the critical cross-sections.
- Cross-sections and calculations for long term and short-term stability for *internal*, *interim*, and *final slopes* and seismic stability for *final slopes*. Electronic copies of the input files or the spreadsheets used to perform the calculations.

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 narrative summary and table of the results of the shallow failure analysis on all the analyzed cross-sections.
- Tables summarizing the input parameters.
- A plan view showing the location of the critical cross-sections.
- Cross-sections and calculations for static stability for *internal* and *final slopes* and seismic stability for *final slopes*, and any other evaluations of shallow translational and rotational failure mechanisms at the *waste containment unit*.
- Electronic copies of the input files or the spreadsheets used to perform the calculations.

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 *waste containment 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).
- *Saturated protective layers* causing head in the drainage layers during the design storm.

Regarding *undrained conditions*, 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. In general, 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 units* at the site will not become *saturated* during construction and filling of a *waste containment unit*. 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.

CHAPTER 3

SUBSURFACE INVESTIGATION

Table of Chapter Contents

Introduction	3-1
Reporting Subsurface Investigation Results	3-2
Conducting the Investigation (Preliminary Investigation , Site Characterization and Screening , Investigating Critical Layers and Compressible Layers , Example Table of Contents , Boring Log Checklist)	3-3
References	3-12

INTRODUCTION

This chapter provides information to use when characterizing the *stratigraphic units* (soil units) beneath a proposed or existing *waste containment facility* during a subsurface investigation in Ohio, as well as recommendations for reporting results of the investigation to Ohio EPA.

The purpose of a subsurface investigation is to determine whether or not the soils beneath a *waste containment facility* exhibit the properties necessary to ensure that the facility will remain stable under static and seismic conditions during construction and operation and after closure; this is ultimately accomplished by evaluating and characterizing subsurface conditions. The results of the investigation and subsequent field and lab testing and evaluation should be a complete and comprehensive interpretation of the *soil stratigraphy* that illustrates the lateral and vertical extent of all *soil units* expected to be beneath the proposed *waste containment facility*.

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; this step may or may not involve visiting the site, depending on ease of access and availability of information about the site. Second, a site-specific investigation is conducted through mobilization of exploratory equipment to the site to allow for collection of *samples* and/or data 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 *waste containment facility* will also be determined during this step. Finally, *samples* that have been gathered from the *critical* and *compressible layers* will be evaluated to produce *higher quality data* through in-depth, hands-on analysis and possibly lab testing.

Soil and stratigraphy 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 different in-situ soils. Any *piezometric surfaces* associated with *bedrock* that may affect the *waste containment facility* during excavation, construction, operations, or closure must also be identified.

Part of this investigation involves identifying all *critical layers* beneath the *waste containment facility*. A *critical layer* is any thickness of soil material that has a *drained* or *undrained shear strength* suspected of being susceptible to a failure if all or part of the mass of a *waste containment unit* were suddenly put in place. *Critical layers* may be only a few inches thick to tens of feet thick and may include parts of one or more *soil units*. Any layer that is potentially liquefiable must also be identified as a *critical layer*.

In addition, all *compressible layers* should be identified during the subsurface investigation. *Compressible layers* are soil or fill materials that are likely to settle as the *waste containment unit* is constructed and even after the *unit* has closed. *Bearing capacity* and settlement potential should be calculated for all *compressible layers*. Analysis must show that bearing failure will not occur and that the engineered components of the *waste containment unit* will meet minimum design requirements during construction, operation, closure, and post-closure of the *waste containment 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).

REPORTING SUBSURFACE INVESTIGATION RESULTS

This section describes the information that should be submitted to demonstrate the results of the subsurface investigation. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report:

Conclusions

- A narrative summary describing the results of the different phases of the subsurface investigation.
- A description of evidence indicating the presence of *critical layers* or *compressible layers* at the proposed landfill site as well as any evidence of historic mass soil or *bedrock* movement and how these findings from the preliminary investigation were incorporated into site characterization.
- A graphical depiction of the findings of the site characterization in the form of multiple cross-section drawings. Drawings should indicate locations, elevations, and thicknesses of potential *critical layers* and *compressible layers*, as well as a synthesis of temporal high *phreatic* and

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.

piezometric surfaces.

- One or more tables summarizing the results of field and laboratory testing. *Samples* should be indexed by *boring* log number and depth interval, and those representing *critical layers* and *compressible layers* should be identified. Relative laboratory test record numbers should be indicated here as well.

Basis of Design

- A narrative justifying the assumptions used to plan site characterization field investigation and formulate conclusions depicted in cross-sections and data tables.
- Publicly available information that went into the preliminary investigation, including zoning classifications, man-made potential explosive gas pathways, information recovered through previous explorations as available, and the sources of this information
- Plan view drawings, which should indicate:
 - *Boring* locations, both planned and executed, indicating ground surface elevations and coordinates
 - Significant geologic features, such as floodplains, parks, wildlife areas, public and private water supply wells, limits of drinking water source areas, faults that have had displacement in Holocene time, and surface and underground mines
 - The extent of *critical layers*, *compressible layers*, temporal high *phreatic* and *piezometric surfaces*, and anticipated limits of waste placement and *waste containment facility* boundaries
 - Locations of provided cross-sections
- A description of reasoning behind selection of *samples* for laboratory testing as well as methodologies used to carry out laboratory testing. This summary must clearly indicate which data were applied to soil calculations as described in later chapters in this manual.

Supporting Information

- Any figures, drawings, or references relied upon during the analysis should be properly cited.
- *Boring* logs and laboratory test records should be included as well.

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 *waste containment 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:

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.

- 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) Division of Geological Survey and Division of Mineral Resources Management, the Department of Transportation (ODOT), and 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).

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 geologic history and *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 obtained and validated and are therefore usable as part of the results of this investigation. 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, they can be used, as appropriate, along with newly acquired data. However, any data that cannot be verified must be excluded from use.

Finally, it is important that the field staff that will be performing the following step of the investigation, site characterization and screening, are aware of the findings of the preliminary

investigation. Awareness of the anticipated field conditions can give direction to the field investigation and help guide analytical decisions made in the field. For example, if the preliminary investigation indicates a possible *critical layer* at a certain depth interval that varies in thickness throughout the site, the field staff may assign greater significance to a compressible material of seeming minimal thickness discovered in a *sample* recovered during the field exploration.

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, such as depth, elevation, thickness, frequency in the *boring* and on site, unusual odor or color, and relative stiffness, plasticity, and water content, but are unreliable for determining the best obtainable definitive measurement of any given characteristic; *samples* will be retained and then submitted to laboratory testing to obtain a more precise evaluation of the soil materials existing on site.

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 *waste containment facility*. Extending the investigation deeper to ensure the *waste containment 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 *waste containment unit* must be measured as well, with multiple measurements being taken throughout the process of making the soil *borings* or test pits.

Critical layers may be relatively thin (on a scale of inches as opposed to feet). As mentioned previously, 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*. Applying an average strength value across part or all of a *soil*

Site topography can reveal evidence of historic slope failures and the potential for failures in the future. Some indications that downslope movement has occurred or may in the future are:

- 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),
- Coastal or river erosion
- 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.

unit is unacceptable because it may mask the lower strength values of the *critical layer(s)* within a *soil unit*.

Applying an average of the characteristics of *compressible layers* should also be avoided, so *differential* and *total settlement* can be properly evaluated. 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 *waste containment unit*. To accomplish this, initial exploratory *borings* should be performed at a minimum frequency of one (1) *boring* for every four (4) acres on a uniform grid across the *waste containment 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, wastewater outlet structures, or dikes having relatively little freeboard).

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).

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

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), cone penetration tests (CPTs), or another testing and *sampling* method should be conducted at each *boring*.

An SPT involves inserting a split-spoon sampler into the *boring* at each *sample* depth and repeatedly dropping a 140-pound weight (hammer) a distance of 30 inches on top of the sampler until it has penetrated through the soil for its entire length (18 to 24 inches). The number of hammer blows is recorded for each 6 inches of penetration, and the total number of hammer blows for the final foot of penetration (or the middle foot, when the sampler measures 24 inches) is known as the N-value. When blow count exceeds 50 blows for a 6-inch interval, hammer blows are stopped, and the

sampler is noted to have hit “refusal.” The N-value can be standardized to N_{60} through the application of several empirical correction factors for hammer efficiency, borehole diameter, sampler type, and rod length.

The CPT is another method of field testing through which a *sample* is not recovered. For this test, a hydraulic ram pushes a long, cylindrical sleeve with a conical tip into the ground, by which resistance is measured with instruments connected to the cone and cylinder.

To find thin *critical layers*, initial exploratory *borings* conducted on a grid pattern should be *sampled* and logged continuously to a minimum depth of 50 feet below the bottom of the deepest excavation. *Borings* may need to be *sampled* and logged continuously to an even greater depth if there are indications that deeper *critical layers* or *compressible layers* may affect the stability of the *waste containment unit*.

If CPTs are used, though blow counts will not be measured, laboratory testing as 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 determine the lateral and vertical extent of each layer. Additionally, a representative number of *samples* of each layer under investigation 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*.

Characterizing *critical layers* is generally accomplished using investigation and *sampling* methods



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

Residual soil and weathered *bedrock* are often weak because of preexisting discontinuities such as faults, bedding surfaces, foliations, cleavages, sheared zones, relict joints, and soil dikes. These discontinuities also often lose shear strength when *saturated*. Slickensided seams, or joints worn down and smoothed through friction, may also be present in residual soil and weathered *bedrock* slopes.

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 reasonably accurate *higher quality data* to characterize the *waste containment facility*, at least ten (10) percent of undisturbed *samples*, or a minimum of three (3) undisturbed *samples*, whichever is more, from each *critical layer* and each *compressible layer* encountered should be collected and laboratory tested from the *borings* that pass through those layers. These relatively undisturbed *samples* are collected with thin-walled samplers (an example of which would be a Shelby tube) that are pushed as opposed to hammered into the ground so as to minimize disturbance.

In Ohio, there are some soils, formations, and features that can be easily determined and are notorious as being potentially unstable or inherently difficult to make acceptable for waste containment. These are:

- Peat and loess, which are poor foundation materials.
- Bedford shale, Kope formation, and Miamitown shale, which are susceptible to mass movement.
- Weathered shales and mudstones of the Conemaugh, Monongahela, and Dunkard groups, which are susceptible to mass movement.
- Lake deposits, which are susceptible to mass movements.
- Cliffs of thick, massive sandstone, which are sources of falling rocks.
- Karst terrain, indicative of the presence of caves and occurrences of sinkholes, which are unstable foundations (risk of void collapse and of flooding from high groundwater table).
- Subsurface mines, which can be unstable foundations.

Consolidation parameters must be obtained from all *compressible layers* to analyze *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*. Field staff must be sure to note site elevation and coordinates of each *boring* location to ensure consistency between the depths and of the *critical layers* and *compressible layers* as observed in the field.

Laboratory testing and analyses should include, but are not limited to, Atterberg limits, 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 considered during shear testing. Consolidation testing must be conducted to provide information for estimating immediate settlement, *primary settlement*, and *secondary settlement* associated with the *waste containment unit* 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 subjected to compaction testing during the site investigation. Soils being considered for construction that are expected to exhibit the

weakest shear strengths should be tested. This information will contribute to stability analyses as discussed in later chapters.

Table 3-1, Example subsurface investigation report table of contents.

Section	Section Title
1.0	Summary and Conclusions
1.1	Preliminary Investigation Results
1.2	Site Characterization Results
1.3	Investigation of Critical and Compressible Layers Results
2.0	Basis of Design
2.1	Summary Narrative
2.2	Summary Tables
3.0	Supporting Information
3.1	Topographical Maps
3.2	Cross-sections
4.0	Field Test Results
4.1	Field Test Type 1
	Results
	Methods
4.2 etc.	Field Test Type #...
	Results
	Methods
4.3	Boring Logs
5.0	Laboratory Test Results
5.1	Laboratory Test Type 1
	Results
	Methods, QA/QC, Data Validation, etc.
5.2 etc.	Test Type #...
	Results
	Methods, Laboratory QA/QC, Data Validation, etc.

Table 3-2, Suggested components of a *boring log*.

Geotechnical Boring Log Checklist		
	Project Information	☐
	• Site Name	☐
	• Site Address	☐
	• Site and weather conditions	☐
	• Consulting firm name	☐
	• Field engineer name	☐
	• Drilling Company	☐
	• Equipment type/description	☐
	• Drilling method (e.g., hollow stem)	☐
	• Hammer type and efficiency	☐
	○ Date of last calibration	☐
	• Auger and rod diameter	☐
	• Equipment operator/driller name	☐
	• Driller helper name	☐
	Boring Information	☐
	• Boring name/number	☐
	• Northing and easting location coordinates	☐
	• Surface elevation surveyed to the nearest tenth of a foot	☐
	• Boring offset, as needed	☐
	• Local Terrain	☐
	• Date and time started	☐
	• Date and time finished	☐
	• Date, time, and elevation of groundwater	☐
	○ Groundwater depth should be measured when first encountered, end of each day, and before backfilling	☐
	• Note: drill chatter	☐
	• Note: void encountered	☐
	• Note: boring refusal	☐
	Sample Information	☐
	• Sample number	☐
	• Sample depth	☐
	• Test method	☐
	• SPT blow count for 6"	☐
	• SPT N-value	☐
	• Recovery per drive length	☐

	• Additional tests and results (e.g., pocket penetrometer or field shear vane)	☐
	• USCS Symbol	☐
Geotechnical Boring Log Checklist, continued		
	• USCS description, including,	☐
	○ Consistency	☐
	○ Color	☐
	○ Soil Type	☐
	○ Moisture content	☐
	○ Identification of unconsolidated stratigraphic units and thicknesses	☐
	○ Stratigraphic features, such as layering, interbedding, or weathering	☐
	○ For fine-grained, field descriptions of plasticity or dilatancy	☐
	○ Additional descriptors (e.g., presence of organic matter or odor)	☐
	Additional but helpful	☐
	• Density/consistency key	☐
	• Any Additional Notes	☐

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.
- Bardet, J., 1997, Experimental Soil Mechanics. Prentice-Hall, New Jersey.
- Coduto, Donald P., 2001, Foundation Design – Principles and Practices, Prentice Hall, Inc., New Jersey
- Edil, T. B., 2002, Soil Engineering for Non-Soils Engineers and Technicians, Course Notebook, Section 2, Subsurface Explorations, University of Madison, Wisconsin.
- Holtz, R. D., and Kovacs, W. D., 1981, Introduction to Geotechnical Engineering, Prentice Hall, Inc., Englewood Cliffs, New Jersey
- McCarthy, D. F., 2002, Essentials of Soil Mechanics and Foundations: Basic Geotechnics. 6th Edition, Prentice Hall Inc., Englewood Cliffs, New Jersey.
- Peck, R. B., Hansen, W. E., and Thornburn, T. H., 1974, Foundation Engineering, 2nd Edition, John Wiley & Sons, Inc. New York.

Chapter 4

MATERIALS TESTING PROGRAM METHODS AND ASSUMPTIONS

Table of Chapter Contents

Introduction (Factors Affecting Validity and Accuracy)	4-1
Reporting	4-5
One-Dimensional Consolidation Properties of Soils (ASTM D 2435)	4-6
Consolidated-Undrained Triaxial Compression Test (ASTM D 4767)	4-8
Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (ASTM D 2850)	4-10
Unconfined Compressive Strength of Cohesive Soil (ASTM D 2166)	4-11
Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method (ASTM D 5321)	4-12
Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method (ASTM D 6243).	4-14
Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D 3080)	4-17
Conformance Testing	4-18
Shear Strength Envelopes - Example	4-22
References	4-25

INTRODUCTION

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 unit* 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 unit*. 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 One-Dimensional Consolidation Properties of Soils (ASTM D 2435),
- Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils (ASTM D 4767),
- 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 Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D 3080).

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).

Factors Affecting the Validity and Accuracy of Soil Shear Strength Testing

Sample Quality

The variability observed in soil shear strength testing results is due to many factors, some of which are self-compensating. 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).

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).

Ohio EPA recommends quantifying sample disturbance with the method discussed in this chapter and using that information to determine what test should be performed and whether additional *samples* should be taken.

Normal Loads and Failure Planes: Developing the Failure Envelope

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.

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](#) for more details). When entering this data into a modeling program a Shear / Normal function is often used. It is important to use the normal loads used in testing and the shear values determined by the testing.

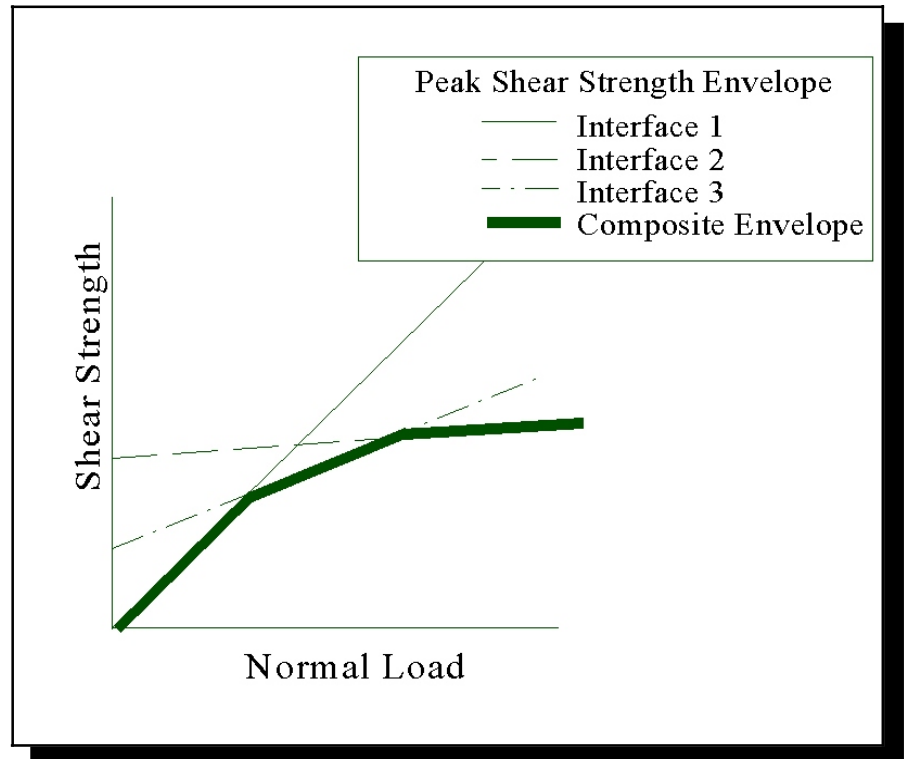


Figure 4-1: Example of a compound peak shear strength envelope for a multi-layered engineered component of a waste containment unit.

Shear strength values may be estimated from empirical correlations for structural fill, recompacted soil components, and foundation materials, unless those materials are considered a *critical layer*. References to the documents where the empirical correlations originate should be provided. Ohio EPA may request a copy of any document referenced. See [Chapter 8](#) Deep-Seated Failure Analysis for more information on *critical layers*.

Site-specific shear strength envelopes should be developed for all *critical layers* found at a site using the methods discussed in this chapter. If laboratory methods are used, Ohio EPA recommends collecting sufficient quantities of soil from the *critical layer* in three separate locations to run a consolidated undrained (CU) tests (using at least 3 separate confining pressures) and a one-dimensional (1D) consolidation tests for each location. The confining pressures used to conduct the CU tests should vary between the three locations. At least two of the confining pressures used in the CU tests should be approximately equal to the effective stress on the *samples* at the time of *sampling* and at least three normal stresses should be more than the preconsolidation stress (σ'_p) of the *samples* being tested. The shear strength envelopes determined by these tests should be compared and, as appropriate, one envelope may be developed for design purposes. One of the appropriate methods to develop this envelope is to plot the normal stresses versus the shear stress and plot the envelope so that 2/3s of the points fall above the line representing the envelope.

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 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 unit* should be included in the subsurface investigation report. The subsurface investigation report is described in [Chapter 3](#) Subsurface Investigation. At a minimum, the following information about materials testing results should be reported to Ohio EPA whenever it is conducted:

- A narrative and tabular summary of the scope, extent, and findings of the material testing,
- A description of collection and transport procedures for *samples*,
- The specimen quality designation (SQD) should be reported for all *samples* tested with the one-dimensional consolidation tests.
- 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 *waste containment unit*.

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.

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 and to quantify sample disturbance. This test will provide the effective stress-void ratio ($\log \sigma'-e$ 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.

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.

effective stress (σ'), and the *secondary compression* coefficient (C_α). The compressibility (M_v), the permeability coefficient (k) (Bardet, J., 1997), 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 unit*.

This test is highly subject to sample quality. The *specimens* should be *saturated* and from a sample tube that has been tested using the One Dimensional Consolidation test and found to have a specimen quality designation (SQD) of either A or B, i.e. volumetric strain of $< 2\%$ for the volume change during laboratory reconsolidation to the effective vertical stress (σ'_{vo}) on the sample (Terzaghi, 1996) or the change in void ratio between initial loading and a load application equal to the σ'_{vo} (Δe) divided by the initial void ratio (e_o) of < 0.07 for soils with an over consolidation ratio (OCR) of < 2 or $\Delta e/e_o < 0.05$ for OCRs between 2 and 4 (DeGroot, 2005) (see Table 1).

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 *waste containment facility*, each layer should be tested to be able to calculate the *differential* and *total settlement* for the *waste containment unit* properly. In addition, enough *samples* from each *compressible layer* should be tested to be able to identify lateral and vertical differences in consolidation and compressibility parameters. For example, if the *waste containment 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.

In the case of over consolidated clays, a better evaluation of recompression parameters may be obtained by imposing an unload-reload cycle after the preconsolidation pressure has been defined.

During testing, the load should be changed after the consolidation caused by the current load reaching 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 soils. 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 *waste containment unit* that will be filled later with water, waste, or other materials; or if the *unit* 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. The specimen quality designation (SQD) for each *specimen* being tested should be reported.

Table 4-1: Quantification of sample disturbance based on specimen volume change during laboratory reconsolidation to σ'_{vo}

Specimen quality designation (SQD) (Terzaghi 1996)		$\Delta e/e_o$ criteria (Lunne 1997)		
Volumetric Strain at σ'_{vo} (%)	SQD	OCR = 1 – 2 $\Delta e/e_o$	OCR = 2 – 4 $\Delta e/e_o$	Rating*
< 1	A	<0.04	<0.03	Very good to excellent
1–2	B	0.04–0.07	0.03–0.05	Good to fair
2–4	C	0.07–0.14	0.05–0.10	Poor
4–8	D	>0.14	>0.1	Very poor
> 8	E			

* Refers to use of *samples* for measurement of mechanical properties. (DeGroot, 2005)

Where

σ'_{vo} = In situ effective stress

Δe = Change in pore volume between initial pore volume and pore volume when *sample* is loaded to in situ effective stress

e_o = initial pore volume

OCR = Over Consolidation Ratio

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 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

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.

applicable to the design of the *waste containment unit*, the test should be run using stress conditions that are similar to the expected worst-case field conditions for the *unit*. 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 *waste containment unit*. Ohio EPA also recommends using this test when stability of the *waste containment unit* 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 *unit* 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 over consolidated 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 *waste containment unit* 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 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.

This test is highly subject to *sample* quality. The *specimens* should be *saturated* and from a sample tube that has been tested using the One Dimensional Consolidation test and found to have a specimen quality designation (SQD) of either A or B, i.e. volumetric strain of < 2% for the volume change during laboratory reconsolidation to the effective vertical stress (σ'_{vo}) on the *sample* (Terzaghi, 1996) or the change in void ratio between initial loading and a load application equal to the σ'_{vo} (Δe) divided by the initial void ratio (e_o) of < 0.07 for soils with an over consolidation ratio (OCR) of < 2 or $\Delta e/e_o < 0.05$ for OCRs between 2 and 4 (DeGroot, 2005) (see Table 1).

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 triaxial chamber. The *specimen* is then subjected to an axial load until the *specimen* fails. No drainage is allowed to occur during the test.

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 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 *waste containment facility* that are or may become *saturated*. If *specimens* are partially 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 *waste containment 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.

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*.

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 D2166)

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.

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.

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 units* 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 *waste containment 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.

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.

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.

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.

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

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 *large*

displacement shear strengths for all interfaces with a geosynthetic that are part of the *waste containment unit* 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*

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.

strengths for each interface and material are near each other, but a wide range of *large displacement shear strengths* exists, either the lowest *large displacement shear strength* measured will need to be used, or *specimens* comprising all the layers in a composite system will need to be tested.

The test must be continued until the *large displacement shear strength* has been determined once the full displacement of the direct shear device has occurred 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 *large displacement shear strength* may not be necessary. However, even here, it should be carefully considered whether knowing the *large displacement shear strength* of the interfaces is necessary for current or future design needs.

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.

Data Validation

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 recompactd 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.

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.

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.

Samples of geosynthetics used for testing interface shear strength should be selected from the geosynthetic rolls that will be used at the *waste containment unit* or from rolls that represent the materials that will be used at the *unit*. 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.

Large Displacement Shear Strength versus Residual Shear Strength:

Large displacement shear strength is considered the minimum shear strength of an interface after being sheared for a distance of about three inches which is typically achievable in a shear box with a minimum dimension of 12 inches. Residual shear strength is the minimum shear strength of an interface after being sheared. This may occur after a few inches or a few feet but is generally not reached in laboratory testing. Large displacement shear strength is acceptable for modeling purposes.

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

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 *waste containment 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 *large displacement shear strengths* of interfaces with GCL, and for determining the internal *peak shear strength* and *large displacement shear strength* of a GCL.

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.

The test must be continued until *large displacement shear strength* has been determined once the full displacement of the direct shear device has occurred or can be conservatively estimated.

Data Validation

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 recompactd 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 *waste containment unit*. 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 *waste containment unit*. 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 *waste containment unit*. An example of this would be choosing *samples* of needle-punched GCL from a roll created just before replacing the needles.
- The hydrating of GCL test *specimens* should be performed 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

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 (0.1 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 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).

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.

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 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.

- 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.

For designs where it is critical to know the *large displacement shear strength* of a material, an alternative is to repeatedly return the shear device 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 *large displacement shear strength* for soils and many types of interfaces (Stark and Poepfel, 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 *waste containment unit*, 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.

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 triaxial chamber. The *specimen* is then subjected to an axial load until the *specimen* fails. No drainage is allowed to occur during the test.

CONFORMANCE TESTING

Conformance testing is conducted on materials that will be used for constructing a *waste containment unit*. *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 are thoroughly tested during the subsurface investigation to identify *critical layers*. 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](#) Subsurface Investigation for more information about investigating and testing in situ foundation materials).
- Materials that will be used for structural fill, added geologic material, or recompacted soil layers (RSL) may have been tested during the subsurface investigation. To confirm these soils are appropriate to use, the design engineer may elect to undergo *conformance testing* on these types of materials 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](#) Subsurface Investigation 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 *waste containment unit*. 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 *large displacement 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/*large displacement*) 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.
- 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 *large displacement 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.

Large displacement 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 *waste containment 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](#)). If this assumption was used, then *large displacement 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 *large displacement shear strength* envelopes, select the *large displacement 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 in [Appendix 4-A](#).

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 *large displacement shear strengths* exists, either the lowest *large displacement shear strength* measured will need to be used, or *specimens* comprising all the layers in a composite system will need to be tested.

APPENDIX 4-A 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 two (2) feet of 1×10^{-7} cm/sec recompacted soil liner (RSL) overlain with a geosynthetic clay liner (GCL), 60 mils thick textured high density polyethylene (XGM), 8 once nonwoven geotextile (NW-GT) and a one-foot-thick drainage layer of layer of #8 gravel (DG) on top. The RSL was tested twice internally using remolded *samples* and the Consolidated Undrained Triaxial Test with Pore Pressure Measurement (CU). The GCL was tested twice internally and twice for both its interfaces using ASTM D 6243. The rest of the interfaces were tested twice using ASTM D 5321.

Table 4-2; Peak Shear Strengths

	GCL internal test 1	GCL internal test 2	RSL vs. GCL test 1	RSL vs. GCL test 2
Normal Stress (psf)	Peak Strength (psf)	Peak Strength (psf)	Peak Strength (psf)	Peak Strength (psf)
1000	2243	1899	650	656
5000	5496	4694	2770	2674
12000	7074	6367	5542	5161
23500	9808	11925	9349	9174
	GCL vs. XGM test 1	GCL vs. XGM test 2	XGM vs. NW-GT test 1	XGM vs. NW-GT test 2
Normal Stress (psf)	Peak Strength (psf)	Peak Strength (psf)	Peak Strength (psf)	Peak Strength (psf)
1000	457	444	814	920
5000	2927	2794	3142	3524
12000	4818	5352	7244	7063
23500	7907	7662	10465	10417
	NW-GT vs. DG test 1	NW-GT vs. DG test 2		
Normal Stress (psf)	Peak Strength (psf)	Peak Strength (psf)		
1000	1229	1225		
5000	3884	3939		
12000	8556	8617		
23500	13557**	13455**		

* Values approximated from remolded *samples* tested with CU with pore pressure measurements

**Test equipment maxed out before reaching *peak shear strength*

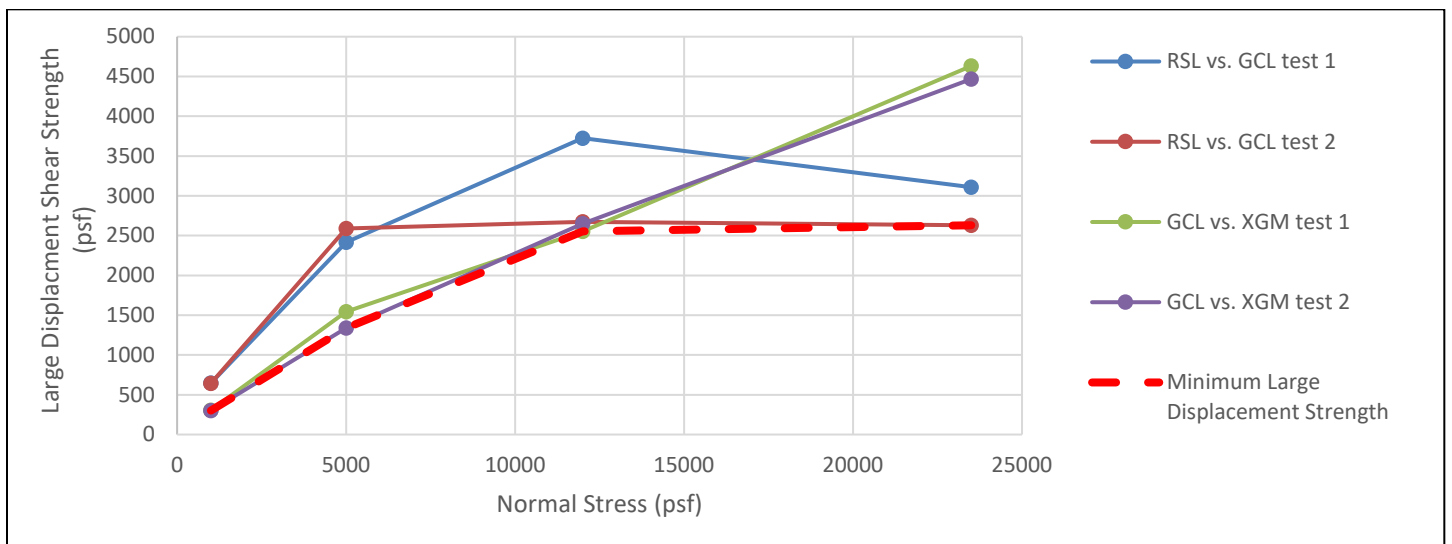
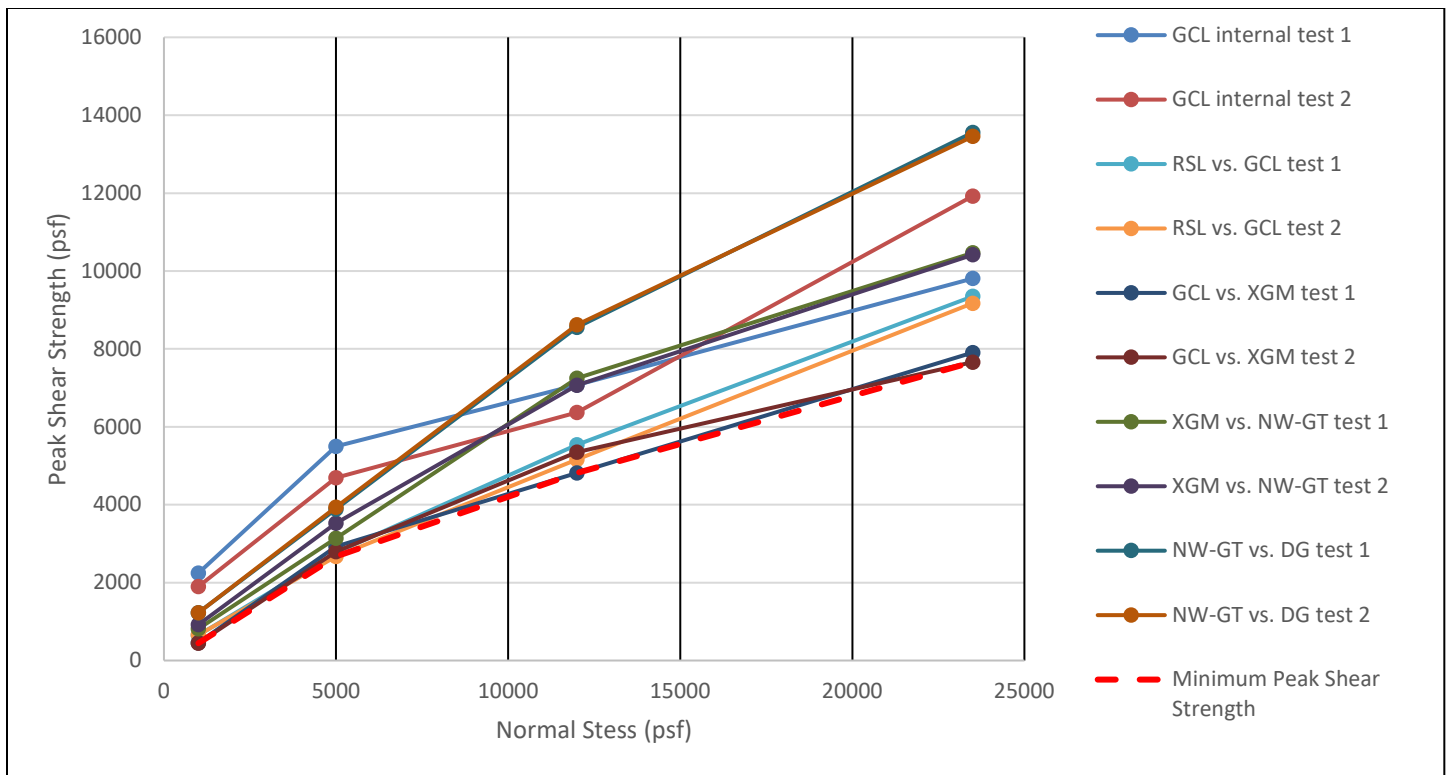
Lowest peak sheer stress at each normal stress highlighted in yellow

Table 4-3; Large Displacement Shear Strengths

	GCL internal test 1	GCL internal test 2	RSL vs. GCL test 1	RSL vs. GCL test 2
Normal Stress (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)
1000	1425	1137	646	644
5000	913	491	2415	2588
12000	1477	1229	3725	2673
23500	1805	3332	3109	2630
	GCL vs. XGM test 1	GCL vs. XGM test 2	XGM vs. NW-GT test 1	XGM vs. NW-GT test 2
Normal Stress (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)
1000	307	301	432	505
5000	1544	1341	1371	1444
12000	2555	2649	2703	2882
23500	4632	4468	4191	4430
	NW-GT vs. DG test 1	NW-GT vs. DG test 2		
Normal Stress (psf)	Large Displacement Strength (psf)	Large Displacement Strength (psf)		
1000	1105	1215		
5000	2934	3378		
12000	7882	7744		
23500	***	***		

***Test equipment maxed out before reaching Large Displacement Shear Strength

For design purposes the large displacement shear strength of the tests with the lowest peak shear strength should be used. The values are highlighted in yellow. Even though the RSL had the lowest shear strength at one of the normal loads it should be model with peak shear strength for design purposes.



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.
- Bardet, J., 1997, Experimental Soil Mechanics. Prentice-Hall, New Jersey.
- Duncan, J. M., and Wong, K. S., 1983, "Use and Mis-Use of the Consolidated-Undrained Triaxial Test for Analysis of Slopes," Proceedings of 25th Anniversary of the Venezuelan Geotechnical Society, Caracas, Venezuela, pp. 131 - 142.
- Fox, P. J., Rowland, M. G., and Scheithe, J. R., 1998, "Internal shear strength of three geosynthetic clay liners," Journal of Geotechnical and Geoenvironmental Engineering, Vo. 124, No. 10, pp. 933-944.
- Fox, P. J., Stark, T. D., and Swan, Jr. R. H., 2004, "Laboratory Measurement of GCL Shear Strength," Advances in Geosynthetic Clay Liner Technology: 2nd Symposium, STP 1456, Mackey, R.E. and von Maubeuge, K., ed., ASTM International, West Conshohocken, Pennsylvania, in press.
- Gilbert, R. B., Scranton, H. B., and Daniel, D. E., 1997, "Shear Strength Testing for Geosynthetic Clay Liners," Testing and Acceptance Criteria for Geosynthetic Clay Liners, STP 1308, L. W. Well, ed., ASTM International, West Conshohocken, Pennsylvania, pp 121-135.
- Gilbert, R. B., 2001, "Peak versus Residual Strength for Waste Containment Systems," Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.
- Jamiolkowski, M., Ladd, C. C., Germaine, J. T., and Lancellotta, R., 1985, "New Developments in Field and Laboratory Testing of Soils," Proceedings of 8th International Conference of Soil Mechanics and Foundation Engineering, San Francisco, Vol. 1, pp. 57 - 153.
- Ladd, C. C., and Foott, R., 1974, "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 100, No. GT7.
- Stark, T. D. and Poeppel, A. R., 1994, "Landfill Liner Interface Strengths from Torsional Ring Shear Tests," Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 3, pp. 597 - 615.
- Stark, T. D., 2002, Conference call with Ohio EPA, GeoRG on 1/31/02.
- Thiel, R., 2001, "Peak vs Residual Shear Strength for Landfill Bottom Liner Stability Analyses," Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.
- Tripplet, E. J. and Fox, P. J., 2001, "Shear Strength of HDPE Geomembrane/Geosynthetic Clay Liner Interfaces," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 6, pp. 543-552.

Chapter 5

LIQUEFACTION POTENTIAL EVALUATION AND ANALYSIS

Table of Chapter Contents

Introduction	Error! Bookmark not defined.
Reporting	5-2
Design Criteria / Factors of Safety	5-3
Liquefaction Evaluation (Soil Unit Screening Criteria , Factor of Safety , Liquefaction Impact Assessment)	5-4
Appendix 5-A Example	5-6
References	5-11

INTRODUCTION

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 unit* liquefies.

Soil liquefaction may occur during strong earthquake shaking in loose, *saturated* cohesionless *soil units* (sands and silts) with poor drainage 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 layer 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*. (after Richardson et al, 1995)

Liquefaction evaluation is a three-tiered process consisting of *soil unit* screening, simplified *soil unit* liquefaction potential analysis, and liquefaction impact assessment. These assessments should be performed in succession and the design engineer may end the evaluation once one of the tiers concludes that the *waste containment facility* is unlikely to be damaged by liquefaction.

REPORTING

This section describes the information that should be submitted to demonstrate that a *waste containment facility* is not susceptible to liquefaction. Ohio EPA recommends that the following information be included in its 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.

Conclusions

- A narrative summary of the findings of the liquefaction *soil unit* screening evaluation including all foundation materials evaluated and as applicable the findings of the simplified *soil unit* liquefaction potential analysis, and the findings of the liquefaction impact assessment.
- Provide a list of *borings* that only go through soils that pass the liquefaction *soil unit* screening evaluations and a list of *borings* that go through soils that do not pass the liquefaction *soil unit* screening evaluations.

Calculations

- Liquefaction Potential Analysis - If the liquefaction *soil unit* screening evaluation identifies soils that do not pass the liquefaction *soil unit* screening evaluations, then the following information should be included in the report:
 - A description of the methods used to calculate the factor of safety against liquefaction for *soil units* susceptible to liquefaction including a rationale for selecting the maximum expected horizontal ground acceleration and earthquake magnitude,
 - For each *boring* that goes through a soil that does not pass the liquefaction *soil unit* screening evaluation, provide a *boring* log that includes a column indicating the soil's depth, the soil's Standard Penetration Test (SPT) blow count normalized to an overburden pressure of approximately 1 tsf (2000 psf) and a hammer energy ratio or hammer efficiency of 60% ($(N_1)_{60}$), the soils factor of safety against liquefaction (FS_L), and other relevant information.
 - Plan views of the *waste containment facility* that include the northings and eastings showing the location of each boring.

- Liquefaction Impact Assessment - If any *soil units* are found to have an FS_L of < 1.0 the following information should be included in the report:
 - A description of the methods used to determine permanent deformations due to liquefaction-related movements.
 - A table summarizing the resulting permanent deformations due to liquefaction-related movements.
 - A description of the methods used to determine the post liquefaction static stability for *final slopes* assuming the liquefiable layers are at *large displacement shear strengths*.
 - A table summarizing the results of the post liquefaction static stability calculations for *final slopes* assuming the liquefiable layers are at *large displacement shear strengths*.
 - Output results of the post liquefaction static stability calculations for *final slopes* assuming the liquefiable layers are at *large displacement shear strengths*.
 - Plan views of the *waste containment facility* that include the northings and eastings, the lateral extent of the potentially liquefiable layers with an FS_L of < 1.0 , and the limits of the *waste containment layer(s)*,
 - Cross-sections of the *waste containment facility* showing *soil units*, full depictions of the potentially liquefiable layers, and the following:
 - Location of engineered components of the *waste containment facility*,
 - Depth and extent of foundation material indicating potentially liquefiable materials with an FS_L of < 1.0 , and
 - Temporal high *phreatic surfaces* and *piezometric surfaces*,
- Electronic copies of the input files or the spreadsheets used to perform the liquefaction calculations

Any figures, drawings, or references relied upon during the analysis should be properly cited. These figures, drawings, or references may be requested separately for review by Ohio EPA.

DESIGN CRITERIA / FACTORS OF SAFETY

The following design criteria and factors of safety should be used unless superseded by rule when demonstrating that a *waste containment unit* will not be adversely impacted by the effects of liquefaction:

At least one of the following design criteria should be met at all locations beneath the *waste containment unit*:

- All *soil units* pass liquefaction *soil unit* screening,
- All *soil units* have a factor of safety against liquefaction (FS_L) ≥ 1.00
- Permanent deformations of the *waste containment unit* due to liquefaction related movements are limited to less than 15 cm and post liquefaction static stability for *final slopes* assuming the liquefiable layers are at *large displacement shear strengths* $FS \geq 1.50$

The use of higher factors of safety against liquefaction or not allowing permissible deformation may be warranted if a failure would have a catastrophic effect upon human health or the environment (i.e., slide mass is likely to contact an occupied structure, public road, stream, lake or high-quality wetland or differential settlement due to liquefaction is likely to cause excessive strain see [Chapter 6 Settlement](#)).

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

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

LIQUEFACTION EVALUATION

Liquefaction Soil Unit Screen Criteria

Evaluate all *soil units* below the *waste containment facility* using the following screening criteria (note each 18-inch or 24-inch Standard Penetration Test should be evaluated separately):

Soils located in Ohio (these criteria may not apply in areas of higher seismicity) can be considered passing if at least one of the following are met:

- It is deeper than 50 feet below the existing ground surface,
- The the Standard Penetration Test (SPT) blow count normalized to an overburden pressure of approximately 1 tsf (2000 psf) and a hammer energy ratio or hammer efficiency of 60%, $(N_1)_{60}$, ≥ 22 .
- it is a cohesive soil with fines content greater than 30 percent and fines that either (1) are classified as clays (e.g., CH, CL, SC, and GC) based on the Unified Soil Classification system or (2) have a Plasticity Index (PI) greater than 30 percent.
- the soil is above the anticipated seasonal water table with a percent saturation of less than 80%.

For all *soil units* below the *waste containment facility* that do not pass the above criteria, evaluate the factor of safety against liquefaction (FS_L)

Liquefaction Potential Analysis - Factor of Safety Against Liquefaction (FS_L)

Determine the factor of safety against liquefaction for all *soil units* not passing the above Liquefaction Soil Unit Screening Criteria. Methods for determining the factor of safety against liquefaction are discussed in various publications such as Richardson et al 1995 and Youd et al 2001, design engineers may reference these publications to determine how to perform these calculations. Alternatively, design engineers may use one of the many affordable computer programs that are available to perform these calculations.

Liquefaction Impact Assessment

For all *waste containment facilities* over *soil units* with an $FS_L < 1.0$ calculate the vertical and lateral permanent liquefaction deformation of the *soil units* and the post liquefaction static stability of the *facility*.

Permanent Liquefaction Deformation

Methods for determining the vertical and lateral deformation are discussed in various publications as well as in Richardson et al 1995, design engineers may reference these publications to determine how to perform these calculations.

If the *waste containment unit* design deforms more than 15 cm, the *unit* should be redesigned. This redesign often consists of removing liquefiable materials beneath the *waste containment facility*.

Post Liquefaction Static Stability

The post liquefaction static stability calculations should assume the *soil units* with a factor of safety less than 1.00 are at *large displacement shear strengths*. Refer to [Chapter 8](#) Deep-Seated Failure Analysis for more information on static stability calculations.

If the *waste containment unit* design does not meet a 1.50 factor of safety for post liquefaction static stability, the *unit* should be redesigned. This redesign often consists of removing liquefiable materials beneath the *waste containment facility*, reducing *interim* and external slopes, adding benches to slopes, or increasing the size of external berms.

APPENDIX 5-A Example

A five-million-gallon concrete biosolids storage tank for an anaerobic digester facility is proposed at a location near Dayton Ohio. *Borings* at the site show various soil types including loose *saturated* sand at the proposed location. The *waste containment unit* was designed to be 16 feet in depth (5 feet above ground) and 240 feet in diameter.

Liquefaction Soil Unit Screening Criteria

There were eight *borings* installed for geotechnical and hydrogeologic purposes at the site (Borings B1 through B8). All eight *borings* at the site went through potentially liquefiable layers so a Simplified Liquefaction Potential Analysis was performed for each boring. For this example, only Boring B3 the *boring* with the lowest factor of safety against liquefaction (FS_L) has been included. For Boring B3 the top of the water table was at a depth of 4 feet.

Table 1: Liquefaction Potential Analysis - Factor of Safety Against Liquefaction (FS_L) Boring B3

Starting Depth	Material description	USCS Classification	SPT blows	$(N_1)_{60}$	FS_L
1	Medium, Moist, Brown, LEAN CLAY WITH SAND	CL	2 / 3 / 3	13.92	Passes Screening
2.5 ▼	Very Loose, Moist, Brown, SANDY SILT	SM	2 / 2 / 1	6.96	Passes Screening
4	Medium Dense, Wet, Brown, CLAYEY SAND	SC	5 / 6 / 6	27.85	19.48
5.5	Stiff, Moist, Brown and Gray, SANDY LEAN CLAY Grading to LEAN CLAY	CL	5 / 6 / 7	30.17	Passes Screening
7			5 / 6 / 7	30.17	Passes Screening
8.5	Stiff, Moist, Brown, SILT	ML	4 / 5 / 5	22.45	Passes Screening
10	Stiff, Moist, Brown, SANDY LEAN CLAY Grading to LEAN CLAY	CL	5 / 5 / 5	22.66	Passes Screening
11.5			5 / 7 / 6	28.03	Passes Screening
13	Stiff, Moist, Brown, SANDY SILT	ML	5 / 6 / 8	28.85	Passes Screening
14.5	Very Loose, Wet, Brown, POORLY GRADED SAND WITH CLAY	SP-SC	1 / 2 / 4	12.60	3.09
16			2 / 2 / 4	12.13	2.89

Chapter 5 - Liquefaction Potential of Evaluation and Analysis

Starting Depth	Material description	USCS Classification	SPT blows	(N ₁) ₆₀	FS _L
17.5	Loose, Wet, Brown, WELL GRADED SAND WITH CLAY	SW-SC	2 / 4 / 4	15.61	3.61
19			3 / 7 / 7	26.42	7.04
20.5	Very Loose, Wet, Brown, POORLY GRADED SAND	SP	3 / 2 / 1	6.13	1.64
22			2 / 2 / 1	5.96	1.58
23.5			2 / 2 / 3	9.66	2.14
25			2 / 2 / 2	7.52	1.77
26.5			1 / 1 / 1	3.67	1.17
28			2 / 3 / 1	7.17	1.67
29.5	Very Loose, Wet, Brown, POORLY GRADED SAND WITH CLAY	SP-SC	3 / 2 / 2	7.01	1.62
31			1 / 2 / 1	5.14	1.34
32.5			2 / 2 / 3	8.39	1.82
34	Medium Dense, Moist, Brown, POORLY GRADED SAND WITH CLAY	SP-SC	4 / 8 / 9	29.45	7.71
35.5			7 / 8 / 7	25.49	5.43
37	Medium Dense, Wet, Brown, POORLY GRADED SAND	SP	6 / 5 / 4	15.01	2.91
38.5			5 / 6 / 5	18.03	3.48
40	Medium Dense, Wet, Brown, CLAYEY GRAVEL WITH SAND	GC	8 / 10 / 9	30.60	Passes Screening
41.5			9 / 8 / 9	26.93	6.17
43	Medium Dense, Moist, Brown, SANDY LEAN CLAY	CL	3 / 5 / 6	17.15	Passes Screening
44.5			3 / 5 / 7	18.42	Passes Screening
46	Soft, Moist, Gray, SANDY LEAN CLAY	CL	2 / 2 / 4	9.07	Passes Screening
47.5			2 / 2 / 4	8.94	Passes Screening
49	Very Stiff, Moist, Brown, SANDY LEAN CLAY	CL	4 / 15 / 16	45.52	Passes Screening
50.5	Medium Dense, Moist, Brown, SILTY SAND	SM	9 / 16 / 19	50.69	Passes Screening
	End of Boring				

Calculations (Boring B3 SPT test depth 26.5 feet)

Equation 5-1: Standard Penetration Test (SPT) blow count normalized to an overburden pressure of approximately 1 tsf (2000 psf) and a hammer energy ratio or hammer efficiency of 60%, $(N_1)_{60}$ (Youd et al 2001)

$$(N_1)_{60} = N_{SPT} C_N (ER/0.6) C_B C_R C_S$$

Where

N_{SPT} = N value determined by standard penetration test = Y + Z

Y = number of blow counts in second 6 – inch interval

Z = number of blow counts in third 6 – inch interval

C_N = overburden correction = $(P_a / (\sigma_{vo}'))^{0.5}$ with a maximum value of 1.7

P_a = standardized pressure = 1 tsf (2000 psf)

σ_{vo}' = effective overburden pressure

ER = drill rod energy ratio as determined by ASTM D 4633

C_B = borehole diameter correction (use $C_B = 1.0$ for 2.5 - 4.5 inch, $C_B = 1.05$ for 6 inch, or $C_B = 1.15$ for 8 inch)

C_R = rod length correction (use $C_R = 0.75$ for < 10ft, $C_R = 0.8$ for 10 – 13ft, $C_R = 0.85$ for 13 - 20ft, $C_R = 0.95$ for 20 - 33ft, or $C_R = 1.0$ for 33 – 98ft)

C_S = sampling method correction (use 1.0 for sampler with liners for samplers without liners value ranges from 1.10 to 1.30.)

$$(N_1)_{60} = 2 \cdot 1.061 \cdot (0.91/0.6) \cdot 1 \cdot 0.95 \cdot 1.2 = 3.67$$

$N_{SPT} =$	2.0	SPT results 1 / 1 / 1	1+1 = 2
$\sigma'_{vo} =$	1776	Depth of SPT = 26.5 ft Average layer weight of soil = 120 pcf Depth of water table = 4 ft Layer weight of water = 62.4 pcf	$(26.5 \cdot 120 - ((26.5 - 4) \cdot 62.4) = 1776$
$C_N =$	1.061	$P_a = 2000$ psf	$((2000) / 1776)^{0.5} = 1.061$
ER =	0.91	Drill rod energy ratio as determined by ASTM D 4633 was 0.91	
$C_B =$	1.0	Borehole diameter was $4\frac{1}{4}$ inches	2.5-4.5 inch =1.0
$C_R =$	0.95	Rod length correction 3 x 10 ft rods	$C_R = 0.95$ for 20-33ft

$C_s =$	1.2	No liner used	Samplers without liners C_s ranges from 1.10 to 1.30 use 1.2
---------	-----	---------------	--

Equation 5-2: Cyclic Resistance Ratio for a 7.5 magnitude earthquake ($CRR_{7.5}$) (Youd et al 2001)

$$CRR_{7.5} = 1/(34 - (N_1)_{60}) + (N_1)_{60}/135 + 50/[10 - (N_1)_{60} + 45]^2 - 1/200$$

$$CRR_{7.5} = 1/(34 - 3.67) + 3.67/135 + 50/[10 - 3.67 + 45]^2 - 1/200 = 0.0741$$

Equation 5-3: Cyclic Stress Ratio (CSR)

$$CSR = 0.65(a_{max} / g)(\sigma_{vo} / (\sigma_{vo}')) r_d$$

Where

a_{max} = peak ground acceleration (PGA)

g = acceleration due to gravity

σ_{vo} = initial total vertical stress at the depth of interest (i.e., the sum of the layer weight(s) of the soil(s) above the SPT sample point times their thickness)

σ_{vo}' = initial effective vertical stress at the depth of interest (i.e., the sum of the layer weight(s) of the soil(s) above the SPT sample point times their thickness minus the layer weight of water times the thickness of *saturated* soil(s) above the SPT sample point)

r_d = Stress reduction coefficient for depth z (use $r_d = 1.0 - 0.00233172z$ for $z < 30$ ft., $r_d = 1.174 - 0.00813816z$ for 30 ft. $< z < 75.5$ ft.)

$$CSR = 0.65(0.080/1)(3180 / 1776)0.9382 = 0.0874$$

$a_{max} =$	0.080	Peak ground acceleration with 2% probability of exceedance in 50 years, V_s30 of 760 m/s using Conterminous U.S. 2014 data for the facility's location, 0.080 g (at time of publication https://earthquake.usgs.gov/hazards/interactive/)	
$g =$	1	Gravity = 1 g	
$\sigma_{vo} =$	3180	Depth of SPT = 26.5 ft Average layer weight of soil = 120 pcf	$26.5 * 120 = 3180$
$\sigma_{vo}' =$	1776	Depth of SPT = 26.5 ft Average layer weight of soil = 120 pcf Depth of water table = 4 ft Layer weight of water = 62.4 pcf	$26.5 * 120 - ((26.5 - 4) * 62.4) = 1776$
$r_d =$	0.9382	Depth of SPT = 26.5 ft $r_d = 1.0 - 0.00233172z$ for $z < 30$ ft.	$1.0 - 0.00233172 * 26.5 = 0.9382$

Equation 5-4: Factor of Safety Against Liquefaction (FS_L)

$$[FS]_L = (CRR_{7.5} / CSR) \cdot MSF \cdot K_\sigma$$

Where

MSF = Magnitude scaling factor ($MSF = (M_w/7.5)^{-1.43}$) After (Idriss 2010 and Moss 2006)

M_w = Earthquake magnitude

K_σ = Effective overburden pressure correction factor ($K_\sigma = (\sigma_{vo}'/P_a)^{(f-1)}$)

σ_{vo}' = Effective vertical overburden pressure

f = Function of site conditions (Idriss et al 2010 recommends value based on relative density (D_r) use $f = 0.8$ for $D_r \leq 0.40$, $f = -0.5D_r + 1$ for $0.40 < D_r < 0.80$, and $f = 0.6$ for $D_r \geq 0.80$)

" D_r " = *Relative Density* = $(e_{max} - e) / (e_{max} - e_{min}) \approx [(N_{SPT} (ER/0.78) (e_{max} - e_{min})^{1.7}) / 9 (2046.7 / (\sigma_{vo}'))^{(1/2)}]^{(1/2)}$ after (Curbrinovski 1999)

e_{max} = Maximum index void ratio (ASTM D4254)

e_{min} = Minimum index void ratio (ASTM D4253)

e = In-situ void ratio (difficult to determine without field sampling using piston sampler or soil freezing)

$$FS_L = (0.0741 / 0.0874) \cdot 1.344 \cdot 1.024 = 1.17$$

$MSF =$	1.344	Algermissen et al 1982 indicates that all of Ohio has an $M_w = 6.1$	$[(6.1/7.5)]^{-1.43} = 1.344$
$D_r =$	0.173	$N_{SPT} = 2$ $ER = 0.91$ $e_{max} = 0.663$ $e_{min} = 0.394$ $\sigma'_{vo} = 1776$ psf	$[(2(0.91/0.78) (0.663 - 0.394)^{1.7}) / 9 (2046.7/1776)^{(1/2)}]^{(1/2)} = 0.173$
$f =$	0.8	use $f = 0.8$ for $D_r \leq 0.40$	
$K_\sigma =$	1.024	$\sigma'_{vo} = 1776$ psf $P_a = 2000$ psf	$(1776/2000)^{(0.8 - 1)} = 1.024$

All of the Factors of Safety Against Liquefaction are above 1.0 so no further evaluation needs to be performed.

REFERENCES

- Algermissen S. T., Perkins D. M., Thenhaus P. C., Hanson S. L. and Bender B. L., 1982, "Probabilistic Estimates of Maximum Acceleration and Velocity in Rock in the Continuous United States," United States Geological Survey, Open-File Report 82-1033.
- Cubrinowski, M., Ishihara, K., 1999, Empirical correlation between SPT N-Value and Relative Density for Sandy Soils, *Soils and Foundations* Vol. 39, No. 5, 61-71, October, Japanese Geotechnical Society.
- Edil, T. B., 2002, *Soil Engineering for Non-Soils Engineers and Technicians*, Course Notebook, Section 2, Subsurface Explorations, University of Madison, Wisconsin.
- Martin, G.R. and Lew, M. Co-chairs and Editors Arulmoli, K., Baez, J.I., Blake, T.F., Earnest, J., Gharib, F., Goldhammer, J., D. Kupferman, Hsu, S., O'Tousa, J., Real, C.R., Reeder, W., Simantob, E., and Youd, T.L. Committee Members, 1999, *Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Southern California Earthquake Center University of Southern California.
- Hays, W.W., 1980, *Procedures for Estimating Earthquake Ground Motions*, Geological Survey Professional Paper 1114.
- Holtz, R. D. and Kovacs, W. D., 1981, *Introduction to Geotechnical Engineering*, Prentice Hall, Inc., Englewood Cliffs, New Jersey, pp. 309 and 390.
- Idriss, I. M. and Boulanger, R. W., 2010, *SPT-Based Liquefaction Triggering Procedures*, Center for Geotechnical Modeling Department of Civil and Environmental Engineering University of California Davis, California
- Marcuson, W. F., III, Hynes, M. E., and Franklin, A. G., 1990, "Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments," *Earthquake Spectra*, Vol. 6, No. 3, pp. 529 - 572.
- Moss, R. E. S., Seed, R. B., Kayen, R. E., Stewart, J. P., Der Kiureghian, A., and Cetin, K. O., 2006, CPT-based probabilistic and deterministic assessment of in situ seismic soil liquefaction potential, *J. Geotechnical and Geoenvironmental Eng.*, ASCE 132(8), 1032-051.
- Richardson, G. N., Kavazanjian, E. and Matasovi, N., 1995, EPA/600/R-95/051, RCRA Subtitle D (258) *Seismic Design Guidance for Municipal Solid Waste Landfill Facilities*.
- Seed, H. B., and Idriss, I. M., 1982, "Ground Motions and Soil Liquefaction During Earthquakes," Monograph No. 5, Earthquake Engineering Research Institute, Berkeley, California, pp. 134.
- Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M., 1985, "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," *Journal of Geotechnical Engineering*, ASCE, Vol. 111, No. 12, pp. 1425 - 1445.

Shibata, T., and Taparaska, W., 1988, "Evaluation of Liquefaction Potentials of Soils Using Cone Penetration Tests," *Soils and Foundations*, Vol. 28, No. 2, pp. 49 - 60.

State of New York Department of Transportation, 2007, *Geotechnical Design Procedure: Liquefaction Potential of Cohesionless Soils*, GDP-9, Revision #2, State of New York Department of Transportation, Geotechnical Engineering Bureau.

United States DOE, and United States NRC, 2012, *Technical Report: Central and Eastern United States Seismic Source Characterization for Nuclear Facilities*. EPRI, Palo Alto, CA.

United States Nuclear Regulatory Commission, 2003, *Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites*, Regulatory Guide 1.198.

Youd, T. L., Idriss, I. M. Andrus, R.D., Arango, I., G. Castro, G., Christian, J.T., Dobry, R., Liam Finn, W. D., Harder Jr., L.F., Hynes, M. E., Ishihara, K., J.P. Koester, J.P., Liao, S.S.C., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe II, K.H., 2001, "Liquefaction Resistance of Soils: Summary report from the 1996 and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE.

CHAPTER 6

SETTLEMENT ANALYSES

Table of Chapter Contents

Introduction	6-1
Reporting	6-1
Design Criteria	6-3
Total Settlement Analysis (Primary , Secondary , Differential)	6-4
Bearing Capacity	6-14
Appendix 6-A Examples (Total Settlement , Differential Settlement)	6-18
References	6-26

INTRODUCTION

This chapter provides the information, including specific examples, for evaluating the effects of settlement at *waste containment facilities* in the state of Ohio. *Total settlement*, *differential settlement*, *tensile strain*, and *bearing capacity* will be examined. The ramifications of these conditions can vary from the failure or damage to the liner and leachate collection systems to the reduction of grades or the reversal of drainage flow paths in leachate collection systems.

Each location of a *waste containment unit* is exposed to settlement based on two primary factors. First, settlement characteristics vary vertical and lateral across a site. Second, the vertical stresses created by the geometry of the *waste containment facility*. Based on these factors, a settlement analysis should include an evaluation of the following materials: in-situ soils, mine spoil, added geologic material, structural fill, recompacted soil liners, and waste materials. In addition, the analysis should focus on areas where significant changes in foundation materials occur (i.e., mining high walls, buried valleys, extremely compressible material) and locations of abrupt loading changes. The *waste containment unit* must be designed to account for the findings documented in the settlement analysis.

REPORTING

This section describes the information that should be submitted to demonstrate that a *waste containment unit* is not susceptible to damage from settlement. Ohio EPA recommends that the following information be included in its section of a report. At a minimum, the information should include an *overall settlement* and *differential settlement* analysis.

In the analysis, it is critical that each *compressible layer(s)* underlying the site be identified. The analysis should document the variability of these layer(s) and whether the layer(s) exist under all or a portion of the *waste containment unit*. In addition, the following should be submitted: a summary of the subsurface investigation, the variability of settlement characteristics in all layers, the changes in vertical stress due to the geometry of the landfill, and significant changes in the foundation materials (i.e., high walls, buried valleys, extremely compressible material).

Conclusions

The settlement analysis should include a tabular summary of the results, the conclusions based on these results, and the recommendations for addressing any slopes and/or engineering components that do not meet regulatory requirements.

Basis of Design

- A summary of the subsurface investigation and the application of the investigation in the analysis.
- A summary of the model used to evaluate the settlement.
- If any of the settlement parameters were interpolated by using random generation or another method, the details of this interpolation should be provided.
- Plan view maps showing the limits of the *waste containment unit(s)*, the critical cross-section(s), the location of the points where settlement is being calculated, and elevations of the following: the top of the liquid containment and collection system, the top of the liner system, the top of the *compressible layer(s)* and the top of *bedrock*. These plan views are typically submitted on 11 x 17 paper with a scale of one inch to 300 feet.
- Drawings showing the critical cross-section(s) analyzed. The sections should include the:
 - *Soil stratigraphy*, including *compressible layer(s)*.
 - The maximum temporal high *phreatic* elevation.
 - Depth of excavation.
 - Location of engineered components of the *waste containment unit* that may be adversely affected by settlement.

Any drawings or cross-sections referred to in this policy that are already present in another part of the geotechnical and stability analyses 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.

Calculations

- The amount of settlement is calculated at each point chosen along the cross-section(s).

- The detailed settlement calculations of the engineering components.
- The detailed *tensile strain* analysis for the geomembrane, including both overall and differential strain.
- Electronic copies of the input files or the spreadsheets used to perform the settlement calculations.
- All figures, drawings, or references utilized in the analysis. Any reference to a journal and/or textbook article should be included in its entirety.

Ohio EPA discourages the use of vertical sump risers in *waste containment units* due to the following: the potential damage to liner systems due to *bearing capacity*, the physical stability during filling operations, and the long-term effects due to *down drag*. If vertical sump risers, however, are included in the design of the *waste containment unit*, the following information is requested for a *bearing capacity* analysis:

- A summary of the subsurface investigation and the application of the investigation in the analysis.
- A summary of the approach, methodologies, and equations used in the analysis.
- A narrative and tabular summary of the results.

DESIGN CRITERIA

Waste containment facilities must be designed to meet component integrity requirements upon waste receipt and minimum slope requirements after *total settlement*. The *total settlement* is the sum of the immediate, *primary*, and *secondary settlement*. A minimum of 100% of *primary settlement* plus the *secondary settlement* which is approximately six (6) percent of the *primary settlement* or other *secondary settlement* calculations can be provided.

To determine settlement, vertical loading must be determined. For example, revisions to containment berms occur in either the horizontal or vertical expansions of *waste containment units*. The berm revisions will create increased loads leading to an additional settlement that should be accounted for in the revised design. Improper evaluation of loading changes can lead to the composite liner system and/or the leachate collection systems being compromised.

MSW is exposed to the following diverse activities: 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)

Design assumptions can include such items as the conservative nature of the design; the input data being site-specific (*a higher quality data*) and the calculation methods being valid and appropriate for

the *waste containment unit*. If these assumptions are met, the following design considerations are appropriate, but not limited to:

- Maintaining the minimum slopes of the composite liner system and the leachate collection pipes.
- Maintaining the minimum slopes of the separatory liner/leachate collection system.
- Maintaining the integrity of composite liner systems, leachate collection piping, and all other engineered components.
- Maintaining the structural integrity of any structures constructed on waste.

A more stringent design, however, with more robust than regulatory requirements may be warranted for the following:

- A failure would have a catastrophic effect on 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.

TOTAL SETTLEMENT ANALYSIS

A *total settlement* analysis accounts for all compressible materials beneath the *waste containment facility*. These materials include in-situ soil, added geologic material, structural fill, and recompacted soil liner. In addition to documenting the integrity of the composite liner system after *total settlement*, the analysis is required to document whether the leachate collection pipes and the liner system will have the required slopes to comply with the design and regulatory requirements after settlement.

The following two items necessitate the need for a thorough subsurface investigation: the damage that settlement can cause to engineered components, leachate collection pipes and drainage pathways, and the vertical and lateral variability of settlement characteristics across a site. The investigation should determine the proper number of critical cross-sections to be evaluated to ensure that the expected amount of settlement has been adequately estimated. In addition to various points being identified along the length of the flow paths, it is important to evaluate those points where increased settlement may occur. All settlement points should be spaced at a distance that best characterizes the *waste containment facility*. Characteristics include the *facility's* size, geometry, and the variability of the soil materials at the site. The settlement of each point should be evaluated considering the consolidation of each *compressible layer* below the points and the vertical stress being applied above these points.

Calculating Settlement

The first steps in a settlement calculation are to determine both the initial effective vertical stress (σ'_{vo} = total vertical stress - pore water pressure) and the change in the vertical stress ($\Delta\sigma_v$) caused by the loading of the *waste containment unit* on a point of interest. The sum of these values is the effective vertical stress ($\sigma'_{vo} + \Delta\sigma_v$) exerted upon the point. Once σ'_{vo} and $\Delta\sigma_v$ have been calculated, a typical settlement analysis would be performed using the procedure below.

Based on the large magnitude of today's landfills, the simple method of one-dimensional direct transfer of load method is primarily utilized. This method states that the direct load above a consolidation point has a total effect on that point without any stress distribution. Conservative and reasonably accurate settlement results can be expected due to minimal load reduction because the stress distribution points are much deeper than the base of the *waste containment unit*.

Primary Settlement (S_c)

Equation 6-1, Estimation of *primary settlement in normal consolidated clay*

$$S_c = \left(\frac{C_c}{1 + e_o} \right) \cdot H \cdot \log \left(\frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \right)$$

Where:

C_c = compression index

e_o = initial void ratio

H = thickness of the layer being evaluated

σ'_{vo} = initial effective vertical stress at the middle of the layer (i.e., The sum of the unit weight(s) of the soil(s) above the point times their depths minus the unit weight of water times the thickness of *saturated* soil(s) above the point.)

$\Delta\sigma_v$ = Change in vertical stress due to loading (i.e., the sum of the unit weights of the materials above the top of excavation times their depths)

Equation 6-2, Estimation of *primary settlement in overconsolidated clay*

$$S_c = \left(\frac{C_r}{1 + e_o} \right) \cdot H \cdot \log \left(\frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vo}} \right)$$

Where:

C_r = recompression index

Dense cohesionless material does not settle significantly and thus, does not have to be evaluated using this equation.

Equation 6-3, If the increase in vertical stress at the middle of the consolidation layer is such that $(\sigma'_{vo} + [\Delta\sigma]_v)$ exceeds the adjusted pre-consolidation pressure $(\sigma'_{p(OCR)})$ of the consolidating layer

$$s_c = \left(\frac{C_r}{1 + e_o} \right) \cdot H \cdot \log \left(\frac{\sigma'_{p(OCR)}}{\sigma'_{vo}} \right) + \left(\frac{C_c}{1 + e_o} \right) \cdot H \cdot \log \left(\frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{p(OCR)}} \right)$$

Where:

$\sigma'_{p(OCR)}$ = the adjusted pre-consolidation pressure at the middle of the layer (i.e., the effective vertical stress at the middle of the layer before excavation times the OCR)

OCR = Over-consolidation ratio (i.e., the pre-consolidation pressure (σ'_p) divided by the existing overburden pressure that the *sample* was experiencing at the time of extraction.)

Ohio EPA stresses the use of laboratory data to determine the various inputs for the *primary settlement* equations. ASTM D 2435 describes methods to determine σ'_p and e_o from laboratory data. Although not directly indicated in the standard, C_c and C_r can also be obtained from the same diagram that σ'_p is obtained. C_c is the slope of the virgin compression curve and C_r is obtained from the slope of the recompression curve (see [Figure 6-1](#)). To improve the likelihood of using an appropriate σ'_p and OCR, the following recommendation is suggested: Use a reduced load increment ratio (e.g. 1/2) at the effective stress increments bracketing the pre-consolidation pressure and the end-of-primary compression curve (Ladd et al 2003).

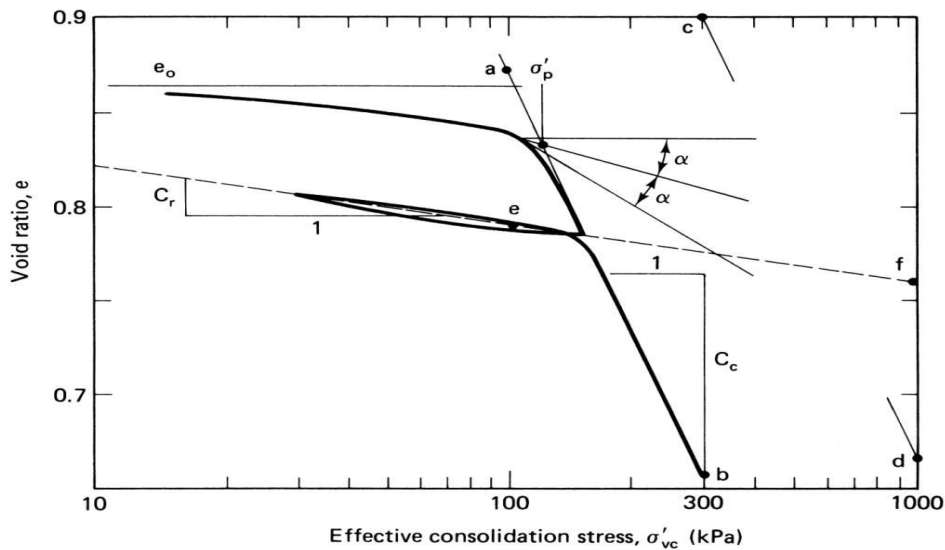


Figure 6-1, Over-consolidated stress diagram. From Ex. Figure 8.9, Holts and Kovacs, pp. 316**Secondary Settlement (S_s)****Equation 6-4, Secondary settlement**

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

Where:

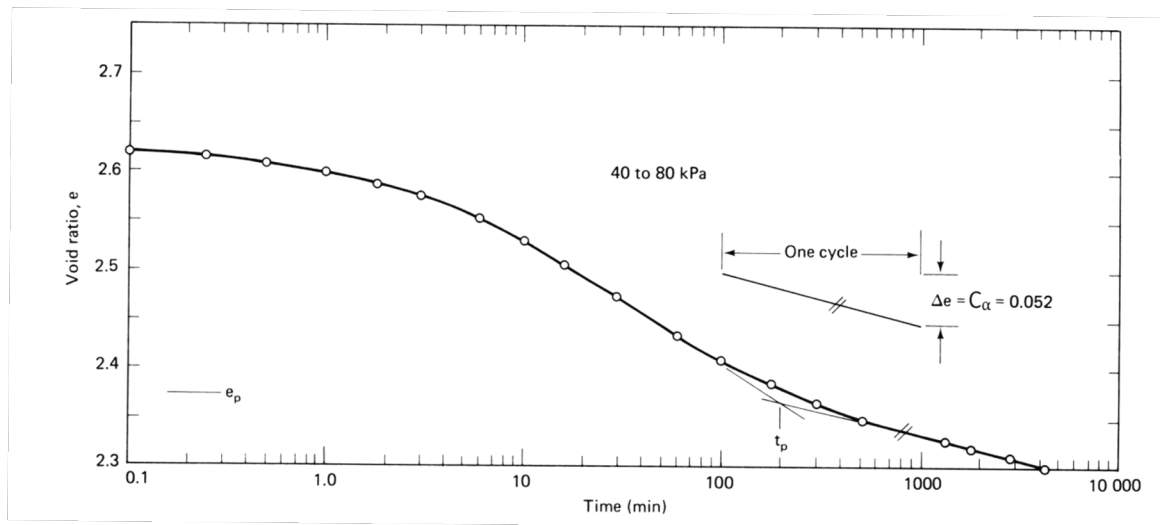
C_α = secondary compression index

e_p = the void ratio at the maximum time of complete *primary consolidation* (t_{pf}) in the test specimen of the *compressible layer*.

t_s = time over which settlement is to be calculated (use 100 years plus t_{pf} , unless some other time frame is acceptable to Ohio EPA for a specific *waste containment unit*).

t_{pf} = the point in time when the principal mechanism of settlement switches from *primary consolidation* to *secondary consolidation*.

The values for e_p and C_α can be determined graphically from a void ratio - log time curve as shown in [Figure 6-1](#). The value of C_α is the slope of the plot exceeding 100 percent *primary consolidation* or t_p in [Figure 6-2](#).

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

As shown in [Figure 6-2](#), the value for t_p can be determined which represents the time for completion of *primary consolidation* in the *specimen*. This value or its field value, t_{pf} , is used in [Equation 6-4](#) to determine the *secondary settlement*. [Equation 6-5](#) is a method for calculating t_{pf} .

Equation 6-5, Method for calculating t_{pf}

$$t_{pf} = \frac{T_v \cdot H_{dr}^2}{C_v}$$

Where:

T_v = a dimensionless time factor associated with the time at which the principal mechanism of settlement switches from *primary consolidation* to *secondary consolidation*. (Refer to T_v information below).

H_{dr} = maximum length of drainage so that H_{dr} is the full thickness of the consolidating layers if it is drained on one side (top or bottom), and H_{dr} is one-half of the thickness of the consolidating layers if it is drained on both sides (top and bottom).

C_v = coefficient of consolidation (converted to ft²/year, m²/year or as appropriate)*

* if more than one *compressible layer* needs to be evaluated a weighted average may need to be used for C_v .

The dimensionless time factor (T_v) has a theoretical relationship with the percent of *primary consolidation* (U_z) that can be expressed using one of the following equations:

Equation 6-6, Primary Consolidation

$$U_z = 1 - \sum_{n=0}^{\infty} f_1\left(\frac{z}{H_{dr}}\right) f_2(T_v)$$

or

Equation 6-7, Primary Consolidation

$$\begin{aligned}
 U_z = 1 - & \left(\left(1.274 \cdot \sin \left(1.57 \cdot \frac{z}{H_{dr}} \right) \cdot \left(\text{EXP}(-1.57^2 \cdot T_v) \right) \right) \right. \\
 & + \left(0.425 \cdot \sin \left(4.71 \cdot \frac{z}{H_{dr}} \right) \cdot \left(\text{EXP}(-4.71^2 \cdot T_v) \right) \right) \\
 & + \left(0.255 \cdot \sin \left(7.85 \cdot \frac{z}{H_{dr}} \right) \cdot \left(\text{EXP}(-7.85^2 \cdot T_v) \right) \right) \\
 & \left. + \left(0.182 \cdot \sin \left(10.99 \cdot \frac{z}{H_{dr}} \right) \cdot \left(\text{EXP}(-10.99^2 \cdot T_v) \right) \right) \right)
 \end{aligned}$$

Where:

z = depth within consolidating layer, or distance to drainage if only drained at bottom.

H_{dr} = maximum vertical length of drainage in consolidating layer.

A graph of these equations is provided in [Figure 6-3](#). Due to an asymptotic line being created by these equations, Ohio EPA recommends a derivation of $T_v \approx 2$ using the following: $U_z = 99\%$ and a $z/H_{dr} = 1$.

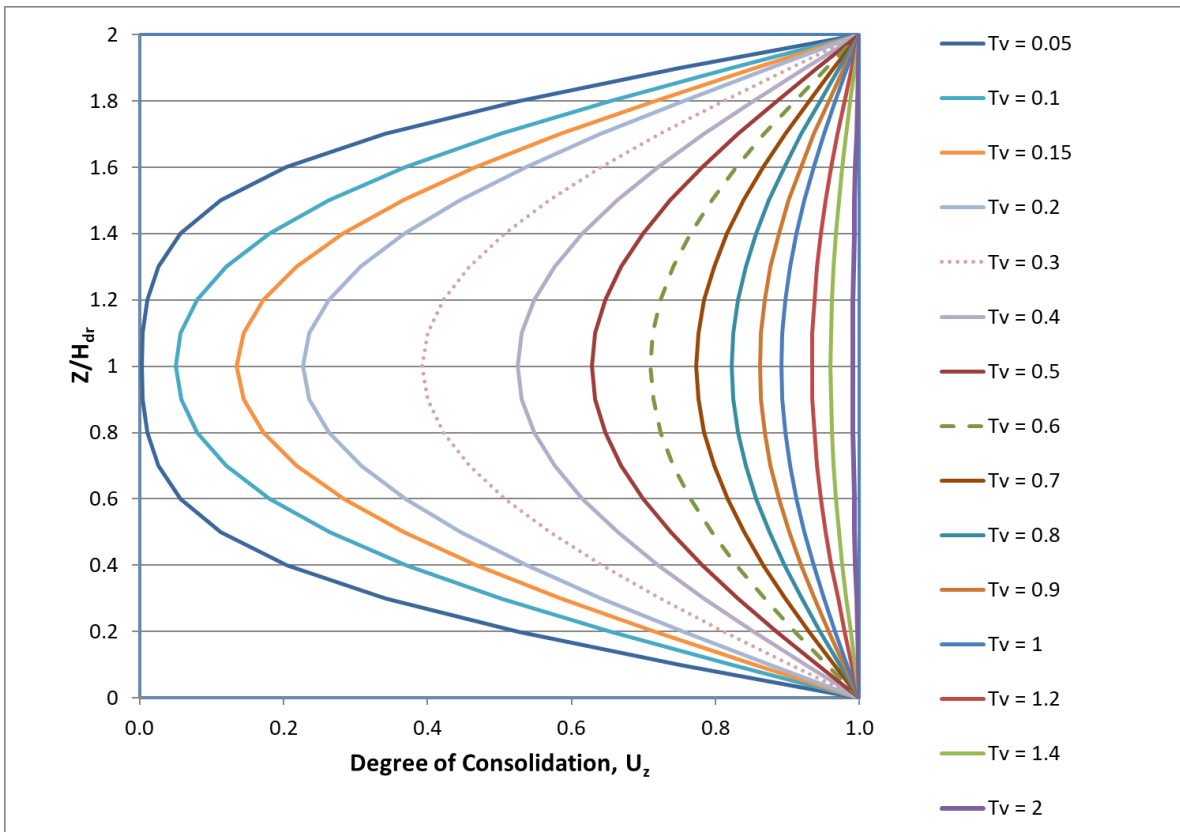


Figure 6-3, Degree of Consolidation for any time factor location (after Taylor, 1948)

Although Ohio EPA recommends a laboratory determination of the above inputs, many of these parameters can be derived from various charts found in engineering textbooks and manuals 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 for the material represented.

Differential Settlement Analysis

Although *differential settlement* can be caused by a variety of issues, this condition typically involves a combination of variable materials underlying the *waste containment unit* and/or abrupt changes in loading being applied to the *waste containment unit*. Examples of these conditions could be mining high walls, buried valleys, or vertical risers. The variability of material generally involves a transition between highly and less compressible materials. Although *differential settlement* can be caused by a variety of issues, this condition typically involves a combination of variable materials underlying the *waste containment unit* and/or abrupt changes in loading being applied to the *waste containment unit*. Examples of these conditions could be mining high walls, buried valleys, or vertical risers. The variability of material generally involves a transition between highly and less compressible materials. In addition to these transitional areas being thoroughly investigated and *sampled* during the sub-surface investigation ([Chapter 3, "Subsurface Investigation"](#)), cross-sections of these areas should be determined for evaluation in a settlement analysis.

A discussion of mining high walls and buried valleys follow. Concerning vertical risers, a *bearing capacity* analysis is an appropriate calculation for this type of structure. This analysis is discussed at the end of this chapter.

Mining Highwalls and Buried Valleys

Although this section specifically addresses high walls and buried valleys, the information described applies to other types of foundation materials susceptible to *differential settlement*. [Figure 6-4](#) illustrates a high wall/mine spoil interface. A high wall is created during mining operations when there is a transition from the existing, excavated strata to the mine spoil used to fill the quarry area that has been mined. This transition can lead to a severe *differential settlement* which can conservatively be estimated based on the following two assumptions: the high wall is rigid and therefore, will not settle and the maximum mine spoil settlement occurring 30 feet from the edge of the high wall. Concerning mining high walls and buried valleys, Ohio EPA recommends that a settlement analysis utilize a two-dimensional stress distribution theory to verify that the engineered components of the bottom liner system and the liquid drainage pathways will not be compromised.

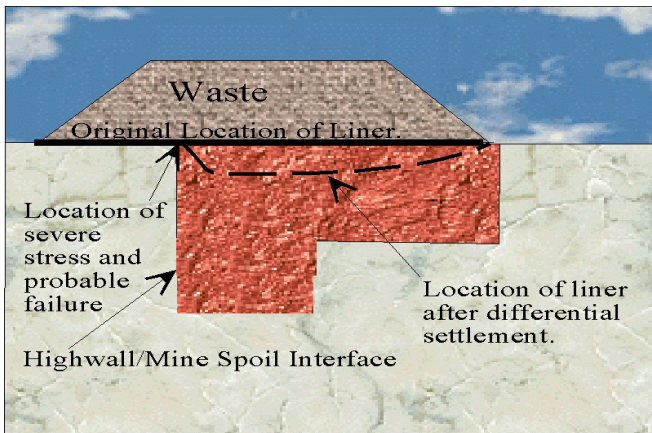


Figure 6-4, Existing Highwall / Mine Spoil Interface.

Due to the difficulty in locating buried high walls and valleys with traditional *borings* ([Chapter 3, “Subsurface Investigation”](#)), Ohio EPA recommends using geophysical surveys such as seismic refraction surveys or electrical imaging surveys. [Figure 6-5](#) shows an electrical imaging survey of a high wall and mine spoil.

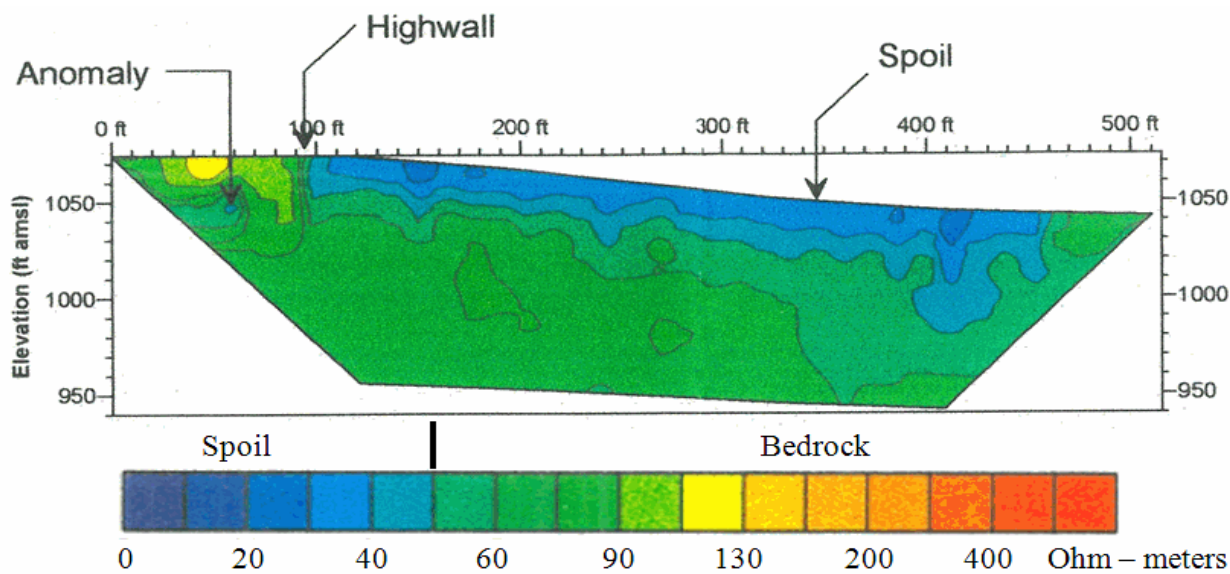


Figure 6-5, Electrical imaging survey of a high wall and mine spoil.

Three alternatives can be considered to reduce the *tensile strain* created by the *differential settlement* that engineered components may encounter at the high wall/mine spoil interface. *Tensile strain* will be further discussed in the next section. First, any abrupt transition slope between the high wall and the mine spoil could be reduced. This reduction will lessen the settlement and the stress due to the vertical load being distributed over a longer and flatter slope length versus the vertical edge of a high wall.

Second, another method consists of surcharging the mine spoil with a significant percentage of the proposed weight of the effective vertical stress. This alternative can be undertaken in conjunction with reducing the slope of the high wall. After removal of the surcharge, a large portion of the settlement occurs before the construction of any engineered components of the landfill in this area. Less settlement, therefore, will occur during landfill construction which should reduce the *tensile strains* in the engineered components.

Third, various geosynthetic tensile reinforcement products, such as geogrids or geotextiles, could be used to bridge the high wall/mine spoil transition in some applications. These geosynthetic products, however, would require sufficient anchorage to create sufficient tensile forces necessary for settlement reduction.

A design proposal with details of the alternative(s) and/or calculations should be submitted to Ohio EPA for approval via a permit application or other authorizing mechanism. The submittal must demonstrate the long-term effectiveness of the proposal and include a proposed plan for monitoring its effectiveness or provide a justification for the absence of long-term monitoring. In addition, the proposal should address the physical orientation of any engineered components that may be sensitive to *tensile strain* (e.g., geomembrane liner seams). In addition to the strain being directed away from the seams where practical, the seams should be installed perpendicular versus parallel in the area of the mining activities.

Tensile Strain (E_T)

After an evaluation of the critical points of the cross-sections for *overall* and *differential settlement*, the following engineered components should be analyzed for *tensile strain*: geotextiles, geomembranes, geosynthetic clay liners, geocomposite drainage layers, leachate collection piping, and wastewater piping. The settlement analysis should verify that these components can maintain their integrity when subjected to strain due to the settlement.

Ohio EPA's experience indicates that *tensile strains* in a typical *waste containment unit* rarely exceed 0.1% at the base, therefore, a strain analysis is not necessary at this location. Greater degrees of *tensile strain*, however, can be experienced in geosynthetics in both caps and separatory liners due to *differential settlement*. The settlement analysis should, therefore, include a comparison of the calculated strain versus the manufacturer's allowable strain specifications for the geosynthetic products utilized in the cap and separatory liner systems. The analysis should, also, document any negative strain(s) between two points which potentially creates compression in the liner system and non-positive drainage issues. Negative strains occur when a point at a higher elevation settles to a final lower elevation than an adjacent point with an original lower, but final higher elevation.

Calculating Strain (E_T)

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 equations:

Equation 6-8, Strain

$$E_T = \frac{L_f - L_o}{L_o}$$

Equation 6-9

$$L = \sqrt{(Elev. A - Elev. B)^2 + x^2}$$

Where:

L_f =the final distance separating the same two points after settlement is complete.

L_o =original distance separating two location points.

x =horizontal distance between points

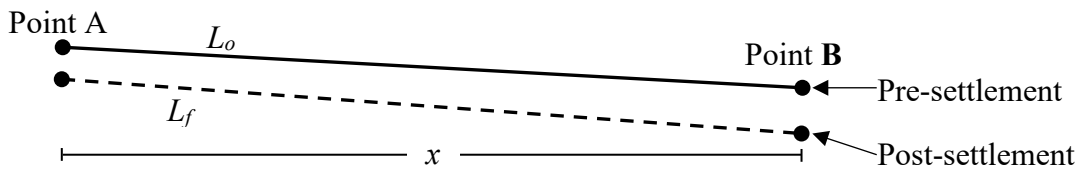


Figure 6-6, Strain development after settlement.

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 *waste containment facility*, and are similarly affected by the geometry of the *facility*. Designing a *waste containment facility* to account for induced settlements usually addresses all concerns except when the entire *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 *waste containment 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.

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 6-7](#)).

Designers should evaluate all potential bearing failure types for applicability to their *waste containment unit* 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

Stabilized waste is defined as any waste, such as sludge or semi-solids or liquids, 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 of 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, the *responsible party* must 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 the 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.

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 *waste containment unit* is designed to be safe against *bearing capacity* failures.

Bearing Capacity Analysis: $FS > 3.0$

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 unit* 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 units* 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 *waste containment unit* over soft clays is less than 3.0, then the *unit* will need to be redesigned or the *soil units* susceptible to a *bearing capacity* failure removed.

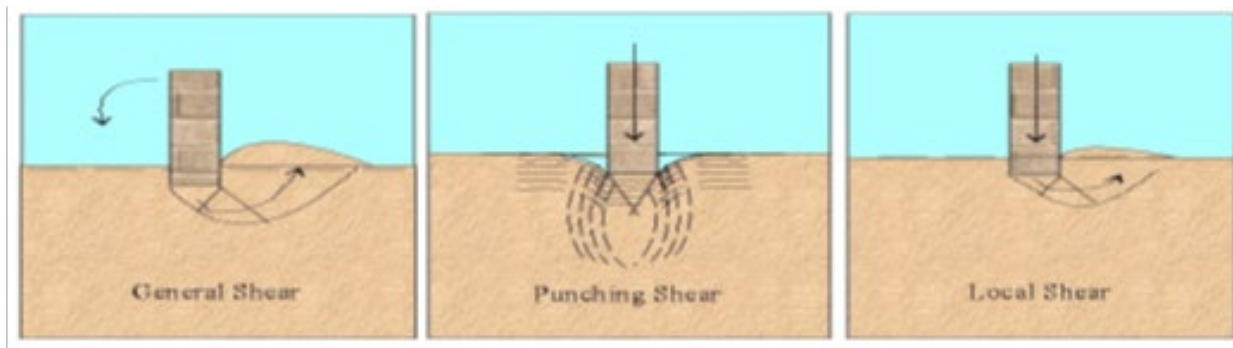


Figure 6-7, “The three modes of bearing failures”

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.

Equation 6-10, Factor of safety against bearing capacity failure (FS_b)

$$FS_b = \frac{q_{ult}}{p_{total}} \geq 3.0$$

Where:

q_{ult} = ultimate *bearing capacity* of the foundation soils.

p_{total} = the total pressure applied to the base of a foundation by an overlying mass.

Due to the difficulties present during filling operations and the potential for damaging liner systems, Ohio EPA recommends that a *bearing capacity* analysis be provided for *waste containment units* that use vertical sump risers to include the following items:

- A detailed summary of the subsurface investigation and the application of the investigation in the analysis.
- A detailed summary of the model used to evaluate the settlement.
- A tabular summary of the analysis results and conclusions based on these results.

Settlement of Municipal Solid Waste

Significant settlement can be expected when constructing over municipal solid waste. The data in the following table can be used in [Equation 6-1](#) and [Equation 6-4](#) to analyze for minimize and maximum settlement rates. For [Equation 6-4](#), the time to complete *primary consolidation* (t_{pf}) for municipal solid waste is very fast, if not immediate, and therefore Ohio EPA recommends using one day for t_{pf} .

Table 6-1, “List of Research Literature Data for Compressibility Values based on Y.M. Chen et al/ Waste Management 29 (2009) 86-95”

No.	Reference	$C_c / (1+e_o)$	$C_\alpha / (1+e_p)$	Remarks Composition	e_o or γ
1	Sowers (1973)	0.1–0.41	0.02–0.07	P: 20–40%; C: 0–15% R1: 16–40%; R2: 11–31%	$e_o = 2-15$
2	Zoino (1974)a	0.15–0.33	0.013–0.03		–
3	Converse (1975)a	0.25–0.3	0.07		–
4	Rao et al. (1977)	0.16–0.24	0.012–0.046		–
5	Landva et al. (1984)	0.2–0.5	0.0005–0.029		–
6	Oweis and Khera (1986)	0.08–0.22	–		–
7	Bjarngard and Edgers (1990)	–	0.004–0.04		–
8	Fassett et al. (1994)	–	<0.1		–
9	Wall and Zeiss (1995)	0.21–0.25	0.033–0.056	P: 21%; C: 17% R1: 54.5%; R2: 7.5%	–
10	Gabr and Valero (1995)	0.2–0.23	0.015–0.023	P: 0%; C: 33% R1: 47%; R2: 20% $e_o = 1-3$	$e_o = 1-3$
11	Boutwell and Fiore (1995)	0.09–0.19	0.006–0.012		–
12	Stulgis et al. (1995)	0.16	0.02	From field study of a test fill	–
13	GeoSyntec (1996)	0.35–0.55	0.10–0.18	From back analysis for OII landfill in the USA	–
14	Green and Jamenjad (1997)	–	0.01–0.08		–
15	Oweis and Khera (1998)	–	0.001–0.04		–
16	Landva et al. (2000)	0.17–0.24	0.01–0.016	P: 0–6%; C: 24–66% R1: 25–62%; R2: 0.8–6%	$\gamma = 7.6-10.4$
17	Qian et al. (2001)	0.17–0.36	0.01–0.10		–
18A	Hossain et al. (2003)	0.16–0.25	0.03–0.10	Typical of conventional Landfill	$e_o = 2-2.8$
18B	Hossain et al. (2003)	0.16–0.37	0.05–0.22	Typical of bioreactor landfill	$e_o = 2-2.8$
19	Liu Rong et al. (2003)	0.13–0.46	–	P: 25%; C: 8–47% R1: 23–53%; R2: 0.8–12%	$e_o = 3.5-5$
20	Chen and Ke (2003)	0.354	–	P: 17–27%; C: 44–57% R1: 20–23%; R2: 2–6.3%	$e_o = 3.8$
21	Chen et al. (2009)	0.1–0.3	–	P: 17–32%; C: 29–69% R1: 12–35%; R2: 2–4%	$e_o = 1-4$
22	Ohio EPA study	–	0.0135	From back analysis, Beech Hollow LDF Ohio	–
	Maximum conventional Landfill	0.55	0.18	50' of MSW under 100' of MSW $t_s=130yr$ $S_T= 48.2$ ft	–
	Minimum conventional Landfill	0.08	0.001	50' of MSW under 100' of MSW $t_s=130yr$ $S_T= 1.1$ ft	–
	Stulgis conventional Landfill	0.16	0.02	50' of MSW under 100' of MSW $t_s=130yr$ $S_T= 6.5$ ft	–
Notes: P – putrescent organics; C – cinder, and dust; R1 – plastic, paper, wood, textile, rubber, etc.; R2 – metal and glass;					
e_o – initial void ratio; γ – unit weight (kN/m ³). No. 1 - 21 are From Chen, Zhan, Wei, and Ke (2008)					

APPENDIX 6-A Examples

Total Settlement

A waste containment facility is proposed to be located over the following units: a normally consolidated brown clay and an overconsolidated gray clay intermixed with sand stringers on top of fractured *bedrock*. The proposed *waste containment facility* will have a maximum waste depth of 185 feet and a maximum excavation of 41 feet. The recompacted soil liner (RSL) is made of brown clay with a unit weight of 130.7 pcf. Three undisturbed *samples* from separate *borings*, as shown in [Figure 6-8](#), were collected for each clay unit. The *samples* were tested using ASTM D2435 / D2435M -11 (2020), “One-Dimensional Consolidation Properties of Soils using Incremental Loading”. The results are presented in [Table 6-2](#), “Consolidation Data”. The settlement point data for the five points in [Figure 6-8](#) (P1, P2, P3, F1, F2) is included in [Table 6-3](#), “Settlement Point Data”. As stated in the section on [Total Settlement Analysis](#), the points should be selected along the length of a flow path and where increased settlement may occur. In addition, the following information is provided: [Table 6-4](#), “Primary Settlement”; [Table 6-5](#), “Secondary Settlement” and [Table 6-6](#), “Results”.

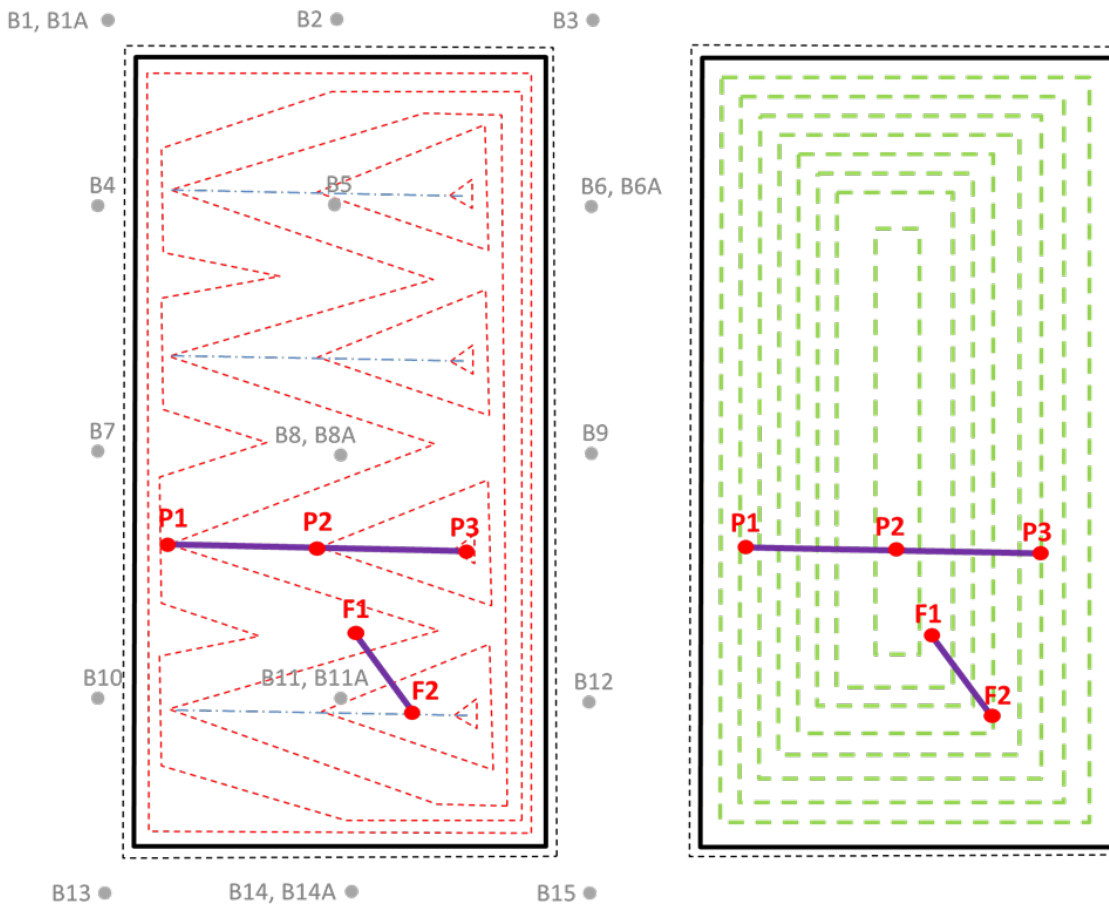


Figure 6-8, “Settlement Points: Top of Liner and Top of Final Cover”

Table 6-2, "Consolidation Data"

Soil type	Depth (ft)	Wet Density (pcf)	σ'_{vo} (tsf)	Specific gravity	σ'_p (tsf)	OCR	C_c	C_r	e_o	C_v ft ² /yr	C_α @ 8 tsf	e_p @ 8 tsf
Brown Clay	18	119.2	1.07	2.82	1.04	0.97	0.598	0.061	1.001	117.0	0.0214	0.8320
	27	118.9	1.60	2.83	1.56	0.97	0.607	0.069	0.964	112.2	0.0189	0.9506
	42	118.0	2.48	2.81	2.62	1.06	0.525	0.070	1.009	99.3	0.0191	0.9171
Averages	-----	118.7	-----	2.82	-----	1.00	0.5767	0.067	0.9914	109.5	0.0198	0.8999
Gray Clay	48	121.9	2.93	2.77	3.68	1.26	0.422	0.027	0.918	49.7	0.0099	0.7185
	54	120.0	3.24	2.81	3.84	1.18	0.324	0.027	0.845	54.6	0.0077	0.8980
	60	119.3	4.27	2.79	4.95	1.16	0.325	0.030	0.971	48.9	0.0076	0.8522
Averages	-----	120.4	-----	2.79	-----	1.20	0.357	0.028	0.911	51.1	0.0084	0.8229

Table 6-3, "Data for Settlement Points"

	Unit weight (pcf)	P1 (ft)	P2 (ft)	P3 (ft)	F1 (ft)	F2 (ft)
Top of Final Cover	118.7	695.0	800.0	727.0	793.0	766.0
Top of Waste	75.0	691.0	796.0	723.0	789.0	762.0
Existing Grade	118.7	652.0	651.0	650.0	650.0	650.0
Top of RSL	118.7	613.0	611.0	609.0	616.0	610.0
Top of Brown Clay	118.7	610.0	608.0	606.0	610.0	607.0
Top of Gray Clay	120.4	604.0	603.0	602.0	602.0	602.0
Top of Rock	-----	586.0	582.0	584.0	582.0	583.0
Top of Aquifer	-----	586.0	582.0	584.0	582.0	583.0
The thickness of Layers (ft)						
Final Cover		4.0	4.0	4.0	4.0	4.0
Waste		78.0	185.0	114.0	173.0	152.0
RSL		3.0	3.0	3.0	3.0	3.0
Brown Clay		6.0	5.0	4.0	11.0	5.0
Gray Clay		18.0	21.0	18.0	20.0	19.0

Table 6-4, “Primary Settlement”

	P1	P2	P3	F1	F2
$\Delta\sigma'_v$ (psf)	6,325	14,350	9,025	13,450	11,875
σ'_{vo} RSL (psf)	178.1	178.1	178.1	178.1	178.1
S_c RSL (ft)	1.357	1.661	1.488	1.637	1.590
$\Delta\sigma'_v$ (psf)	6,681	14,706	9,381	13,806	12,231
σ'_p (OCR) Brown (psf)	5341.5	5400.9	5460.2	5044.8	5400.9
σ'_{vo} Brown (psf)	356	296.8	237.4	652.9	296.8
S_c Brown (ft)	0.445	0.854	0.468	1.784	0.740
σ'_p (OCR) Gray (psf)	8137.4	8354.2	8137.4	8281.9	8209.7
σ'_{vo} Gray (psf)	2151.9	2213.8	1914.5	2865.8	2093.4
S_c Gray (ft)	0.272	1.380	0.645	1.270	1.023
Total S_c (ft)	2.074	3.895	2.601	4.691	3.353

Table 6-5, “Secondary Settlement”

	P1	P2	P3	F1	F2
$H_{dr\ max}$ (ft)	27	29	25	34	27
C_v (ft ² / yr - weighted average)	70.567	67.210	67.452	75.147	68.404
t_{pf} (yr - weighted average)	20.661	25.026	18.532	30.766	21.315
t_s (yr – maximum)	120.661	125.026	118.532	130.766	121.315
t_{pf} (yr – Brown Clay / RSL)	20.661	25.026	18.532	30.766	21.315
e_p (Brown Clay / RSL)	0.849	0.844	0.844	0.855	0.846
S_s (ft - Brown Clay / RSL)	0.074	0.060	0.061	0.094	0.065
t_{pf} (yr – Gray Clay)	12.681	17.260	12.681	15.656	14.129
e_p (Gray Clay)	0.823	0.823	0.823	0.823	0.823
S_s (ft – Gray Clay)	0.081	0.083	0.081	0.085	0.082

Table 6-6, “Settlement and Strain Results”

	P1	P2	P3	F1	F2
Original elevation (ft)	613.0	611.0	609.0	616.0	610.0
Total Settlement (ft)	2.229	4.038	2.742	4.870	3.501
Final elevation (ft)	610.771	606.962	606.258	611.130	606.499
X location (ft)	0	400	800	0	300
Original Slope (%)	-----	-0.50	-0.50	-----	-2.00
Final Slope (%)	-----	-0.95	-0.18	-----	-1.54
Strain (E_T - %)	-----	0.0033	-0.0011	-----	-0.0081

Conclusion

The *final slopes* of the leachate pipe from section P2 - P3 does not meet the solid waste regulatory requirement of 0.5%. The leachate flow path F1 – F2 does not meet the solid waste regulatory requirement of 2%. The *waste containment unit* would need to be redesigned to meet the regulations through methods such as overbuilding, surcharging, removal of the material causing the problem, or engineered reinforcement.

Calculation

Primary Settlement (Sc) Calculation for P1

$$\Delta\sigma'_v = 118.7 \times (695 - 691) + 75 \times (691 - 613) = 6324.8psf$$

$$\sigma'_{vo} RSL = \frac{613 - 610}{2} \times 118.7 = 178.05psf$$

Since the RSL was placed and compacted, it is a conservative assumption that the layer is normally consolidated, and therefore, [Equation 6-1](#) is used to calculate S_c .

$$S_c = \left(\frac{0.5767}{1 + 0.9914} \right) \cdot 3 \cdot \log \left(\frac{178.05 + 6324.8}{178.05} \right) = 1.357ft$$

$$\Delta\sigma'_v = 118.7 \times (695 - 691) + 75 \times (691 - 613) + 118.7 \times (613 - 610) = 6680.9psf$$

$$\sigma'_{p(OCR)Brown} = 1.0 \times \left(\left(652 - 610 + \frac{610 - 604}{2} \right) \times 118.7 \right) = 5341.5psf$$

$$\sigma'_{vo} Brown = \frac{610 - 604}{2} \times 118.7 = 356.1psf$$

Since $\sigma'_{p(OCR)} < (\sigma'_{vo} + \Delta\sigma'_v)$, using [Equation 6-3](#) to calculate S_c for the brown clay.

$$S_c Brown = \left(\frac{0.067}{1 + 0.9914} \right) \cdot 6 \cdot \log \left(\frac{5341.5}{356.1} \right) + \left(\frac{0.5767}{1 + 0.9914} \right) \cdot 6 \cdot \log \left(\frac{356.1 + 6680.9}{5341.5} \right) = 0.445ft$$

$$\sigma'_{p(OCR)Gray} = 1.2 \times \left((652 - 604) \times 118.7 + \left(\frac{604 - 586}{2} \right) \times 120.4 \right) = 8137.44psf$$

$$\sigma'_{vo} Gray = (613 - 604) \times 118.7 + \frac{604 - 586}{2} \times 120.4 = 2151.9psf$$

Since $\sigma'_{p(OCR)} < (\sigma'_{vo} + \Delta\sigma'_v)$ use [Equation 6-3](#) to calculate S_c for the gray clay.

$$S_c Gray = \left(\frac{0.028}{1 + 0.911} \right) \cdot 18 \cdot \log \left(\frac{8137.44}{2151.9} \right) + \left(\frac{0.357}{1 + 0.911} \right) \cdot 18 \cdot \log \left(\frac{2151.9 + 6680.9}{8137.44} \right) = 0.272ft$$

Secondary Settlement (Sc) Calculation for P1

1. Calculation of Maximum Drain Length ($H_{dr \max}$)

A. $H_{dr \max} = \text{Top of RSL (613 ft)} - \text{Top of Rock (586 ft)} = 27\text{ft}$

2. Coefficient of Consolidation (C_v) – Weighted Average

A. The weighted average of the C_v for the RSL, Brown and Gray Clays

B. $C_v = \frac{(109.5 \times (3+6) + 51.1 \times 18)}{(3+6+18)} = 70.567 \text{ ft}^2/\text{yr}$

3. Calculation of t_{pf} and Time of Settlement (t_s)

A. $t_{pf} (\text{Weighted Average of RSL, Brown and Gray Clays}) = \frac{2 \times 27^2}{70.567} = 20.661 \text{ yr}$

B. $t_s = 100 \text{ yr} + 20.661 \text{ yr} = 120.661 \text{ yr}$

4. Void Ratio (e_p)

A. $e_p (\text{Weighted Average of RSL, Brown and Gray Clays}) = \frac{(0.8999 \times (3+6) + 0.8229 \times 18)}{(3+6+18)} = 0.849$

5. Calculate *Secondary Settlement* (S_s)

A. Use [Equation 6-4](#) to calculate S_s for the RSL, brown and gray clays.

B. $S_s (\text{Brown Clay} + \text{RSL}) = \left(\frac{0.0198}{1+0.849} \right) \cdot (3+6) \cdot \log \left(\frac{120.661}{20.661} \right) = 0.0738 \text{ ft}$

C. $S_s (\text{Gray}) = \left(\frac{0.0084}{1+0.8229} \right) \cdot 18 \cdot \log \left(\frac{120.661}{12.681} \right) = 0.081 \text{ ft}$

1. $t_{pf} (\text{Gray}) = \frac{2 \times 18^2}{51.1} = 12.681 \text{ yr}$

D. $\text{Total Settlement} = S_c (1.357 + 0.445 + 0.272) + S_s (0.0738 + 0.081) = 2.229$

6. Evaluation Between P1-P2

A. $\text{Original Slope (P1 - P2)} = \frac{(613-611)}{(0-400)} = -0.005 = -0.50\%$

1. To avoid missing grade reversals, the slope formulas should not use absolute values.

B. Final Slope = $\frac{(610.771-606.962)}{(0-400)} = -0.00952 = -0.95\%$

C. Strain (P1-P2)

1. $E_T = \frac{\sqrt{(610.771-606.962)^2+(0-400)^2}-\sqrt{(613-611)^2+(0-400)^2}}{\sqrt{(613-611)^2+(0-400)^2}} = 0.00003282$

Differential Settlement

In this example, the *waste containment facility* will be constructed mostly on *bedrock* except where a narrow valley exists at one side of the *facility*. For this example, the same unit weight will be used for the waste and the consolidation parameters of the soil structural fill will be the same as the brown clay from the previous example.

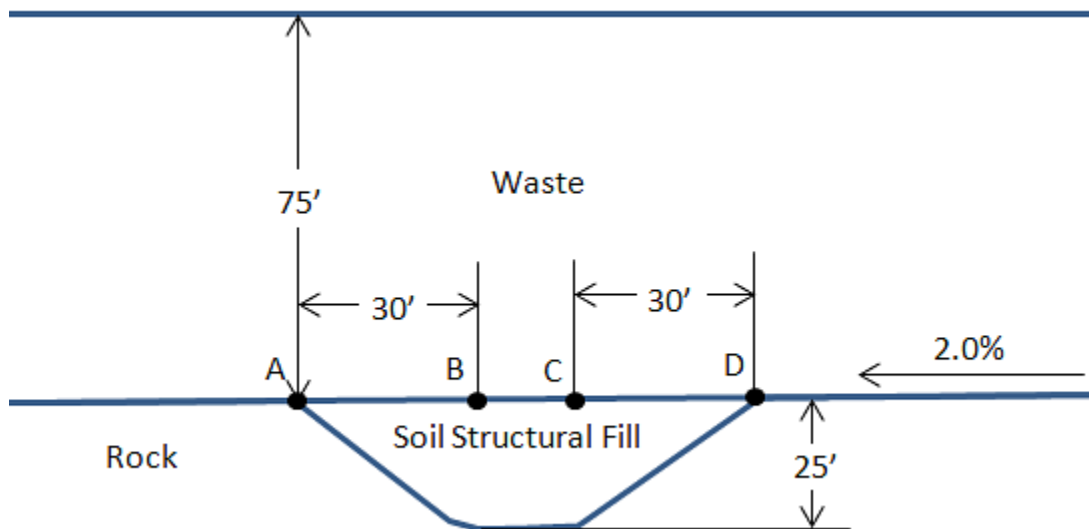


Figure 6-9, Differential Settlement Example Cross-Section

Table 6-7, “Data for Settlement Points”

	Unit weight (pcf)	A (ft)	B (ft)	C (ft)	D (ft)
Top of Final Cover	118.7	688.0	688.0	688.0	688.0
Top of Waste	75.0	684.0	684.0	684.0	684.0
Existing Grade	118.7	610.0	585.0	585.0	611.5
Top of RSL	118.7	613.0	613.6	613.9	614.5
Top of Brown Clay	118.7	610.0	610.6	610.9	611.5
Top of Rock	-----	610.0	585.6	585.6	611.5
Thickness of Layers (ft)					
Final Cover		4.0	4.0	4.0	4.0
Waste		71.0	70.4	70.1	69.5
Brown Clay / RSL		3.0	28.0	28.3	3.0

Table 6-8, “Primary Settlement” (Normally Consolidated Clays)

	A	B	C	D
$\Delta\sigma'_v$ (psf)	5,800	5,755	5,732	5,687
σ'_{vo} (psf - Brown Clay / RSL)	178.1	1,661.8	1,679.6	178.1
S_c (ft – Brown Clay / RSL)	1.326	5.267	5.284	1.319

Table 6-9, “Secondary Settlement”

	A	B	C	D
$H_{dr\ max}$ (ft)	3.0	28.0	28.3	3.00
C_v (ft ² / yr - weighted average)	109.5	109.5	109.5	109.5
t_{pr} (yr - weighted average)	0.399	34.725	35.473	0.399
t_s (yr – maximum)	100.399	134.725	135.473	100.399
t_{pf} (yr – Brown Clay / RSL)	0.399	34.725	35.473	0.399
e_p (Brown Clay / RSL)	0.900	0.900	0.900	0.900
S_s (ft - Brown Clay / RSL)	0.075	0.172	0.172	0.075

Table 6-10, “Settlement and Strain Results”

	A	B	C	D
Original elevation (ft)	613.0	613.6	613.9	614.5
Total Settlement (ft)	1.401	5.439	5.455	1.394
Final elevation (ft)	611.6	608.2	608.4	613.1
X location (ft)	0	30	45	75
Original Slope (%)	-----	2.00	2.00	2.00
Final Slope (%)	-----	-11.46	1.89	15.54
Strain (E_T) (%)	-----	0.6345	-0.0021	1.1798

Conclusion

Based on the results, the buried valley in this example had a profound effect on the slope of the bottom liner which resulted in a reversal of flow. Although the amount of settlement was not severe enough to compromise the integrity of the liner system due to strain, a revised design would be required to account for the increased settlement. Revisions may include such things as overbuilding, surcharging, removal of the material causing the problem, or engineered reinforcement.

REFERENCES

- American Society of Civil Engineers, 2000, Guidelines for Instrumentation and Measurements for Monitoring Dam Performance, Reston, Virginia.
- American Society for Testing and Materials, 1999, D 2435, "Standard Test Method for One-Dimensional Consolidation Properties of Soils." American Society for Testing and Materials (ASTM), Vol. 04.08, West Conshohocken, Pennsylvania, pp. 210 - 219.
- Boussinesq J., (1874), Annales des Ponts et Chaussées: Mémoires et Documents 5 Série, Vol. VII, 2nd Semestre, pp169-187.
- Bray, J.D., and Merry, S.M., 1999, "A Comparison of the Response of Geosynthetics in the Multi-Axial and Uniaxial Test Devices", Geosynthetics International, Vol. 6, No. 1, pp. 19-40.
- Chen, Y.M., Zhan, T. L.T., Wei, H.Y., Ke, H., 2008, "Aging and compressibility of municipal solid wastes", Elsevier Ltd.
- Das, B. M., 1990, Principles of Foundation Engineering. 2nd Edition. PWS-KENT Publishing Company.
- Fellenius, B. H., 1998, "Recent Advances in the Design of Piles for Axial Loads, Dragloads, Downdrag, and Settlement" ASCE and Port of NY&NJ Seminar1, Urkkada Technology Ltd. Ottawa, Ontario.
- Holtz, R. D. and Kovacs, W. D., 1981, Introduction to Geotechnical Engineering, Prentice Hall, Inc., Englewood Cliffs, New Jersey, pp. 309 and 390.
- Ladd, C. C. and D. J. DeGroot, 2003, "Recommended practice for soft ground site characterization: Arthur Casagrande Lecture", Proc. 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering., Massachusetts Institute of Technology, Cambridge, MA.
- Ling, H. I., Leshchinsky, D., Mohri, Y., and Kawabata, T., 1998, "Estimation of Municipal Solid Waste Landfill Settlement." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 1, pp. 21 - 28.
- McCarthy, D. F., 2002, Essentials of Soil Mechanics and Foundations: Basic Geotechnics. 6th Edition. Prentice-Hall.
- Peggs, I., Schmucker, B., and Carey P., 2005, "Assessment of Maximum Allowable Strains In Polyethylene and Polypropylene Geomembranes," GSP 142 Waste Containment and Remediation, ASCE
- Skempton, A.W., 1951, "The Bearing Capacity of Clays," Building Research Congress, London. pp. 180 - 189.

Sowers, G. F., 1968, "Foundation Problems in Sanitary Landfills." Journal of Sanitary Engineering, ASCE, Vol. 94, No.1, pp. 103 - 116.

Sowers, G. F., 1973, "Settlement of Waste Disposal Fills." Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, pp. 207 - 210.

Spikula, D. R., 1996, "Subsidence Performance of Landfills." Proceedings of the 10th Geosynthetics Research Institute Conference, Drexel University, Philadelphia, PA, pp. 210 - 218.

Taylor, D.W., 1948, "Fundamentals of Soil Mechanics," Wiley, New York.

United States Army Corps of Engineers, 1990, Engineering and Design. Bearing Capacity of Soils. EM 1110-1-1904. Department of the Army, Washington, DC.

Wall, D. K. and Zeiss, C., 1995, "Municipal Landfill Biodegradation and Settlement." Journal of Environmental Engineering, ASCE, Vol. 121, No. 3, pp. 214 - 224.

Chapter 7

HYDROSTATIC UPLIFT AND SEEPAGE ANALYSES

Table of Chapter Contents

Introduction	7-1
Reporting	7-1
Factor of Safety	7-3
Analysis (hydrostatic uplift , heaving , seepage potential/piping , erosion rate index)	7-5
Appendix 7-A Examples (hydrostatic uplift of a pump station , hydrostatic uplift at the sump , seepage analysis using FEM , estimation of soil erosion rate)	7-17
Appendix 7-B Determination of Total Head and the Concept of Seepage Forces	7-24
References	7-26

INTRODUCTION

This chapter provides information to use when evaluating the hydrostatic uplift and seepage potential at a *waste containment facility* in Ohio. Either of these conditions can undermine the foundations of ancillary structures when the groundwater level (*phreatic* or *piezometric*) rises above the bottom elevation of the structures. Groundwater pressure buildup may lift structures entirely or damage the integrity of the foundation soils beneath them.

Hydrostatic uplift is a floatation condition caused by the displaced volume of the rising water table. This condition can be counteracted by equalizing the hydrostatic uplift force with heavier structures or overburden material atop the area of uplift force, and also by anchoring. However, it should be noted that methods employed to counteract hydrostatic uplift forces are unlikely to have a measurable impact on the reduction of pore water pressure buildup within the soil and cannot be employed to reduce the seepage potential and associated damage.

Seepage is the flow of water through soils caused by difference in head, which is a measure of the energy lost in overcoming the resistance provided by the soils and other underground obstructions. To restrict sand particle movement, a properly designed filter accompanied by an increase in the vertical stress from the constructed liner or deposited waste may reduce the likelihood of the formation of soil boils.

Both failure mechanisms, uplift and seepage, must be considered as part of the design of a *waste containment unit*.

REPORTING

This section describes the information that should be submitted to demonstrate that a *waste containment unit* is not susceptible to hydrostatic uplift and seepage damage. Ohio EPA recommends

that the following information be included in its own separate section of the geotechnical and stability analysis report submitted for Ohio EPA review.

Conclusions

- One or more tables summarizing the results of the hydrostatic uplift analysis and seepage analysis on all the analyzed cross-sections.

Basis of Design

- A summary of the worst-case scenarios used to analyze the hydrostatic uplift and seepage potential at the *waste containment facility*.
- The scope, extent, and findings of the subsurface investigation as they pertain to the analyses of potential hydrostatic uplift and seepage failures at the *waste containment facility*.
- A narrative description of the logic and rationale used for selecting the critical cross-sections for the *waste containment unit*.
- A narrative justifying the assumptions made in the calculations and describing the methods and rationale used to identify the areas of the *waste containment facility* that are most likely to be involved in a hydrostatic uplift or seepage failures. This narrative should include the site conditions that were considered, including, temporal high *phreatic* and *piezometric surfaces*.
- Plan views of the *internal slope* grading plans, clearly showing the location of each analyzed cross-section and the limits of the *waste containment unit(s)*. Northings and eastings of the cross-section should be indicated on the plan view. These plan views are typically submitted on 11 x 17 paper with a scale of one inch to 300 feet showing the top of recompacted soil liner / the top of added geologic material / bottom of excavation, with an isopach of the temporal high *phreatic* / *piezometric surfaces*.

Any drawings or cross-sections referred to in this guidance document 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.

Calculations

- Hydrostatic Uplift Analysis
 - Perform if the temporal high *phreatic* or *piezometric surface* is higher than the top of the recompacted soil liner in any area of the *waste containment unit*.

- Seepage Analysis
 - Perform heave analysis if the temporal high *phreatic* or *piezometric surface* is higher than the top of the recompacted soil liner in any area of the *waste containment unit* and is not controlled by a gravity underdrain.
 - Perform a piping/internal erosion analysis on soil units at the *waste containment unit* where the temporal high *phreatic* or *piezometric surface* is higher than the top of the recompacted soil liner, added geologic material, and/or the base of the excavation, and is not controlled by a gravity underdrain.
 - Perform an erosion rate index analysis on plastic soils within soil units at the *waste containment unit* where the temporal high *phreatic* or *piezometric surface* is higher than the top of the recompacted soil liner, added geologic material, and/or the base of the excavation and is not controlled by a gravity underdrain.

Supporting Information

- Electronic copies of the input files or the spreadsheets used to perform the hydrostatic uplift and seepage calculations; and,
- Any figures, drawings, or references utilized during the analysis should be properly cited; these may be requested separately for review by Ohio EPA.

FACTOR OF SAFETY

The following factors of safety should be used, unless superseded by rule, when demonstrating that a *waste containment unit* will resist the hydrostatic uplift and seepage potential.

Hydrostatic Uplift: $FS \geq 1.4$

Seepage Heave: $FS \geq 1.1$

Seepage Piping/Internal Erosion: $FS \geq 1.1$

The use of a higher factor of safety against hydrostatic uplift or seepage potential may be warranted whenever:

1. A failure would have a catastrophic effect upon human health or the environment;
2. Uncertainty exists regarding the accuracy, consistency or validity of data, and there is not an opportunity to conduct additional testing to improve or verify the quality of the data;
3. Repairing a *waste containment unit* after a failure would create a hardship for the *responsible party* or the waste disposal customers;

4. Uncertainty exists about the effects that changes to the site conditions over time may have on the *phreatic* or *piezometric surfaces*, and engineered controls cannot be implemented to reduce the uncertainty; and/or,
5. The soil is classified to have a moderately rapid to extremely rapid erosion rate (or an Erosion Rate Index less than 6).

The *waste containment unit* must be designed to prevent failures due to hydrostatic uplift. Factors of safety against hydrostatic uplift and seepage potential lower than 1.4 and 1.1, respectively, are not considered sound engineering practice under most circumstances. There are inherent uncertainties in calculating factors of safety against hydrostatic uplift and seepage potential, and any failure of the *waste containment unit* due to hydrostatic uplift or seepage potential is likely to increase the potential for harm to human health and the environment. If a *waste containment unit* has a factor of safety against hydrostatic uplift less than 1.4, the groundwater table might be lowered until enough stabilizing material is placed above the liner system to allow for a factor of safety greater than 1.4 when the groundwater table is at its natural maximum level. However, if the factor of safety against seepage potential is less than 1.1, the *waste containment unit* may have to be redesigned or moved to another site that is not at risk of failure from seepage.

The factors of safety specified in this guidance are subject to, but are not limited to only, the following conditions:

1. The use of conservative, site-specific, high quality data;
2. Proper selection of worst-case geometry; and,
3. The use of calculation methods that are demonstrated to be valid and appropriate for the *waste containment unit*.

If the *responsible party* does not adhere to these conditions, then the factors of safety may not be appropriately protective of human health and the environment. For instance, applying the average depth of excavation and the average elevation of the *piezometric surface* to the factor of safety evaluation results in the conclusion that hydrostatic uplift is unlikely to occur, which is not necessarily accurate. Note that the temporal high *piezometric surface* intersects the liner system, which indicates the potential for hydrostatic uplift.

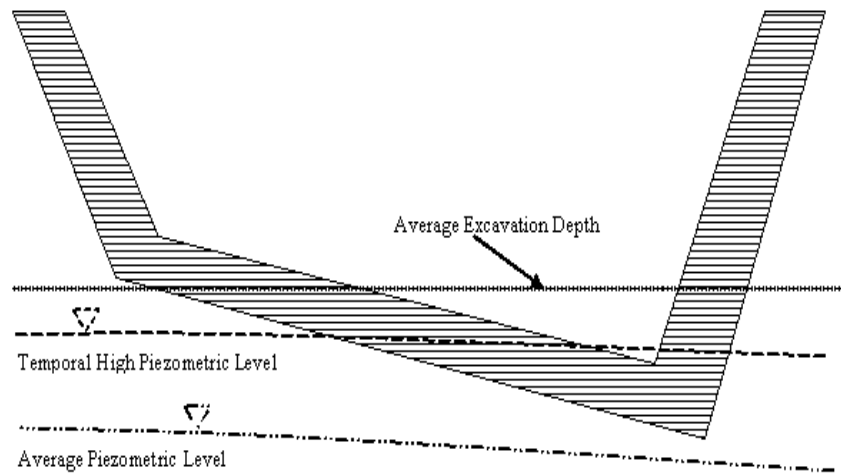


Figure 7-1 Illustration of relationship between pressure heads and bottom of excavation

ANALYSIS

Hydrostatic Uplift, Concept and Analysis

Maximum uplift force should be calculated based on the maximum measured *piezometric* head compared to the normal stress created by the overlying *soil units*. This method will provide a conservative evaluation of the resistance created by a *soil unit* against hydrostatic uplift.

Because percolation through *soil units* can affect hydrostatic uplift force as well, calculation of seepage and shear resistance of the soil may theoretically yield a more accurate overall failure analysis. However, results of the recommended method of analysis will be especially applicable when checking an interface between a subbase and a clay (or plastic) liner, where any significant seepage through the liner material is exceptionally undesirable and generally unanticipated.

The restricting force of the waste mass may be added in this formula only when an underdrain system beneath the liner drains to a sump and is then pumped to the surface.

Worst-case interactions of the recompacted soil liner grade with the *phreatic* and *piezometric surfaces* must be selected for analysis of hydrostatic uplift and seepage potential. Temporal changes in *phreatic* and *piezometric surfaces* are to be considered with the highest temporal *phreatic* and *piezometric surfaces* used in the analyses. Using average depth of excavation or average elevation for the *phreatic* and/or *piezometric surfaces* **is not acceptable** because the goal of the analyses is to identify all areas within the *waste containment unit* where liners will be constructed and may have a factor of safety less than 1.4 for hydrostatic uplift or 1.1 for seepage potential.

Figure 7-2 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, which might allow for hydrostatic uplift where the resisting forces are overtaken by the uplift forces.

The factor of safety against hydrostatic uplift 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 for the condition described in [Figure 7-2](#) can be expressed as:

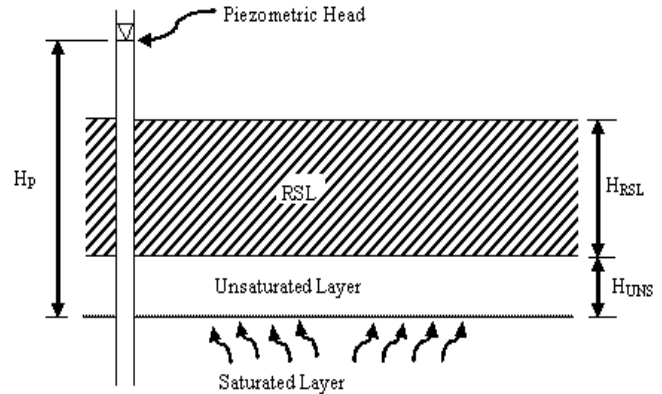


Figure 7-2 Example of uplift pressure acting below liner system

Equation 7-1

$$FS_{Uplift} = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{\gamma_{RSL}H_{RSL} + \gamma_{UNS}H_{UNS}}{\gamma_w H_P}$$

Where,

γ_{RSL} = field density of clay liner, pcf

γ_w = density of water, pcf

γ_{UNS} = field density of *unsaturated* foundation soil, pcf

H_{RSL} = thickness of recompacked soil liner, ft

H_{UNS} = thickness of *unsaturated* foundation soil, ft

H_P = *piezometric* level above the *unsaturated* foundation soil (head), ft

Heaving, Concept and Analysis

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.

Heaving occurs when the effective stress in the soil approaches zero; at this point, there will not be any surface contact between the soil particles, leading to the formation of surface cracks, or the more severe case of soil dispersing below the surface. Heave is typically analyzed by comparing the seepage force exerted by the groundwater with the effective or buoyant unit weight of the overburden counteracting materials.

The factor of safety against heave should be calculated in any area of the *waste containment unit* where a temporal high *phreatic* or *piezometric surface* is higher than the top of the recompacked soil

liner and/or the bottom of excavations. Heave at the bottom of an excavation is through the mechanism of *bearing capacity* failure; the factor of safety against heave may be obtained by modifying Terzaghi's *bearing capacity* theory as presented below.

Equation 7-2

$$FS = \frac{11c}{\gamma H}$$

Where,

c = soil cohesion ($\Phi = 0$), psf

H = depth of excavation, ft

γ = bulk unit weight of soil, pcf

Theory presented in Janbu et al (1966) suggests assigning a more conservative constant to the equation above, perhaps between 5 and 8, taking into account depth and width of the excavation. However, Ohio EPA has found through experience with Ohio landfills that a constant of 11 is sufficient, especially as the evaluation of whether or not landfill construction is stable is generally governed by factor of safety against seepage rather than heave.

Seepage Potential – Piping or Internal Erosion, Concept and Analysis

Piping, or internal erosion, usually occurs when the seepage (drag) forces exceed the cohesive (resisting) forces between the soil particles. This phenomenon will ultimately lead to the formation of soil erosion channels called soil pipes. Once soil piping starts, the flow in the soil pipe will continue to increase due to the decrease in resistance to the flow (friction loss) and the increase in the velocity head, potentially resulting in structural damage to the *waste containment unit* components.

Soil Gradation and Piping Damage. There is a high risk of soil piping in a permeable sandy/gravelly formation. A soil layer is considered to be prone to piping damage when the following conditions are met:

1. There is more than 10% by weight of particles finer than 0.25 mm;
2. There is a lack of particles with a grain size in the range of 0.5 to 2 mm;
3. The coefficient of uniformity is greater than 20; and,
4. The coefficient of curvature is greater than 3.

When two soils with similar grain size distribution curves are being compared, the soil with the higher friction angle will tend to exhibit a higher critical gradient and will therefore be less prone to piping damage.

Soil piping has been witnessed at several excavation sites in Ohio. At those sites, seepage pressures (hydraulic gradients) formed hairline cracks and soil boils at the ground surface. As the internal

erosion progressed, several soil pipes formed between the ground surface and the source of the groundwater pressure, thereby undermining the integrity of the foundation soil and liner system and providing a direct conduit between the bottom of the landfill and the aquifer system.



Figure 7-3 Example of piping through an excavation

[Figure 7-3](#) depicts a case where the *piezometric* water pressure overcame the resistance of the internal structural forces of over 20 feet of in-situ clay and ultimately created a piping condition through the wall of an excavation at an Ohio landfill, which in turn caused flooding of the excavation.



Figure 7-4 Example seepage through the foundation

This cloudy disturbance in the flooded excavation depicted in [Figure 7-4](#) is evidence of the upward flow of seepage water. During construction of the *waste containment unit* and prior to backfilling of excavations, quality assurance and control personnel should document the presence or absence of any seeps as well as the efforts made to mitigate the formation of seeps within the excavations. All discovered seeps should be over-excavated and backfilled with material meeting US Army Corps of Engineers filter design criteria (see USACE 2000 – EM1110 Appendix D) and allow for appropriate drainage to an underdrain system. Seepage from surface water structures may also cause piping failures. Water pooled in a surface



Figure 7-5 Piping failure in recompacted soil liner at an Ohio landfill

water ditch was attributed to the piping failure in the recompacted soil liner depicted in [Figure 7-5](#).

This chapter discusses three methods (analytical method, flow net method, and finite element analysis method) to determine the seepage potential at a site.

Analytical Method

The seepage potential factor of safety can be determined by comparing the actual hydraulic gradient (i_{act}) with the critical hydraulic gradient (i_{cr}) as shown in the following formula:

Equation 7-3

$$FS = \frac{i_{cr}}{i_{act}}$$

The actual hydraulic gradient, i_{act} , is defined as the rate of total head loss or energy dissipation through the soil matrix.

Equation 7-4: for zones of saturation

$$i_{actual} = \frac{\Delta h}{\Delta L} = \frac{\text{top of piezometric surface elev.} - \text{top of clay liner elev.}}{\text{top of clay liner elev.} - \text{top of water bearing unit elev.}}$$

Equation 7-5: for uppermost aquifers

$$i_{actual} = \frac{\Delta h}{\Delta L} = \frac{\text{top of piezometric surface elev.} - \text{bottom of excavation elev.}}{\text{bottom of excavation elev.} - \text{top of water bearing unit elev.}}$$

Where,

Δh = head loss between two points, ft

ΔL = apparent flow distance at which the hydraulic gradient is being measured, ft

Head loss increases linearly with increasing velocity when the flow is in the laminar and transitional phases; the relationship becomes nonlinear in the turbulent phase. Once flow becomes turbulent, even if velocity is reduced, the flow will remain turbulent in part throughout the transition zone until the laminar zone is reached again. As such, once seepage damage begins, engineering intervention is required to mitigate it.

When considering the effect of the hydraulic gradient, the effective vertical stress may be defined by:

Equation 7-6

$$\sigma'_v = \sigma_v - iu$$

As the hydraulic gradient increases, iu approaches σ_v and σ'_v approaches zero. At this point, the gradient approaches the critical gradient, i_{cr} , which can be determined by any of the following methods:

Equation 7-7

$$i_{cr} = \frac{\sigma_v}{u} = \frac{\gamma'}{\gamma_w}$$

The restricting force of the waste mass may be added in the following formula only when an underdrain system beneath the liner drains to a sump and is then pumped to the surface.

$$i_{cr} = \frac{(\gamma_{MSW} * h_{MSW}) + (\gamma_{DL} * h_{DL})}{z\gamma_w} + \frac{\gamma_{RSL} - \gamma_w}{\gamma_w}$$

Where,

γ_{MSW} = Unit weight of MSW

h_{MSW} = Placed height of MSW

γ_{DL} = Unit weight of drainage layer

h_{DL} = Depth of drainage layer

z = thickness of RSL

γ_w = Unit weight of water

γ_{RSL} = Saturated unit weight of RSL

Equation 7-8

$$i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{e + 1}$$

Where,

i_{cr} = critical hydraulic gradient, unitless

γ' = submerged unit weight, pcf

γ_w = unit weight of water, pcf

G_s = soil specific gravity, unitless

e = soil void ratio, percent

The actual internal and exit hydraulic gradients can be calculated using a finite element or a flow net method. The answer obtained from these methods should be compared to the critical hydraulic gradient, i_{cr} , calculated using the above formulas.

As long as the actual internal and exit hydraulic gradients are less than the critical gradient, i_{cr} , the seepage is expected to be in accordance with the principle of Darcy's law, and the permeability of the soil should remain constant. This suggests that there will be minimal disturbance to the soil structure and that internal erosion will not occur.

However, when the actual internal or exit hydraulic gradient exceeds the critical gradient, i_{cr} , the hydraulic conductivity of the soil is expected to increase, ultimately resulting in a loss of strength of the foundation soils, leading to soil piping and the formation of soil boils at the ground surface. This condition, when it occurs in cohesionless soil, is termed as a quick condition. Visual inspection of the flow coming from the soil boils can indicate the severity of the soil piping condition; if coarse

soil particles are observed within the flow from the sand boils, the soil piping condition is severe. In this situation, immediate action should be implemented to remedy the problem.

Flow Net Method

The flow net analysis method can be used to calculate seepage through the use of the continuity equation, which will result in a special form of the equation known as the “Laplace Equation.” In three dimensions, the equation takes the following form:

Equation 7-9

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} + \frac{\delta^2 h}{\delta z^2} = 0$$

Graphical representation of this equation will yield ellipses for the flow lines and hyperbolas for the equipotential lines. The two families of curves will intersect at 90-degree angles to form a pattern of peculiar square figures. Derived from Darcy’s Law, the following form of the above equation can be used to determine the flow rate of seepage from the groundwater table into an excavation or from an elevated ground reservoir (e.g., sedimentation pond) through the embankment.

Equation 7-10

$$q = Kh_L \frac{N_f}{N_d}$$

Where,

q = seepage flow rate, ft^3/sec

k = soil hydraulic conductivity, ft^2/sec

h_L = total head loss, ft

N_f = total number of flow channels in the flow net, unitless

N_d = total number of equipotential drop lines in the flow net, unitless

Finite Element Analysis Method (FEM)

Finite element models have been developed to perform seepage analysis for *saturated* or *unsaturated* steady and unsteady state flow conditions. The basic steps involved in most finite element programs are:

1. Select the cross-section to be modeled;
 - a. The section geometry is entered into the finite element program;
 - b. The cross-section is discretized by dividing it into smaller sections or elements (this option is automatic in most finite element programs, but some finite element programs allow the user to select the type of element to be used in the finite element analysis).
2. Input hydraulic conductivity(ies) of material(s) and initial fluid head or potential; and,

3. Apply accurately determined boundary conditions.

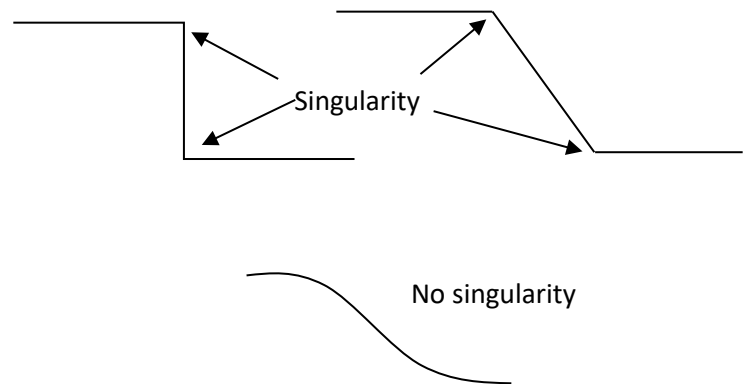
These finite element programs can calculate flow, potential, gradient, and velocity relationships that will allow the designer to solve for unknown components at each node established by the discretizing step, which incorporates element stiffness matrices and equations. The programs will assemble the stiffness matrices, account for the known boundary conditions and solve for the unknowns.

Finite element modeling programs are used to solve the partial differential flow equation, which is undefined at points of singularity. Therefore, unless the surface has sharp and abrupt changes, surfaces should be modeled as they are encountered in the field (i.e., transition between the floor and side slope should be modeled as a curve instead of as a sharp angle).

Erosion Rate Index for Plastic Soils, Analysis

The factors of safety specified in this policy assume also that the soil has an extremely slow erosion rate, with an erosion rate index, $I_{erosion}$, greater than 6.0. The erosion rate index represents the combination of the rate of erosion and the critical shear stress of the soil when erosion is expected to begin. The rate of erosion is dependent on such soil characteristics as gradation, fines content (clay particles), Atterberg limits, in-situ water content and unit weight, and the soil mineralogy, including whether it is cemented or prone to cementing.

Figure 7-6 Example of singularity vs no singularity



The erosion rate index for plastic soils may be estimated using the following equation developed by Wan and Fell, 2004:

Equation 7-11

$$I_{erosion} = 0.153\gamma_d - 0.042RD + 0.1\omega + 0.0097\Delta\omega_r - 0.0056F + 0.042Clay - 0.09LL + 0.11I_p + 0.44P - 10.2$$

Where,

- $I_{erosion}$ = erosion rate index
- γ_d = soil dry density, lb/ft³
- RD = relative density of the soil, % (use 95 for undisturbed foundation soil)
- ω = water content, %
- $\Delta\omega$ = water content ratio, % = $\frac{\omega - OMC}{OMC} \times 100$
- OMC = optimum water content, %
- F = percent of fines in the soil, %

Clay = percent of clay (0.005mm) in the soil, %
 LL = liquid limit, %
 I_p = plasticity index, %
 P = Pinhole test classification value

The following table provides guidance for selecting the pinhole test classification value.

Dispersiveness Category	Pinhole Test Classification Value	Description
D1	1	Dispersive clays that fail rapidly under 2 inches of head
D2	2	
ND4	3	Slightly to moderately dispersive clays that erode slowly under 2 to 7 inches of head
ND3	4	
ND2	5	Nondispersive clay with very slight to no colloidal erosion under 15 to 40 in of head
ND1	6	

The dispersiveness category is based on ASTM D 4647, Methods A&C.

Qualitative descriptions of the erosion rate index are as follows:

Erosion Rate Index (I_{erosion})	Description
<2	Extremely rapid
2 – 3	Very rapid
3 – 4	Moderately rapid
4 – 5	Moderately slow
5 – 6	Very slow
>6	Extremely slow

The *responsible party* should ensure that all authorizing documents and the QA/QC plan clearly require that the design and specifications developed through hydrostatic uplift and seepage analyses for the *waste containment facility* be followed and that the conditions of the analyses are confirmed throughout the construction, operations and closure phases.

Occasionally, the conditions on which the hydrostatic uplift and seepage analyses were based might change because of alterations made to the *waste containment unit* design throughout its lifetime. If this occurs, a formal request to change the *waste containment facility* design is required and shall be submitted for Ohio EPA approval in accordance with applicable rules. The request to change the

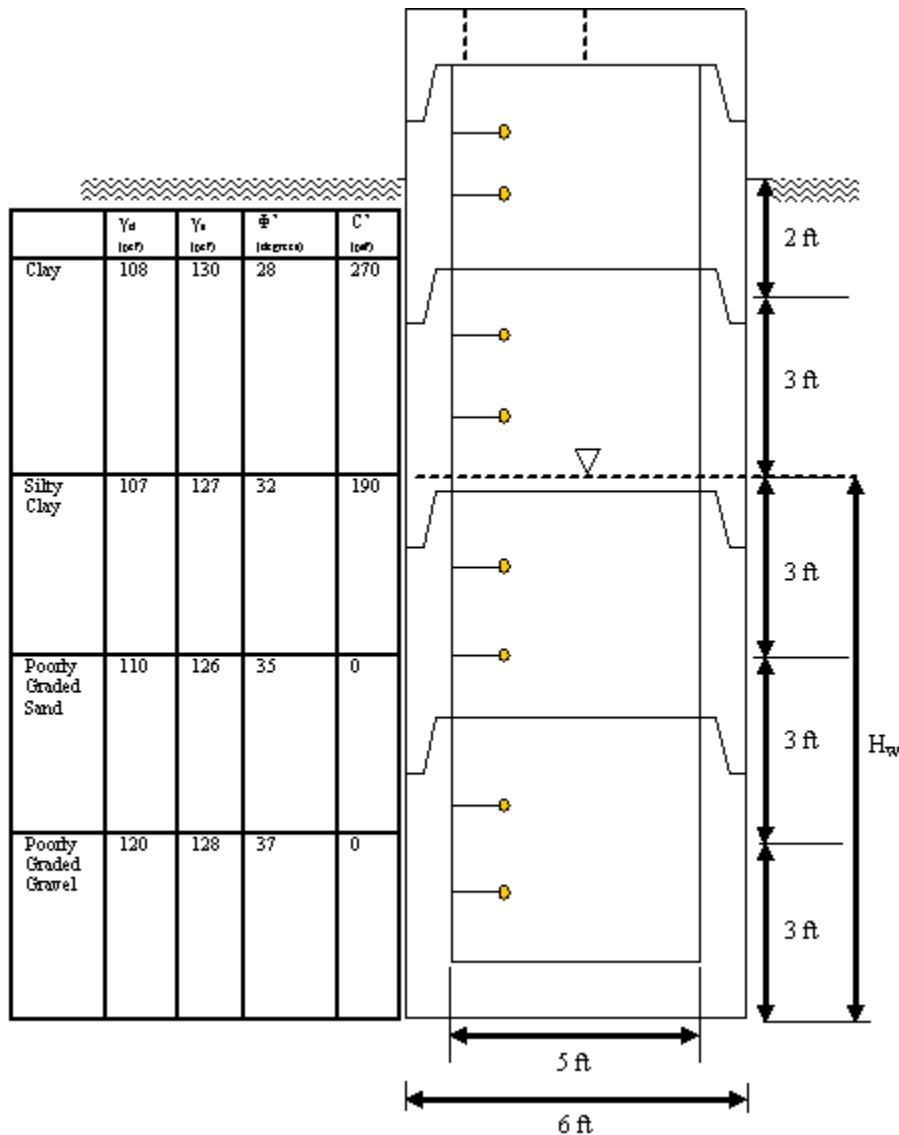
waste containment facility design must include hydrostatic uplift and seepage analyses conditions and specifications updated as per the changes made to the *facility* design.

APPENDIX 7-A Examples

Hydrostatic Uplift of a Pump Station - Example Calculation

Find the factor of safety against hydrostatic uplift for the pumping station.

Assume the pumping station walls are coated with impermeable epoxy.



$$FS_{Uplift} = \frac{\text{Pullout Resistance}}{\text{Uplift Pressure}}$$

$$FS_{Uplift} = \frac{\text{Wall Friction} + \text{Weight of Soil above Extended Base} + \text{Deadmen Resistance} + \text{Structure Weight}}{\text{Uplift Pressure}}$$

In this case, we do not have an extended base or deadmen, therefore,

$$FS_{Uplift} = \frac{\text{Wall Friction} + \text{Structure Weight}}{\text{Uplift Pressure}}$$

$$\text{Uplift Pressure} = \gamma_w A_o H_w$$

$$\text{Uplift Pressure} = (62.4 \text{pcf}) \left(\frac{\pi(6 \text{ft})^2}{4} \right) (9 \text{ft}) = 15,871 \text{lbs}$$

$$\text{Wall Friction} = K_o \gamma_b \frac{H}{2} A_o \tan \varphi' + A_o C_s$$

$$\text{Weight of Soil above Extended Base} = (A_o - A_i) H \gamma_b$$

Where,

K_o = at rest pressure coefficient = $1 - \sin \varphi'$, unitless

γ_b = bouyant soil weight = $\gamma_{sat} - \gamma_w$, lbs

A_o = outside area of manhole shaft, ft^2

A_i = inside area of manhole shaft, ft^2

C_s = saturated cohesion, psf

Therefore, the wall friction along the different layers is as follows:

Wall friction = friction along gravel + friction along sand
+ friction along silty clay + friction along clay

$$\begin{aligned} \text{Friction along gravel} &= K_o \gamma_b \frac{H}{2} A_o \tan \varphi' + A_o C_s \\ &= (1 - \sin \varphi') (\gamma_{sat} - \gamma_w) \frac{H}{2} (2\pi r H) \tan \varphi' + A_o C_s \\ &= (1 - \sin 37)(128 - 62.4) \frac{3}{2} (2\pi(3)(3)) \tan 37 + A_o(0) \\ &= 1,670 \text{ lbs} \end{aligned}$$

$$\begin{aligned} \text{Friction along sand} &= (1 - \sin 35)(126 - 62.4) \frac{3}{2} (2\pi(3)(3)) \tan 35 + A_o(0) \\ &= 1,611 \text{ lbs} \end{aligned}$$

$$\begin{aligned} \text{Friction along silty clay} &= (1 - \sin 32)(127 - 62.4) \frac{3}{2} (2\pi(3)(3)) \tan 32 + A_o(0) \\ &= 1,610 \text{ lbs} \end{aligned}$$

$$\begin{aligned} \text{Friction along clay} &= (1 - \sin 28)(130 - 62.4) \frac{3}{2} (2\pi(3)(3)) \tan 28 \\ &\quad + (1 - \sin 28)(108) \frac{2}{2} (2\pi(3)(2)) \tan 28 + A_o(0) \end{aligned}$$

$$= 2,766 \text{ lbs}$$

$$\begin{aligned} \text{Weight of the Structure} &= (A_o - A_i)H\gamma_{\text{concrete}} \\ &= \left(\frac{\pi D_o^2}{4} - \frac{\pi D_i^2}{4} \right) H\gamma_{\text{concrete}} \\ &= \frac{\pi}{4} (D_o^2 - D_i^2) H\gamma_{\text{concrete}} \\ &= \frac{\pi}{4} ((6\text{ft})^2 - (5\text{ft})^2) (19\text{ft}) (150\text{pcf}) \\ &= 24,622 \text{ lbs} \end{aligned}$$

Therefore,

$$FS = \frac{1,670 + 1,611 + 1,610 + 2,766 + 24,622}{15,871} = 2.0$$

Factor of safety without considering wall friction is:

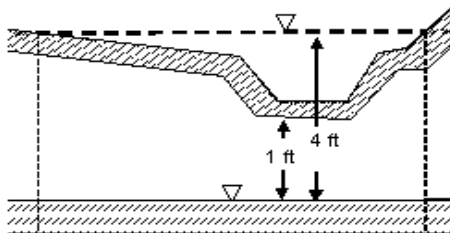
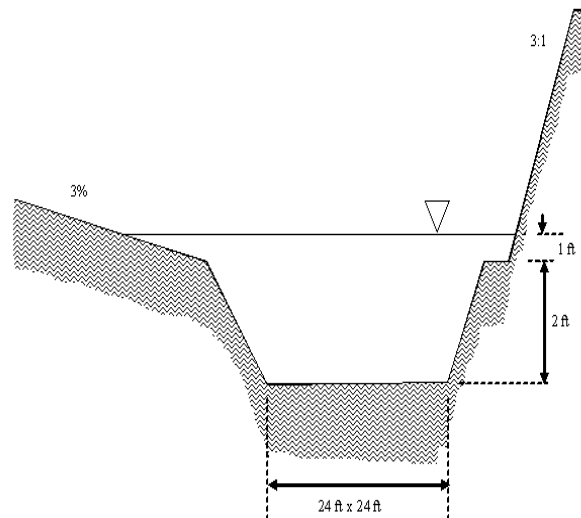
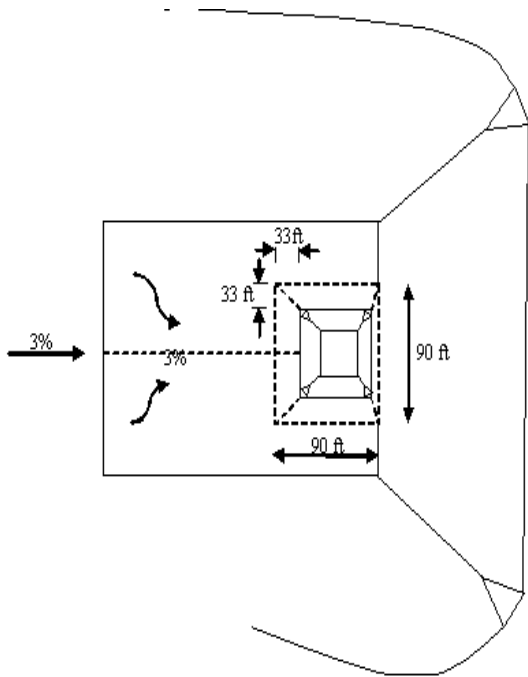
$$FS = \frac{24,622}{15,871} = 1.6$$

In either case, this design is acceptable with regard to hydrostatic uplift, which requires a minimum factor of safety of 1.4.

Hydrostatic Uplift at the Sump - Example Calculation

Given:

- Sump with dimensions of 24 ft x 24 ft x 2 ft
- Floor and flow line slopes = 3%
- *Piezometric* head level is determined to be 1 foot above the sump rim
- Foundation soil has $\Phi = 22^\circ$ and $c = 100$ psf



Since the *piezometric* head is 1 foot above the sump rim, this will mean it will extend a distance equal to 33 feet from the rim if the floor slope is 3%.

The volume of the area delineated by the *piezometric* line intersecting the sump area is the volume of the sump + the volume of the 3 spectra (the volume above the edge of the sump and below the 1 foot of head) + the volume of a small spectrum between the sump and the 3:1 side slope.

Assume the volume of the small spectrum is equal to zero.

$$\text{Volume of the sump} = [24\text{ft} \times 24\text{ft} \times 2\text{ft}] = 1,152 \text{ ft}^3$$

$$\text{Volume of the 3 spectra} = \left[\frac{[(24\text{ft} \times 1\text{ft}) + (90\text{ft} \times 1\text{ft})](33\text{ft})}{2} \right] (3) = 5,643 \text{ ft}^3$$

$$\text{Total volume subject to uplift} = 1,152 \text{ ft}^3 + 5,643 \text{ ft}^3 = 6,795 \text{ ft}^3$$

$$\text{Uplift force} = 6,795 \text{ ft}^3 \times 62.4 \text{ pcf} = 424,008 \text{ lbs}$$

$$\text{Volume of dry soil resisting uplift} = [90\text{ft} \times 90\text{ft} \times 4\text{ft}] - 6,795 \text{ ft}^3 = 25,605 \text{ ft}^3$$

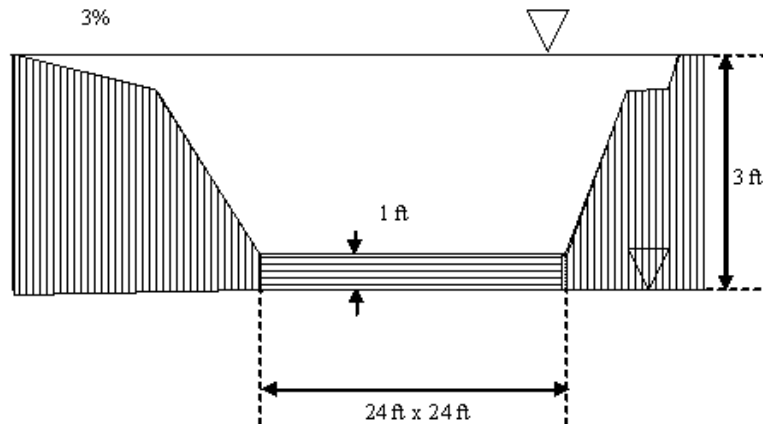
Assume $\gamma_{\text{soil}} = 120 \text{ pcf}$

$$\text{Weight of soil wedge} = 25,605 \text{ ft}^3 \times 120 \text{ pcf} = 3,072,600 \text{ lbs}$$

$$\therefore F.S = \frac{3,072,600}{424,008} = 7.2$$

Note that this is without considering the soil shear resistance to uplift.

Now, let's redo the analysis for the situation shown in the next figure, where the *phreatic* head is only 1 foot below the bottom of the sump and the sump is 2 feet full.



$$\text{Volume subject to uplift} = [24\text{ft} \times 24\text{ft} \times 2\text{ft}] = 1,152 \text{ ft}^3$$

$$\text{Uplift force} = 1,152 \text{ ft}^3 \times 62.4 \text{ pcf} = 71,885 \text{ lbs}$$

$$\text{Volume of dry soil resisting uplift} = [24\text{ft} \times 24\text{ft} \times 1\text{ft}] = 576 \text{ ft}^3$$

$$\text{Resisting force} = 576 \text{ ft}^3 \times 120 \text{ pcf} = 69,120 \text{ lbs}$$

$$\therefore FS = \frac{69,120}{71,885} = 0.96$$

Now, consider the foundation soil shear resistance properties:

$$\tau = c + \sigma \tan \phi$$

$$\sigma = 120 \text{ pcf} \times 1 \text{ ft} = 120 \text{ psf}$$

$$c = 100 \text{ psf}$$

$$\tau = 100 \text{ psf} + 120 \tan 22^\circ = 148 \text{ psf} = c + \sigma \tan \phi$$

$$\sigma = 120 \text{ pcf} \times 1 \text{ ft} = 120 \text{ psf}$$

$$\tau = 100 + 120 \tan 22^\circ = 148 \text{ psf}$$

$$\text{Area resisting uplift below sump} = [24 \text{ ft} \times 1 \text{ ft} \times 4 \text{ sides}] = 96 \text{ ft}^2$$

$$\text{Uplift force} = [24 \text{ ft} \times 24 \text{ ft} \times 2 \text{ ft}] \times 62.4 \text{ pcf} = 71,885 \text{ lbs}$$

$$\text{Resisting force} = 96 \text{ ft}^2 \times 148 \text{ psf} = 14,208 \text{ lbs}$$

Therefore,

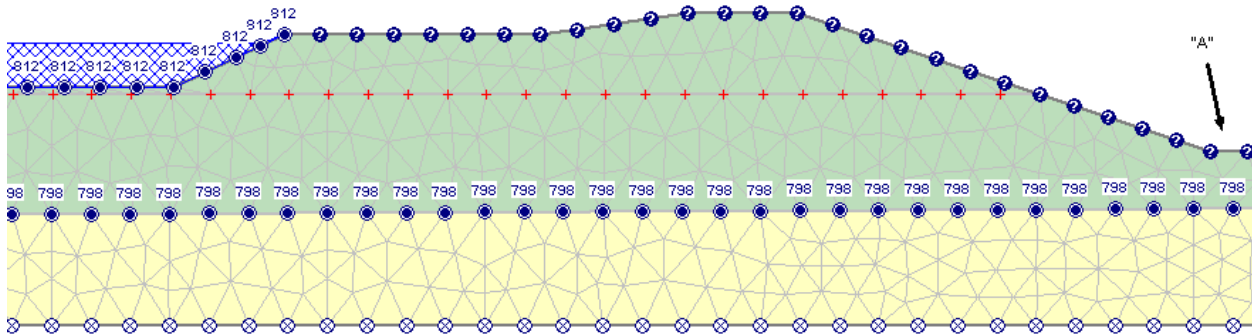
$$FS = \frac{69,120 + 14,208}{71,885} = 1.16$$

Remember, an acceptable factor of safety against hydrostatic uplift is 1.4. Additionally, Ohio EPA does not recommend relying on the soil cohesion or friction properties to calculate the resisting force for the uplift pressure due to the uncertainty associated with soil fracturing.

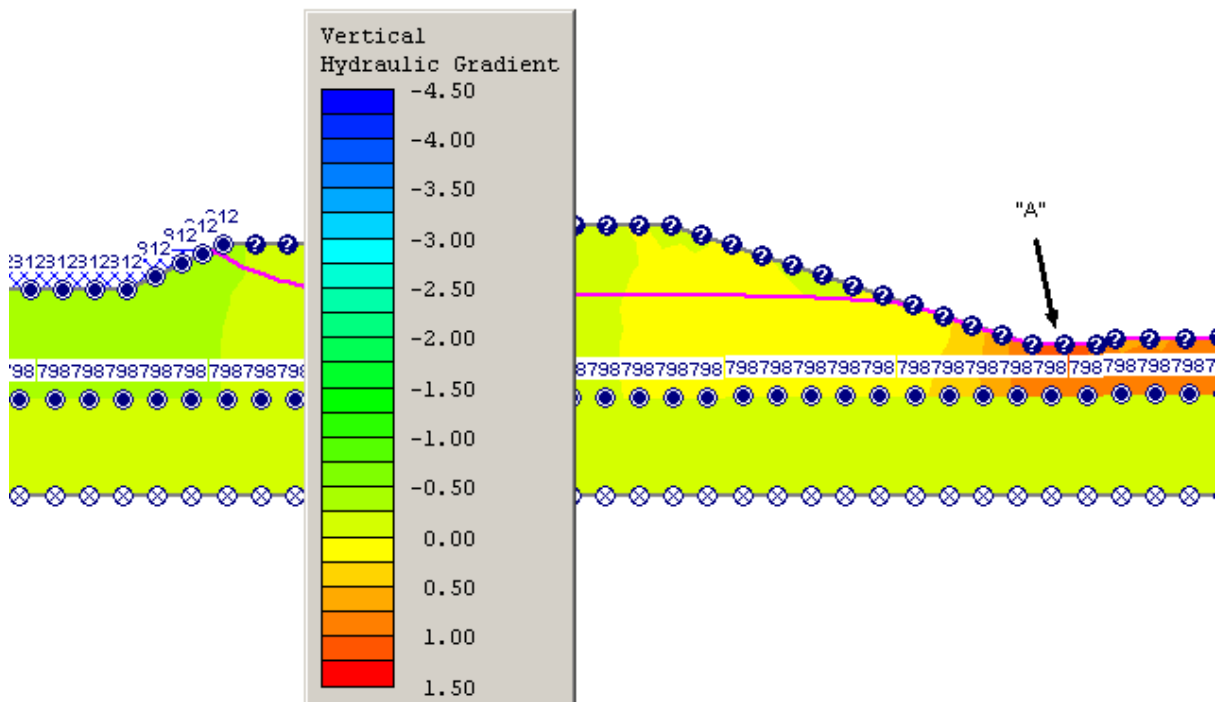
Seepage Analysis using FEM and Traditional Methods - Example Calculation

To assess the seepage potential, the hydraulic gradient due to upward seepage force at the base of excavation should be evaluated on a 100 ft by 100 ft grid, at a minimum, and at the toe of each side slope. For illustrative purpose, the seepage potential is evaluated at point “A,” depicted below, where the following analysis components have been evaluated:

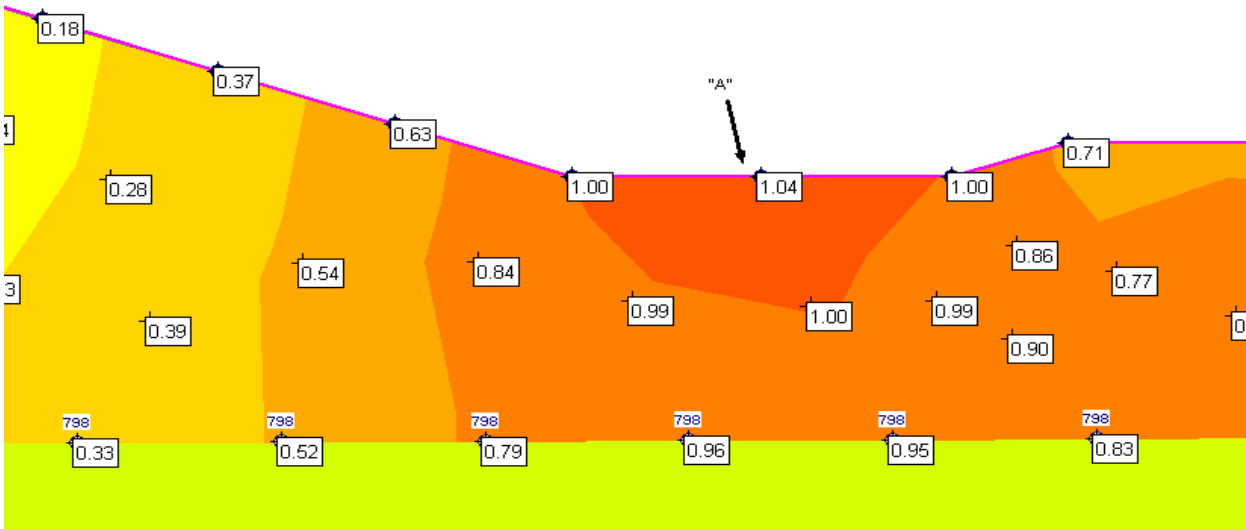
- Specific Gravity Foundation Soil = 2.75;
- Void Ratio = 0.5; and,
- Saturated Unit Weight Soil = 139.2 pcf.



The results of the vertical hydraulic gradients have been determined using a finite element program.



Enlarged view near the area of concern at point “A” indicates that the expected exit gradient for this example is 1.04.



Alternatively, solving manually with [Equation 7-4](#) yields the following gradient at point “A:”

$$i_{act} = \frac{\Delta h}{\Delta L} = \frac{\text{potentiometric surface elev} - \text{water table elev}}{\text{subgrade surface elev} - \text{top of UAS elev}} = \frac{798 - 782.5}{782.5 - 766.5} = 0.97$$

This is comparable to the 1.04 result obtained from the finite element program, with 1.04 being the more conservative result.

The critical gradient is the smaller of:

$$i_{cr} = \frac{\gamma'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} = \frac{135.2 - 62.4}{62.4} = 1.16$$

$$i_{cr} = \frac{G_s - 1}{e + 1} = \frac{2.75 - 1}{0.5 + 1} = 1.17$$

Therefore,

$$i_{cr} = 1.16$$

Conservatively, the smallest factor of safety against seepage per the analyses is as follows:

$$FS = \frac{i_{critical}}{i_{actual}} \quad FS = \frac{1.16}{1.04} = 1.12 > 1.1$$

Therefore, the design at point “A” is acceptable.

Estimation of the Soil Erosion Rate – Example Calculation

An embankment was constructed to the following specifications:

- Average field density = $\gamma_d = 120$ pcf
- Average moisture content = $\omega = 12\%$
- Find the predicted erosion rate index for an embankment that was constructed from soils that have the following properties:
- Maximum dry density = MDD = 130 pcf
- Optimum moisture content = OMC = 10%
- % passing the No. 200 sieve = $F = 65\%$
- % passing the No. 200 sieve = Clay = 35%
- Liquid limit = LL = 28%
- Plastic limit = PL = 15%
- Pinhole test classification = $P = 3$

$$I_{erosion} = 0.153\gamma_d - 0.042RD + 0.1\omega + 0.0097\Delta\omega_r - 0.0056F + 0.042Clay \\ - 0.09LL + 0.11PI + 0.44P - 10.2$$

Where,

$$RD = \frac{\gamma_d}{MDD} \times 100 = \frac{120}{130} \times 100 = 92.3\% \\ \Delta\omega_r = \frac{\omega - OMC}{OMC} \times 100 = \frac{12 - 10}{10} \times 100 = 20\% \\ PI = LL - PL = 28 - 15 = 13\%$$

Therefore,

$$I_{erosion} = 0.153(120) - 0.042(92.3) + 0.1(12) + 0.0097(20) - 0.0056(65) + 0.042(35) \\ - 0.09(28) + 0.11(13) + 0.44(3) - 10.2$$

$$I_{erosion} = 7.0$$

This is greater than 6, so the soil is not prone to erosion, and therefore the normal factors of safety applied for seepage will likely be appropriate.

APPENDIX 7-B Determination of Total Head and the Concept of Seepage Forces

Bernoulli's Equation defines the total head for steady-state, non-viscous, incompressible fluids as follows:

Total Head = Pressure Head (*piezometric*) + Elevation Head (potential) + Velocity Head

$$\text{Total Head (feet)} = \frac{P}{\gamma_w} + Z + \frac{V^2}{2g}$$

The elevation head or potential head is the distance between an arbitrarily selected datum and the point in question. Elevation head can be negative if the point in question falls below the datum line. Bernoulli's equation can be used to determine the gradient and the pore water pressure at a point.

Water seepage through soil imparts energy to soil grains in the form of friction; seepage force can be represented as:

$$F = 62.4iV = 62.4A\Delta h$$

Where,

F = seepage force, lbs

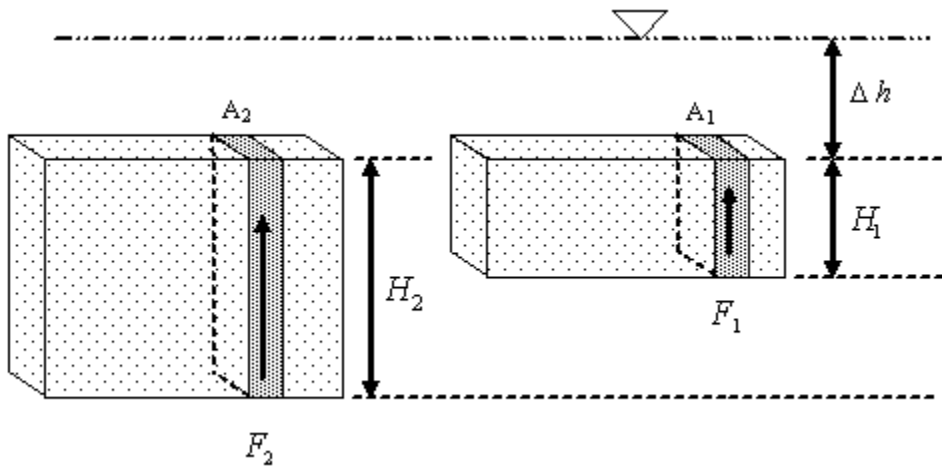
i = average hydraulic gradient, unitless

V = volume on which the hydraulic gradient is acting, ft^3

Δh = head loss between two points, ft

A = Area, ft^2

For a given area and energy loss, the seepage force is constant regardless of the distance over which it travels. The following diagram and proof illustrate this concept:



$$F_1 = 62.4i_1V_1$$

$$i_1 = \frac{\Delta h}{H_1}$$

$$V_1 = A_1H_1$$

$$\therefore F_1 = 62.4 \frac{\Delta h}{H_1} A_1 H_1 = 62.4 \Delta h A_1$$

$$\text{Assume } A_1 = A_2 = A$$

$$F_1 = 62.4 \Delta h A = 62.4 i_1 V_1$$

$$i_1 = \frac{\Delta h}{H_1}$$

$$V_1 = A_1 H_1$$

$$\therefore F_1 = 62.4 \frac{\Delta h}{H_1} A_1 H_1 = 62.4 \Delta h A_1$$

$$\text{Assume } A_1 = A_2 = A$$

$$F_1 = 62.4 \Delta h A$$

Similarly,

$$F_2 = 62.4 i_2 V_2$$

$$i_2 = \frac{\Delta h}{H_2}$$

$$V_2 = A_2 H_2$$

$$\therefore F_2 = 62.4 \frac{\Delta h}{H_2} A_2 H_2 = 62.4 \Delta h A_2$$

$$\text{Assume } A_1 = A_2 = A$$

$$F_2 = 62.4 \Delta h A = 62.4 i_2 V_2$$

$$i_2 = \frac{\Delta h}{H_2}$$

$$V_2 = A_2 H_2$$

$$\therefore F_2 = 62.4 \frac{\Delta h}{H_2} A_2 H_2 = 62.4 \Delta h A_2$$

$$\text{Assume } A_1 = A_2 = A$$

$$F_2 = 62.4 \Delta h A$$

This indicates that $\vec{F}_1 = \vec{F}_2$

Therefore, seepage force will not be reduced by the thickness of the layer it travels through.

REFERENCES

- Abt, S. R., P. L. Thompson, and T. M. Lewis, 1996. "Enhancement of the Culvert Outlet Scour Estimation Equations," Transportation Research Record 1523.
- Cedergren, H. R., 1989, Seepage, Drainage, and Flow Nets, 3rd ed., John Wiley & Sons, Inc., New York, New York.
- Chi Fai et. al., 2004 Investigation of Rate and Erosion of Soils in Embankment Dams, Journal of Geoenvironmental Engineering.
- Das, B. M., 1994, Principles of Geotechnical Engineering, 3rd ed., PWS Publishing Company, Boston, Massachusetts.
- Dunn, I. S., 1959. "Tractive Resistance of Cohesive Channels," Journal of the Soil Mechanics and Foundations, Vol. 85, No. SM3, June.
- Holtz, R. D., and Kovacs, W. D., 1981, An Introduction to Geotechnical Engineering, Prentice Hall, Englewood Cliffs, New Jersey.
- Khilar, K.C., Folger, H.S., and Gray, D.H. (1985). "Model for Piping-Plugging in Earthen Structures", J. Geotech. Engrg., ASCE, 111 (7), 833-846.
- Philip L. Thompson and Roger T. Kilgore, 2006, "Hydraulic Design of Energy Dissipators for Culverts and Channels Hydraulic Engineering Circular Number 14, Third Edition," FHWA-NHI-06-086 HEC 14, U.S. Department of Transportation, Federal Highway Administration.
- <http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>
- Sowers, G. F., 1979, Introductory Soil Mechanics and Foundations, 4th ed., Macmillan Publishing Co., Inc., New York, New York.
- Terzaghi, K., Peck, R. B. and Mesri, G., 1996, Soil Mechanics in Engineering Practice, 3rd ed., John Wiley & Sons, Inc., New York, New York.

Chapter 8

DEEP-SEATED FAILURE ANALYSIS

Table of Chapter Contents

Introduction	8-1
Reporting	8-3
Factors of Safety	8-5
Assigning Shear Strengths (drained , undrained , failed slope , waste , high moisture wastes , interface , seismic , specifying)	8-6
Accounting for the Effects of Water (phreatic , piezometric)	8-13
Analysis (Static Analysis [Long Term , Short Term , Staged Loading], Seismic Analysis , [Screening Analysis])	8-15
Appendix 8-A Examples (static Long Term - Example , seismic Long Term - Seismic Conditions , Short Term - Example)	8-21
References	8-51

INTRODUCTION

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 with a normal load more than 1,440 psf to be a deep-seated failure. Deep-seated failures 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 practice design, material testing, construction, and operations.

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 loading, and the internal and interface shear strengths of the slope components.

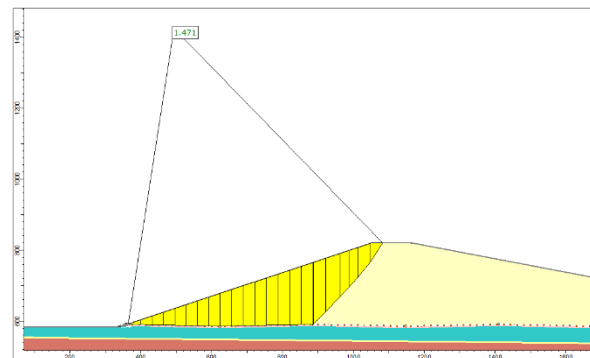


Figure 8-1, Translational Failure

Deep-seated translational failures or block failures occur along weak interfaces or through weak foundation layers. Translational failures are more prevalent at *waste containment units* employing geosynthetics.

Rotational failures or circular failures occur through weak layers of a foundation and possibly a weak waste layer or engineered component of a *waste containment unit*. Rotational failures are more prevalent at *waste containment units* that are made of or filled with weak materials or are supported by weak foundation soils. Rotational failures tend to occur through a relatively uniform material, whereas translational failures tend to occur when dissimilar materials are involved.

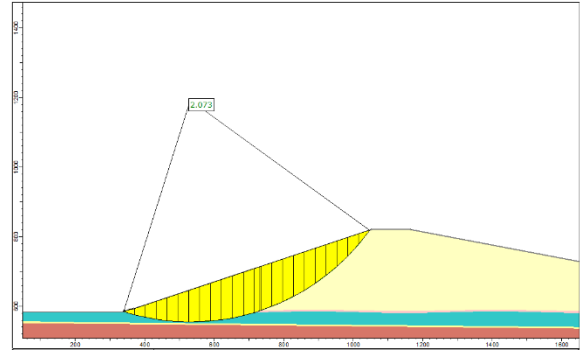


Figure 8-2, Rotational Failure

Most stability modeling programs now also offer composite surface searches which combine both rotational and translational surface at the same time which will typically yield the most realistic failure surface. Designers may choose to use this method as long as they examine both foundational and liner involved failure surfaces and can clearly depict this to Ohio EPA's reviewer.

Most of the slope failures that have occurred in Ohio have occurred under static conditions. Ohio, however, has experienced over 250 felt earthquakes since 1900 and has experienced since 1995 at least two earthquakes with peak ground accelerations exceeding 0.2g. 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 a magnitude of 8.0 occurred in New Madrid, Missouri damaging buildings in Ohio. Thus, seismic stability should be considered at all Ohio *waste containment facilities*.

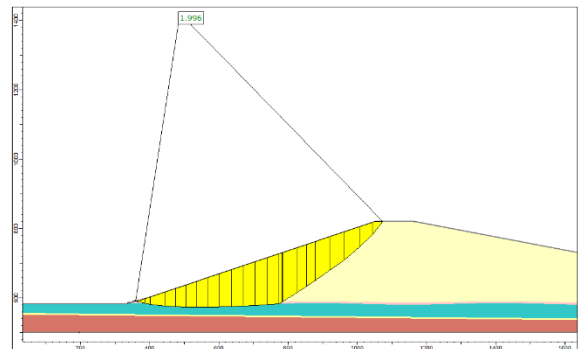


Figure 8-3, Composite Surface

REPORTING

This section describes the information that should be submitted to demonstrate that a *waste containment unit* has an acceptable margin of safety for deep-seated translational and rotational failures. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report:

Conclusions

- 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.

Basis of Design

- A narrative justifying the internal and interface shear strengths of the various components of the *internal*, *interim*, and *final slopes*.
- One or more tables summarizing the input parameters used for the deep-seated failure analyses. The tables should include the internal and interface shear strengths of the various components of the internal, *interim* and *final slopes*.
- The scope, extent, and findings of the subsurface investigation as they pertain to the analyses of potential deep-seated failures at the *waste containment unit*.
- 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 identify the slip surface with the lowest factor of safety 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 and temporal high *phreatic* and *piezometric surfaces*.
 - the rationale 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 location of each analyzed cross-section and the limits of the *waste containment unit(s)*. Northings and eastings of the cross-section should be indicated on the plan view (e.g., see [Figure 8-20](#), [Figure](#)

8-21, and Figure 8-22, and Table 8-6). These plan views are typically submitted on 11 x 17 paper with a scale of one inch to 300 feet showing the top of intermediate cover/ drainage layer/ final cover, the top of recompacted soil liner, and the existing contour in different colors.

Any figures, drawings, or references relied upon during the analysis should be properly cited. These figures, drawings, or references may be requested separately for review by Ohio EPA.

- 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.
 - material interface shear strengths (peak or residual, as applicable).
 - material internal shear strengths (*drained* or *undrained*, as applicable).
 - a depiction of each critical failure surface and its factor of safety.
 - the engineered components of the *waste containment unit*.
 - the engineered components of the *waste containment unit*.



Figure 8-4, 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.

Any drawings or cross-sections referred to in this document 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.

Calculations

- Long-term static stability calculations for *internal*, *interim*, and *final slopes* assuming *drained conditions* beneath the *waste containment unit*.
- If one or more clay or silt units exist below the *waste containment facility*, calculate short-term static stability for *internal*, *interim*, and *final slopes* assuming *undrained conditions* in the appropriate *soil units* beneath the *waste containment unit*. These *undrained soil units* should be

examined using both translational and rotational failure search methods. The worst-case factor of safety is often found by translational search methods in *undrained soil units*.

- If any short-term static stability calculations for *internal*, *interim*, and *final slopes* yield factors of safety that are ≤ 1.3 , the *waste containment unit* should be redesigned or be analyzed for intermediate-term partially *drained conditions* by performing a staged loading calculation.
- Long-term seismic stability calculations for *final slopes*.
- Electronic copies of the input files or the spreadsheets used to perform the stability calculations.
- Any other calculations used for the analyses.

FACTORS OF SAFETY

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

Long-Term Factors of Safety*

Static analysis: $FS \geq 1.50$

Seismic analysis: $FS \geq 1.00$ or verify that permanent seismic deformations are limited to less than 15 cm.

Short-Term Factors of Safety

Static analysis: $FS \geq 1.30$

The factors of safety specified in this document are based on the assumptions contained in this document. 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 *waste containment unit*. If different assumptions are used, these factors of safety may not be appropriately protective of human health and the environment.

The use of higher factors of safety (i.e. 2.00, and 1.5 respectively) may be warranted if a failure would have a catastrophic effect upon human health or the environment (i.e. slide mass is likely to contact an occupied structure, public road, stream, lake or high-quality wetland).

A *waste containment unit* should be designed to prevent deep-seated failures. If a *waste containment unit* has a static factor of safety against deep-seated failure lower than those listed above, the *unit* should be redesigned.

A *waste containment unit* should be designed to prevent excessive permanent seismic deformations. If the calculated seismically induced displacements are estimated as being greater than 15 cm at the bottom liner interface, then the *waste containment unit* should be redesigned.

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 *waste containment unit* 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 *waste containment facility* through such means as are appropriate (e.g., regulatory compliance requirements, approval conditions, orders, settlement agreements).

From time to time, changes to the *waste containment unit* 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 *waste containment unit* design is required to be submitted for Ohio EPA approval in accordance with applicable rules. The request to change the *waste containment unit* design must include a new deep-seated failure analysis that uses assumptions and specifications appropriate for the change.

ASSIGNING SHEAR STRENGTHS

Shear strength values may be estimated from empirical correlations for structural fill, recompacted soil components, and foundation materials, unless those materials are considered a *critical layer*. References to the documents where the empirical correlations originate should be provided. Ohio EPA may request a copy of any document referenced. The following are often considered *critical layers* in Ohio:

- Any landslide-prone soil or shale listed on GeoFacts No. 8.
- Any soil with a normalized and corrected for overburden standard penetration test (SPT) blow counts (N'_{60}) ≤ 8 or other field tests that yield an empirical correlation for *undrained shear strength* of ≤ 1000 psf.
- Any material that when modeled with estimated shear strength values yield a static factor of safety less than 1.5.

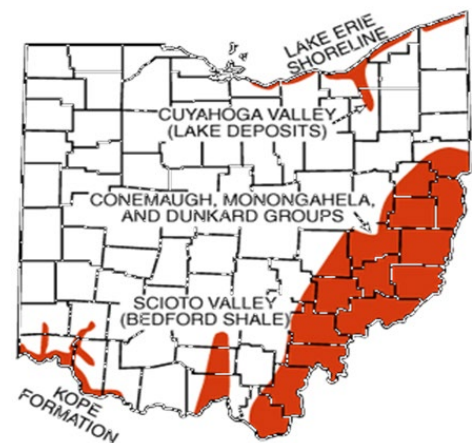


Figure 8-5, Landslide-Prone Soil and Shale (Hansen, 1995). Note additional soils other than those shown above are listed in this fact sheet.

Site-specific shear strength envelopes should be developed for all *critical layers* found at a site using the methods discussed in [Chapter 4](#). If laboratory methods are used, Ohio EPA recommends collecting sufficient quantities of soil from the *critical layer* in three separate locations to run a *consolidated undrained* (CU) tests (using at least 3 separate confining pressures) and a one-dimensional (1D) consolidation tests for each location. The confining pressures used to conduct the CU tests should vary between the three locations. At least two of the confining pressures used in the CU tests should be approximately equal to the effective stress on the *samples* at the time of sampling and at least three normal stresses should be more than the preconsolidation stress (σ'_p) of the *samples* being tested. The shear strength envelopes determined by these tests should be compared and, as appropriate, one envelope may be developed for design purposes. One of the appropriate methods to develop this envelope is to plot the normal stresses versus the shear stress and plot the envelope so that 2/3s of the points fall above the line representing the envelope.

Drained Shear Strength

Drained shear strength (also called long-term shear strength) of a *critical layer* can be modeled using the Mohr-Coulomb effective shear strength envelope ($S_d = c' + \sigma_{ff} \tan \phi'$) determined using Consolidated-Undrained Triaxial Compression (CU) test (ASTM D 4767) with pore pressure measurements or the Consolidated-Drained Triaxial Compression (CD) test.

Undrained Shear Strength

Undrained shear strength (S_u) (also called short-term shear strength) of a *critical layer* can be modeled using only cohesion (c) while setting the friction angle (ϕ) = 0. *Critical layers* that are more than ten feet thick are typically modeled as more than one layer since in situ S_u typically increases with depth since deeper portions of the *critical layer* have consolidated under higher vertical stresses. S_u for modeling purposes can be considered the preconstruction *undrained shear strength* (S_{uo}) which is the normalized undrained shear strength ratio (S_u/σ'_{vc}) obtained from a CU test run at a confining pressure approximately equal to the effective stress on the *sample* at the time of sampling multiplied by the preconsolidation pressure (σ'_p). If soils are soft ($SPT \leq 4$), *samples* are best taken using a piston sampler.

Equation 8-1

$$S_{uo} = \frac{S_u}{\sigma'_{vc}} \sigma'_p = \frac{1/2 (\sigma_1 - \sigma_3)}{\sigma_3} \sigma'_p$$

Where

S_{uo} = Preconstruction *undrained shear strength*

S_u/σ'_{vc} = Normalized undrained shear strength ratio

σ'_p = preconsolidation pressure

σ_1 = Major principal stress

σ_3 = Minor principal stress

Alternatively, S_u can be determined using one of the following methods:

1. Field Vane Shear Test (ASTM D2573) if bore holes are appropriately managed by using weighted drilling mud with a weight of approximately 76 pcf (Ladd, 2003). See Ladd 2003 if performing Field Vane Shear Test below the water table for more information.
2. Cone Penetration Test (ASTM D5778) with results calibrated to laboratory test data for the soil being tested.
3. Unconsolidated undrained (UU) (ASTM D2850) if results from one dimensional consolidation tests taken from the same sample tube indicate that the *sample* was of a *specimen* quality designation (SQD) of either A or B i.e. volumetric strain of $\leq 2\%$ for the volume change during laboratory reconsolidation to the effective vertical stress (σ'_{vo}) on the *sample*. (Terzaghi, 1996) or the change in void ratio between initial loading and a load application equal to the σ'_{vo} (Δe) divide by the initial void ratio (e_o) of ≤ 0.07 for soils with an over consolidation ratio (OCR) of ≤ 2 or $\Delta e/e_o \leq 0.05$ for OCRs between 2 and 4 (DeGroot, 2005).

Table 8-1, Quantification of sample disturbance based on specimen volume change during laboratory reconsolidation to σ'_{vo}

Specimen quality designation (SQD) (Terzaghi 1996)		$\Delta e/e_o$ criteria (Lunne 1997)		
Volumetric Strain (%)	SQD	OCR = 1 – 2 $\Delta e/e_o$	OCR = 2 – 4 $\Delta e/e_o$	Rating*
< 1	A	<0.04	<0.03	Very good to excellent
1–2	B	0.04-0.07	0.03-0.05	Good to fair
2–4	C	0.07-0.14	0.05-0.10	Poor
4–8	D	>0.14	>0.1	Very poor
> 8	E			

* Refers to use of *samples* for measurement of mechanical properties. (DeGroot, 2005)

Failed Slope Shear Strength

If during the site investigation slope failures are found within the limits of the *waste containment facility*, Ohio EPA recommends removing the failure mass prior to *waste containment unit* construction. If soils involved in the failure will remain, the shear strength of the soils should be

modeled using the *large displacement shear strength* of the soils. This can be estimated using the factor of safety of 1.0 for the failure area and back calculating. The value can then be verified using the *large displacement shear strengths* determined from a ring shear test or equivalent.

Waste Shear Strengths

At *waste containment units* that accept mainly municipal solid waste (MSW), a shear strength value may be estimated with the following formula:

Equation 8-2

$$\tau = c + \sigma_n \cdot \tan\left(\phi_o - \Delta\phi \cdot \log\left(\frac{\sigma_n}{p_a}\right)\right) = 313 + \sigma_n \cdot \tan\left(36 - 5 \cdot \log\left(\frac{\sigma_n}{2116.2}\right)\right)$$

where

τ = shear strength,

c = 313 psf (15 kPa), cohesion,

σ_n = total normal stress,

ϕ_o = 36°, friction angle measured at a normal stress of 1 atm,

$\Delta\phi$ = 5°, change of the friction angle over 1 log-cycle change of normal stress,

p_a = 2116.2 psf (101.3 kPa), atmospheric pressure

This equation and the prescribed inputs are derived from 103 large-scale direct shear tests conducted on MSW from 9 different studies. The shear strength envelope that is derived from this equation “is reasonably consistent with those recommended previously by Kavazanjian et al. (1995), Manassero et al. (1997), and Eid et al. (2000) but is based on significantly more large-scale in situ and laboratory data, as well as laboratory data at higher confining stresses.” (Bray et al 2009).

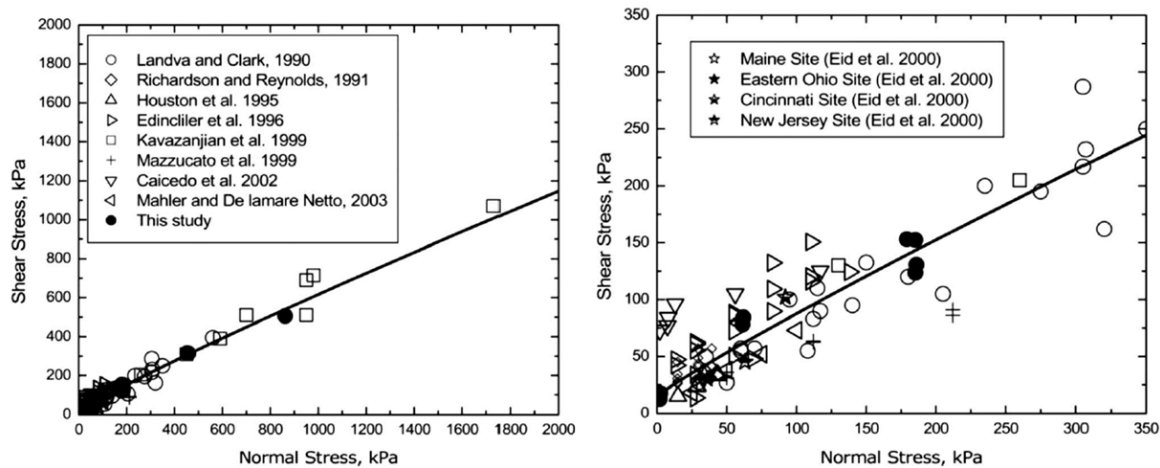


Figure 8-6, Recommended static shear strength of MSW based primarily on direct shear tests and field observations of static slope stability (Bray et al 2009)

For those *waste containment units* that accept only or mostly industrial waste, those industrial wastes should be *sampled* and evaluated appropriately using the method discussed in [Chapter 4](#).

If a landfill has had a significant fire or heating event, it is likely that the high shear strength materials (plastics, etc.) have been destroyed or weakened. So, a new shear strength envelope should be developed for the waste mass. Stark et al 2010 reports shear strength as low as $c = 0$ and $\phi = 20^\circ$ at one *waste containment facility* in northwest Ohio that experienced a significant heating event.

High Moisture Wastes Shear Strengths

“*High moisture wastes*” is a term commonly used for waste such as sludges, oil & gas drill cuttings/mud or other wastes with high moisture content and low shear strength. Many *responsible parties* are self-limiting the quantities of these wastes that they take into any area of the *waste containment unit*. The [Figure 8-8](#) should help *responsible parties* determine what amount of high moisture waste to allow in the *waste containment unit*. If a *responsible party* plans to accept or has accepted more than 25% high moisture waste in any area it is recommended that the shear strength of the waste mass in that area be adjusted appropriately, depending on the degree of waste saturation and on high moisture waste content, when stability modeling is performed.

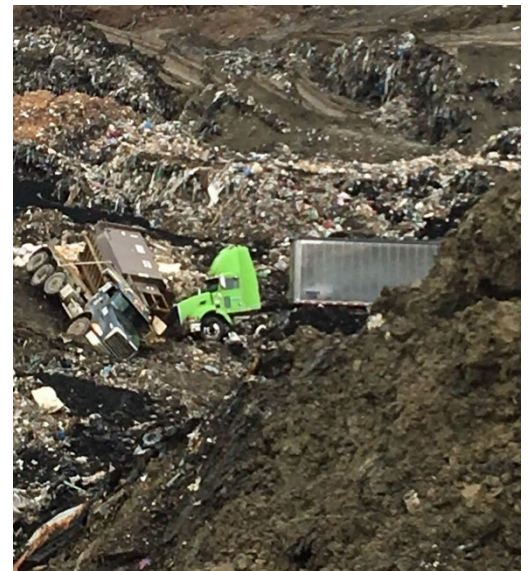


Figure 8-7, Pennsylvania Landfill after slope failure involving high moisture wastes, photo from Metropolitan Engineering Consulting & Forensics blog.

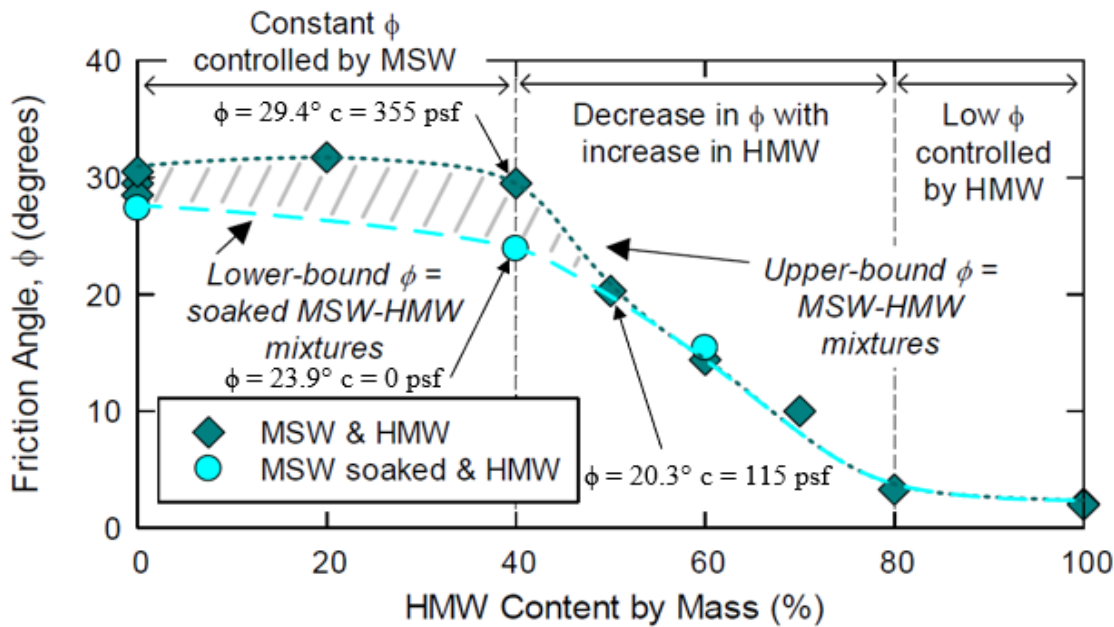


Figure 8-8, High Moisture Waste (HMW) vs. Friction Angle (Scalia et al 2019)

Interface Shear Strengths

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 by back calculating what interface strength will be needed to meet the required factor of safety. This will provide the maximum flexibility for choosing materials during construction. Most static deep-seated slope failures involving a geosynthetic failed under *large displacement shear strengths* on side slopes and *peak shear strengths* on the bottom slopes; therefore, static conditions should be modeled using *large displacement shear strengths* on side slopes and *peak shear strengths* on the bottom slopes.

Seismic Shear Strengths

Seismic shear strengths can be assigned as follows:

- For *saturated* or nearly *saturated* soils, rapid stress application during earthquake shaking occurs as *undrained* loading.
 - Accordingly, fine grained soils with greater than 90% saturation should be modeled during seismic evaluations with the *undrained shear strength* as determined by the normalized undrained shear strength ratio (S_u/σ'_v) assuming the *waste containment unit* has reached final fill height and the effective consolidation stress (σ'_v) after all excess pore pressures from loading have dissipated.
- Liquefiable coarse-grained soils can be modeled with *undrained* residual shear strengths.

- Fine grained soils with less than 90% saturation and non-liquefiable coarse-grained soils can be modeled with effective shear strengths.
- Waste shear strengths should be modeled using [Equation 8-2](#) or as otherwise described in the section on [waste shear strengths](#).
- Interfaces can be modeled using *large displacement shear strengths* on side slopes and *peak shear strengths* on the bottom slopes when performing screening evaluations and using the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* being used for construction when performing deformation analysis.

Specifying Shear Strengths in Authorizing Documents

The internal and interface shear strengths of construction materials must be verified before construction (see Conformance Testing in [Chapter 4](#) starting on page [4-15](#)). All geosynthetic interfaces should be tested twice upon initial selection and the material(s) with the *lowest peak shear strength* at each load interval being tested shall be retested prior to each construction event. The compound nonlinear *peak shear strength* envelope developed from these tests shall be greater than or equal to the shear strength envelope determined as necessary by the back calculations indicating the required safety factor for interfaces with a slope of 5% or less or for interfaces that will never be loaded with more than 1,440 psf. The compound nonlinear *large displacement shear strength* envelope developed from the material(s) with the *lowest peak shear strength* at each load interval shall be greater than the shear strength envelope determined as necessary by the back calculations indicating the required safety factor for interfaces with a slope of greater than 5% that will be loaded with more than 1,440 psf.

The *responsible party* should ensure that the design and specifications in all authorizing documents and the quality assurance quality control (QA/QC) plans clearly require that the assumptions and specifications used in the analysis for the *waste containment 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 *waste containment facility* through such means as are appropriate (e.g., regulatory compliance requirements, approval conditions, orders, settlement agreements).

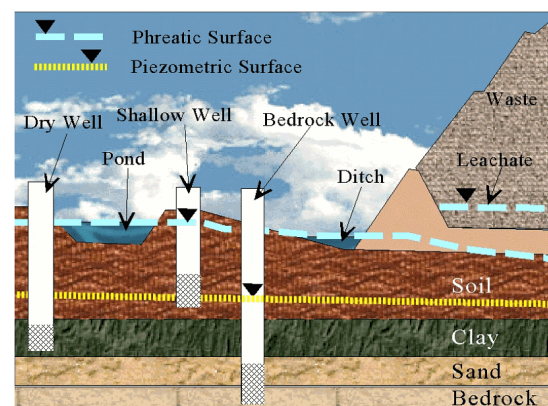


Figure 8-9, Examples of phreatic and piezometric surfaces.

ACCOUNTING FOR THE EFFECTS OF WATER

Water or other liquids are 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.

Phreatic Surfaces

Phreatic surfaces (see [Figure 8-10](#)) 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.
- Perched leachate caused by low permeability wastes or poor removal of daily cover.
- Surface water levels in sedimentation ponds, ditches, streams, rivers, lakes, or lagoons that are part of the cross-section that is being analyzed.
- The groundwater tables associated with *soil units saturated* for only part of their thickness.
- Anticipated levels of water to be found in engineered components such as berms.



Figure 8-10, Looking through the failed containment berm of a storm water retention basin 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 (rapid drawdown conditions). 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 8-11](#)).

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 likely to occur.

For example, modeling is often performed with one foot of leachate head on the liner of a solid waste landfill because, by rule, that is the maximum amount of head allowed. However, after the end of post closure, leachate removal pumps may be turned off resulting in substantially increased head that could threaten the stability of the *waste containment unit*. Another example would be modeling the normal water levels in a wastewater lagoon. However, a heavy rain event may cause the water level in the lagoon to increase by several feet. The *phreatic surface*, in this case, should be modeled at the elevation of the water when it is discharging through the emergency spillway, in addition to an analysis when water is discharging at the elevation of the primary spillway.

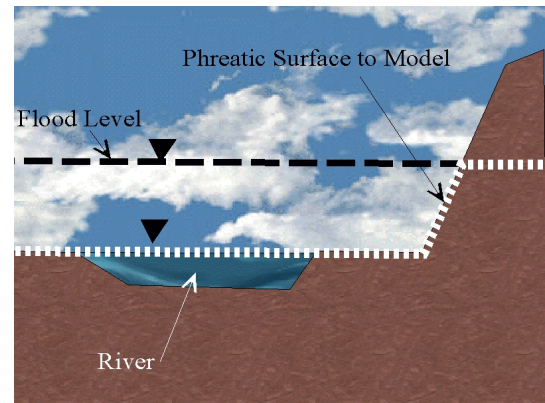


Figure 8-11, Example *phreatic surface to model* to account for porewater pressure created by flooding and then flood

Piezometric Surfaces

Piezometric surfaces (see [Figure 8-12](#)) identified during the subsurface investigation or that can be anticipated to occur must be included as part of all modeling when the failure surfaces being analyzed pass through the unit associated with the *piezometric surface*. *Piezometric surfaces* include, but are not limited to:

- Surfaces that identify the pressure head found in a confined *saturated* layer,
- Surfaces that identify the pressure head found beneath an engineered component of a *waste containment unit* that act as an aquiclude to an underlying *saturated soil unit* (see [Figure 8-12](#)).

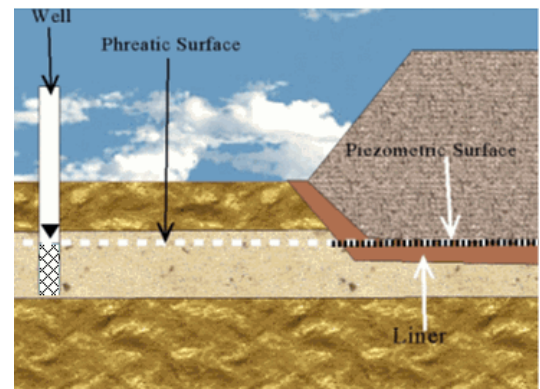


Figure 8-12, Example of a *piezometric surface* created by engineered components of a waste containment facility.

Generally, *piezometric surfaces* should 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 8-12](#), 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. However, in instances where the

clay unit has an in-situ moisture content near its liquid limit, the *piezometric surface* of the sand unit should be applied to both the sand and the clay unit. This is most common in lacustrine clay deposits.

ANALYSIS

Three types of slopes are 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. All *final slopes* need to remain stable indefinitely.

Most if not all the following analyses will be modeled using a computer program performing a limit equilibrium method such as Spencer's Method and Morgenstern-Price Method. When using computer programs that allow a variable number of randomly generated failure surfaces, a sufficient number of slip surfaces should be used to assure that the worst-case slip surface has been located. This may require 5,000 or more searches depending on the size of the search boxes, search areas, and the length of the cross-sections or the modeling program being used. 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.

Static Analysis

The configuration of the *waste containment facility* or proposed *facility* should be examined by an experienced engineer to determine what *internal slope*, *interim slope*, and *final slope* cross-sections are most likely to have a low factor of safety. Slopes with low factors of safety typically are long and steep with very little buttressing. For example, an *interim slope* built with a relatively short base liner compared to its *internal slope*, is less stable than a *waste containment unit* built with more base liner than *internal slope*.

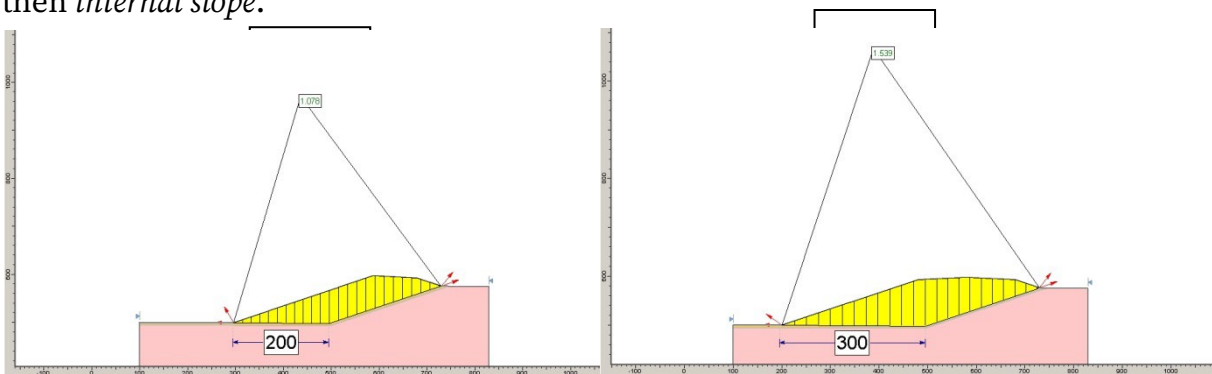


Figure 8-13, Comparison between *internal slopes* with short base liner to those with longer base liner

In the above example, all the shear strengths and slopes are identical except the slope on the right is buttressed with 100 feet of additional waste.

Internal slopes should be examined at the point of maximum excavation, *interim slopes* should be examined when the maximum amount of waste has been placed in a phase of the *waste containment unit* or whenever the design engineer feels it is most critical, and *final slopes* should be examined after *waste containment facility* closure is complete.

The design engineer should then develop cross-sections along these slopes that extend laterally well beyond the toe and the crest of the slope being examined. The rationale for the extension is to prevent deep wide failure surfaces from being inadvertently missed due to insufficient cross-section size. Once these cross-sections are chosen, they should then be entered into a slope stability modeling program that calculates the factor of safety using two-dimensional limit equilibrium methods. Care should be taken to accurately enter the cross-section.

Long-Term Calculations

For all cross-sections the long-term stability, i.e. drained stability, should be determined using the effective shear strengths of the various materials or interfaces using both a translational and rotational stability analyses, composite searches should be incorporated into both of these analyses if possible. The translational analyses should examine failure surfaces along geomembrane interfaces and through thin weak soil or shale layers. Rotational analyses should examine failure surfaces that initiate in the area around the crest of the slope and terminate in the area around the base of the slope.

If the *waste containment unit* design does not meet the required 1.50 factor of safety for long-term *drained conditions*, the *unit* should be redesigned. This redesign often consists of reducing *interim* and external slopes, adding benches to slopes, or increasing the size of external berms.

Short-Term Calculations

If one or more clay or silt units exist below the *waste containment facility*, calculate short-term static stability for internal, *interim*, and *final slopes* assuming *undrained conditions* in the appropriate *soil units* beneath the *facility*. Several *internal slope*, *interim slope*, and *final slope* cross-sections should be examined using composite surface searches or both translational and rotational failure search methods to determine the worst-case short-term factor of safety at the *waste containment facility*.

The worst-case factor of safety is often found by translational search methods with search boxes within the *undrained soil units*. The translational analyses should ideally examine failure surfaces that begin near the toe of the slope, travel circularly through the upper portion of the *undrained soil unit*, then horizontally through the weakest portion of the *undrained soil unit*, then travel through the drained units above the *undrained soil unit* and often exit past the crest, see [Figure 8-14](#). Rotational analyses should examine failure surfaces that initiate in the area around the base of the slope and terminate in the area around the crest of the slope.

As stated in the [undrained shear strength](#) (S_u) section, *undrained shear strength* of a *critical layer* can be modeled using only cohesion (c) while setting the friction angle (ϕ) = 0. *Critical layers* that are more than ten feet thick are typically modeled as more than one layer since in situ *undrained shear strength* typically increases with depth since deeper portions of the *critical layer* have consolidated under higher vertical stresses. *Undrained shear strength* for modeling purposes can be determined using the normalized undrained shear strength ratio (S_u/σ'_v) obtained from CU tests. S_u/σ'_v may vary with vertical stress, for some soils depending on the soils consolidation history. If soils are soft, *samples* are best taken using a piston sampler. Alternatively, normalized undrained shear strength ratio can be determined using field tests like the Field Vane Shear Test (ASTM D2573) or the Cone Penetration Test (ASTM D5778).

If the soft ground conditions occur sporadically across the site in small pockets or in thin layers and these occurrences won't be removed and replaced with structural fill, then the cross-sections analyzed for short-term stability should focus on the areas with soft ground conditions. If soft ground conditions are widespread at the site, a more global search for the worst-case cross-section is appropriate.

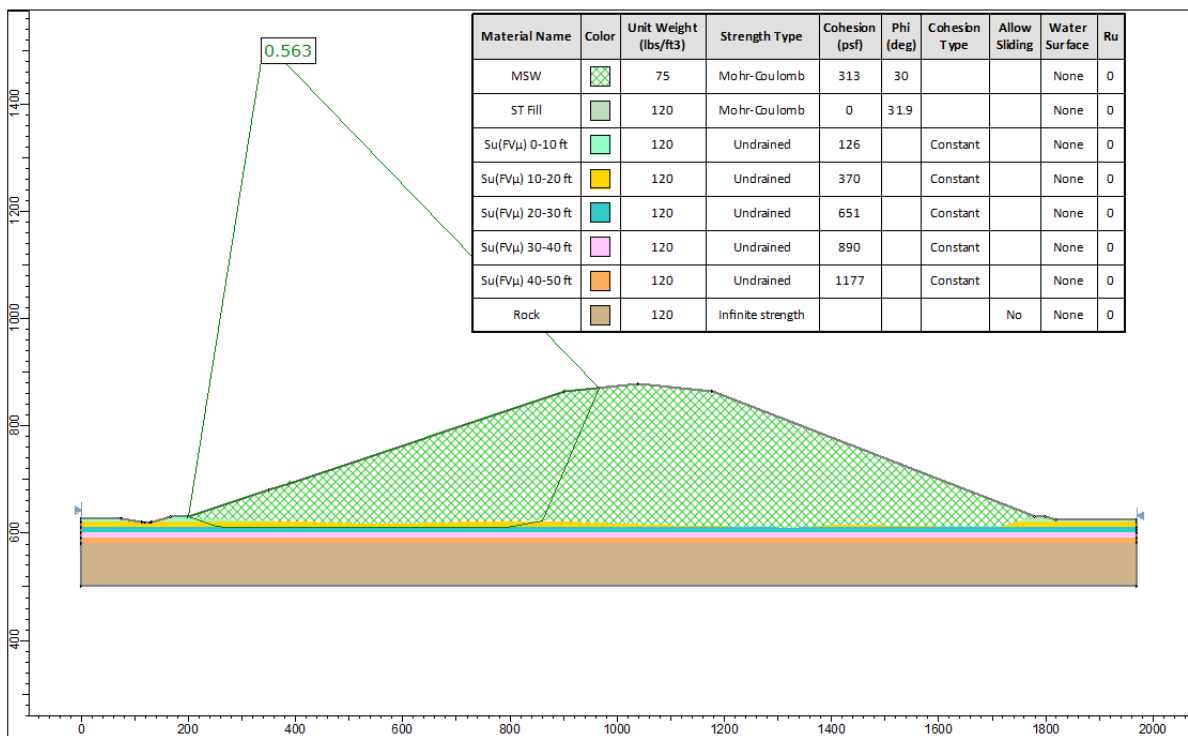


Figure 8-14, Examples of Undrained composite surface search and failure surface. In this example composite surface search yielded a FOS of 0.563, translational (block) search yielded 0.574 and rotational yielded a 1.089.

If the *waste containment unit* design does not meet a 1.30 factor of safety for short-term *undrained conditions*, the *unit* should be redesigned or be analyzed for intermediate-term *partially drained conditions* by performing a staged loading calculation.

Staged Loading Calculation

A staged loading analysis is perhaps the most complicated slope stability analysis performed and should be performed by an engineer with relevant experience with soft soils (e.g. alluvial, lacustrine, and marine deposits). *Responsible parties* wishing to develop on alluvial, lacustrine or marine deposits should recognize the possibility that staged loading calculation may be necessary and evaluate a consultant's experience in this area before hiring them. It may also be advisable to have the staged loading analysis peer reviewed by an engineer experienced in soft soil engineering.

Soft ground conditions that do not meet a 1.30 factor of safety for short-term *undrained conditions* are deemed too weak to support the entire load of a *waste containment unit* without first consolidating the cohesive soil to improve its strength. Methods of improving the soil's strength in these cases include constructing the *waste containment unit* in stages, allowing time for partial consolidation between stages, or more often constructing the *unit* continuously but slowly to allow time for partial consolidation, and/or using wick drains, sand drains or drainage blankets to accelerate the rate of drainage and pore pressure dissipation. As the soft cohesive soil consolidates, its strength increases, and stability is improved. (after Duncan, et al. 2005)

When performing a staged loading calculation, Ohio EPA recommends making the following assumptions:

- The soft soil is completely *saturated*.
- Soil particles and water are incompressible.
- Darcy's Law is valid and can be generalized for anisotropic medium.
- The effective vertical stress generated by a stage is applied instantaneously at the end of the time it takes to construct the stage.
- The bottom liner of a landfill is impervious and will prevent the upward drainage of pore water pressure unless a drainage blanket is constructed below the composite liner system.

If the *waste containment unit* design does not meet the required 1.30 factor of safety for intermediate-term partially *drained conditions*, the *unit* should be redesigned. The loading rate used in the staged loading calculations should be clearly summarized and set as the maximum allowed loading rate for the *waste containment unit*.

Seismic Analysis

The United States Geological Survey (USGS) 2014 version of the National Seismic Hazard Mapping Program (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) predicts that over 90% of Ohio has a Maximum Horizontal Acceleration in Rock (MHA_{Rock}) less than 0.10 g. Which indicates that damage to a *waste containment facility* due to an earthquake is unlikely in Ohio. Therefore most sites will only need to perform a screening analysis. Most sites that do not pass this screening analysis will need to perform a method similar to the simplified procedure outlined in either Bray et al's *Simplified Seismic design Procedure for Geosynthetic-Lined, Solid-Waste landfills* as published by Geosynthetics International in 1998 or Richardson et al's *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities* 1995. Sites built on soft clay or materials subject to liquefaction (Uniform Building Code Site Category F) (see [Chapter 5](#) for more information on Liquefaction) should not use the screening analysis or the simplified procedure. These sites should perform a site-specific probabilistic determination of the horizontal acceleration using a one-dimensional site response program followed by a pseudo-static slope stability analysis. This evaluation should either be checked by or performed by someone with significant experience in doing this type of analysis.

Ohio earthquakes occur at depths of about 3 to 6 miles along faults in Precambrian *bedrock*. “Very few faults in Ohio are visible at the surface, and no surface faults in Ohio are known to be associated with historic earthquakes. Furthermore, none of these faults exhibit evidence of movement during recent (Holocene) time; most of them probably have not been active since the Paleozoic. In fact, fault scarps generated by historic earthquakes have not been clearly demonstrated in the eastern United States. Most faults known in the sub surface of Ohio appear to originate in Precambrian rocks and die out upward in the overlying Paleozoic rocks. Therefore, most of them do not reach the surface where they can be directly mapped and studied. In addition, the *bedrock* surface over about two-thirds of the state is masked by a thick cover of glacial sediments.” (Hansen 2012) It should be assumed that an unknown fault exists near any *waste containment facility* in Ohio.

Screening Analysis

1. Determine the peak ground acceleration (PGA) for the location where the *waste containment facility* is located using site-specific USGS NSHMP's data for a spectral period of 0.0 seconds i.e. the peak ground acceleration based on a probability level of 2% exceedance in 50 years and a V_{s30} of 760 m/s.
2. Perform pseudo-static analyses for the *final slope* cross-sections that have low static factors of safety. Modeling geosynthetic interfaces using *large displacement shear strengths* on side slopes and *peak shear strengths* on the bottom slopes and the other materials with the shear strengths

as described in the [Seismic Shear Strengths](#) section. Setting the horizontal seismic load coefficient (k_H) for the base of the *waste containment unit* as $k_H = \text{PGA}$.

3. If the factors of safety determined by these analyses are ³ 1.0 the calculated seismically induced displacements will be 0 cm for the base of the *waste containment unit*, and the seismic stability analysis can stop here.

If the factors of safety are < 1.0 designers should perform a method similar to the Simplified Procedure outlined in Bray et al 1998 or Richardson et al 1995. If the Simplified Procedure shows the calculated seismically induced displacement is more than 15 cm the *waste containment unit* should be redesigned.

APPENDIX 8-A Examples

Deep Failure – Example Calculations

The following example examines multiple *internal*, *interim*, and *final slopes* to find the factor of safety for the worst-case deep-seated translational and rotational failure surfaces assuming *drained conditions* and, where appropriate, *undrained conditions*. Next, it examines the seismic factor of safety assuming *drained conditions* for the *final slope* with the worst-case cross-section (i.e. the lowest factor of safety) determined by the static *unsaturated* analysis.

Long-Term - Example Calculation

Long-term deep-seated analyses are used to challenge the in-situ foundation materials under the waste mass to ensure that they provide a static factor of safety of at least 1.50 and to prevent excessive seismic deformation. The deep-seated analyses are also used to determine the minimum shear strength necessary to provide a static factor of safety of at least 1.50 through the composite liner system and to prevent excessive seismic deformation along the composite liner system.

A 54-acre landfill is proposed to be located in southern Allen County Ohio. The existing contours slope gently to the southwest. The *waste containment unit* has 3(h):1(v) *internal slopes*, and 4(h):1(v) *interim slopes*, and *final slopes*. The liner system comprises 3 feet of recompact soil liner, a 60-mil textured FML, a geotextile cushion layer, a 1-foot granular drainage layer, and a geotextile filter layer. Standard penetration tests (SPTs), performed at a frequency of one per four acres, found that the *waste containment facility* is underlain by approximately:

- 25 to 30 feet of stiff moist brown lean clay (CL) with N_{60s} that ranged from 14 to 20 with an average of 16.
- 12 to 18 feet of medium to stiff wet grey clayey silt (ML-CL) with some fat clay (CH) with N_{60s} that ranged from 6 to 13 with an average of 8.
- 4 to 8 feet of *saturated* loose to medium dense sand (SM) with N_{60s} that ranged from 9 to 16 with an average of 11. The *piezometric surface* for this layer was located about 25 feet below the surface.
- Limestone *bedrock*.

Assigning Shear Strengths

The *responsible party* has limited the *waste containment unit* to receive less than 40% high moisture wastes like sludges and oil and gas drill cuttings/mud in any area of the *unit*. Therefore, the following non-linear shear strength envelope is being assigned to the waste using the [Equation 8-3](#).

Equation 8-3 (Bray et al 2009)

$$\tau = c + \sigma_n \cdot \tan \left(\phi_o - \Delta\phi \cdot \log \left(\frac{\sigma_n}{p_a} \right) \right)$$

where

τ = shear strength,

c = 313 psf (15 kPa) - cohesion,

σ_n = total normal stress,

ϕ_o = 36° - friction angle measured at a normal stress of 1 atm,

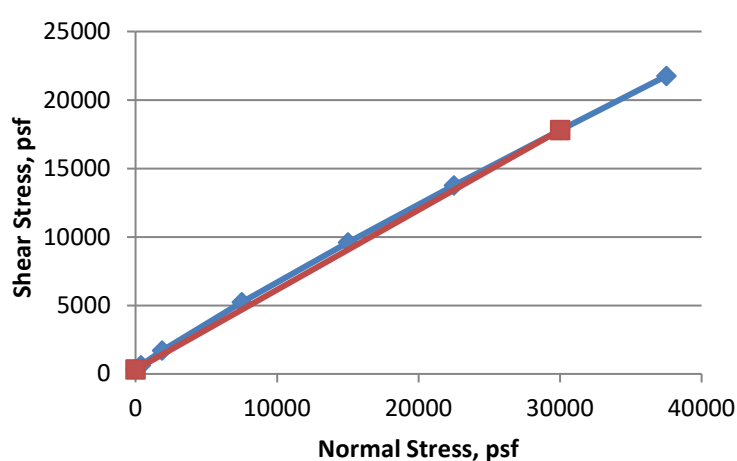
$\Delta\phi$ = 5° - change of the friction angle over 1 log-cycle change of normal stress,

p_a = 2116.2 psf (101.3 kPa) - atmospheric pressure

$$\tau = 313 + \sigma_n \cdot \tan \left(36 - 5 \cdot \log \left(\frac{\sigma_n}{2116.2} \right) \right)$$

A linear shear strength envelope of $c = 313$ psf and $\phi = 30^\circ$ is a conservative approximation of this non-linear envelope for waste thickness up to 400 feet.

Shear Strength MSW



Normal Stress (σ_n)	Shear Strength(τ)
psf	psf
0	313
375	625
1875	1688
7500	5231
15000	9594
22500	13761
30000	17803
37500	21753

Figure 8-15, MSW Shear Strength

Unit Weight of Waste

The unit weight of waste will be estimated using the method proposed by Zekkos et al in “Unit Weight of Municipal Solid Waste” 2006. The following equation estimates the unit weight of municipal solid waste (γ) at a certain depth of waste; if this calculation is performed repeatedly to a modeling depth then a running average can be applied to estimate the average unit weight. Since the *responsible party* plans to accept mostly municipal solid waste (MSW) with limited contaminated soil and limited industrial waste, typical MSW inputs are used.

Equation 8-4, Unit Weight of MSW at a Certain Depth

$$\gamma = \gamma_i + \frac{z}{\alpha + \beta z}$$

Where

γ = unit weight of municipal solid waste

γ_i = initial unit weight of municipal solid waste

z = depth

α = hyperbolic parameter

β = hyperbolic parameter

Table 8-2, Recommended Inputs (Zekkos et al 2006)

Compaction effort & Soil content	γ_i (kN/m ³)	β (m ³ /kN)	α (m ⁴ /kN)
Low	5	0.1	2
Typical	10	0.2	3
High	15.5	0.9	6

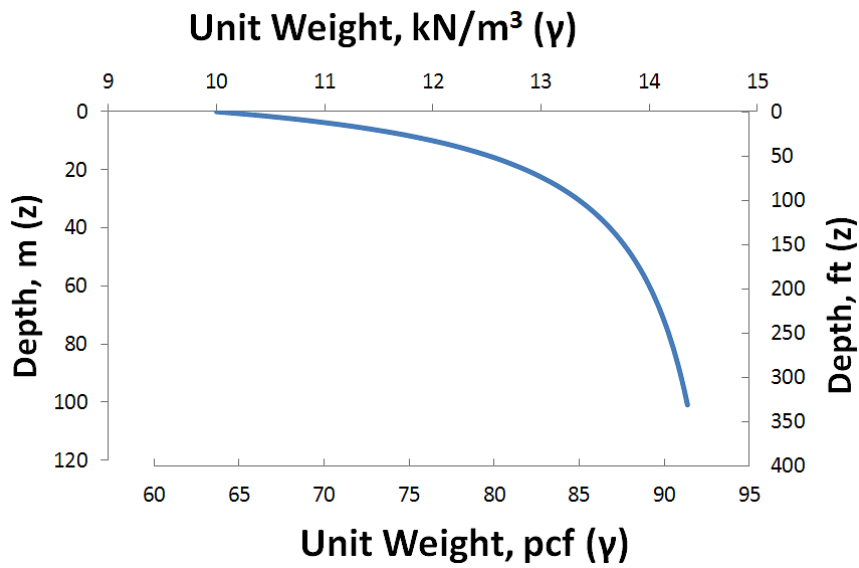


Figure 8-16, Chart of Typical MSW Unit Weights

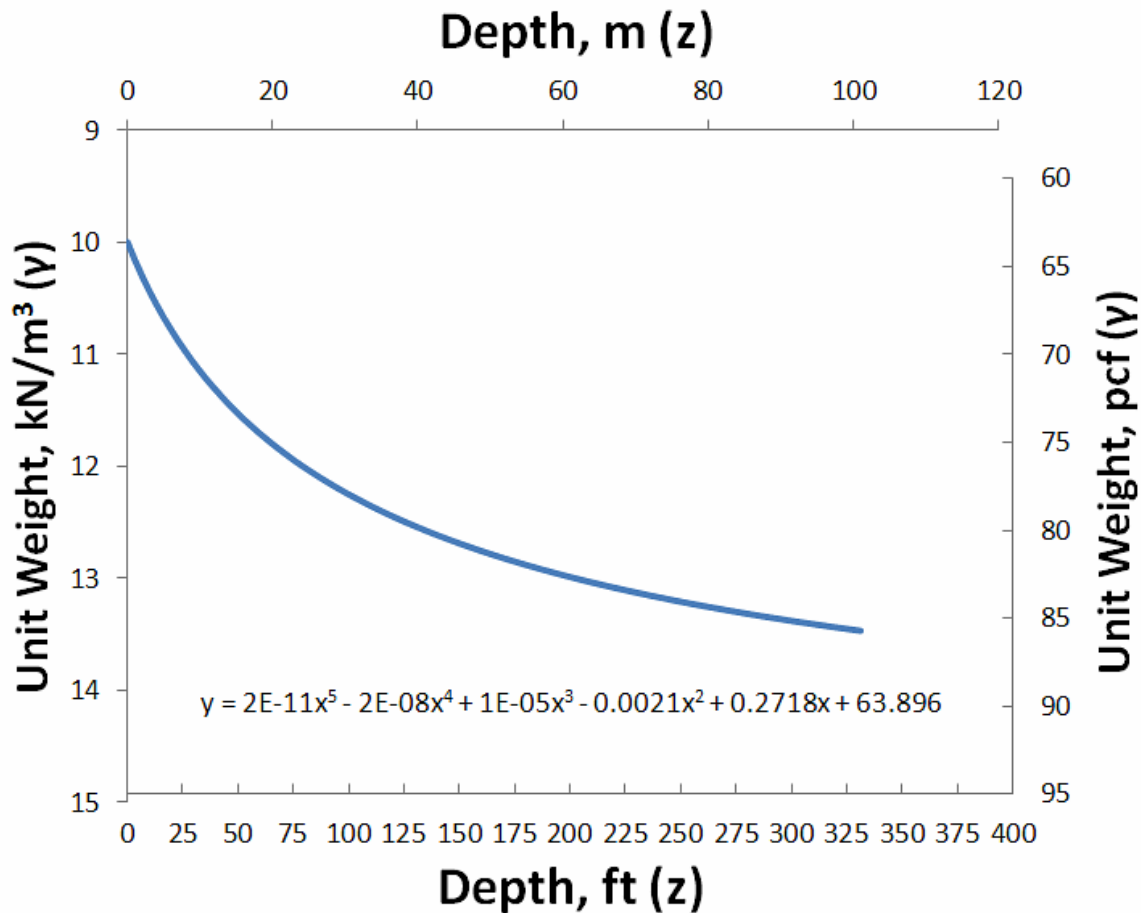


Figure 8-17, Typical Unit Weight Running Average

Using the excel trendline equation of the running average shown in [Figure 8-17](#) the unit weight of waste for this example problem is 75 pcf since the average waste depth is 64 feet.

Gravel Drainage Layer

The shear strength of the gravel drainage layer is estimated from [Table 8-3](#) “Typical Properties of Compacted Soils” from the Naval Facilities Engineering Command’s, Design Manual 7.02, “Foundations & Earth Structures.” This table indicates that a poorly graded clean gravel or gravel sand mixture has an effective shear strength of greater than 38° and zero cohesion and a dry unit weight ranging from 115 to 125pcf. To be conservative, 38° and a wet unit weight of 135 pcf is used.

Geosynthetic Interfaces

The interface shear strengths of all the geosynthetics making up the bottom liner can be modeled as one layer however the thickness will depend on the software’s ability to search for translational failure surfaces. Software with the ability to search along a line that is within the layer representing the interfaces may use a 60 mil thickness; software without this capability should use one foot as the thickness. This layer will represent the weakest shear strength of the interfaces. The interfaces shear strength needed to construct a stable *waste containment unit* will be calculated using the following steps.

1. In this example, all slopes with a slope over 5% are given an initial shear strength with an adhesion (a) of 22.8 psf and an interface friction angle (δ_δ) of 10° representing a *large displacement shear strength* and all slopes less than 5% are given an initial shear strength with an adhesion of 30.1 psf and an interface friction angle of 14° representing a *peak shear strength*. If all cross-sections meet the factor of safety of 1.5 proceed to step 2, if not, choose higher initial shear strengths.
2. In order to back calculate the needed interface shear strengths, the slope with the lowest factor of safety is analyzed. The shear strengths in this analysis will be varied until the factor of safety of 1.50 is reached. In order to develop a *peak shear strength* envelope that is able to also meet a 1.50 factor of safety for shallow slope stability assuming an interface shear strength of 26.5° with no adhesion, the shear strength at a normal load of 120 psf (one foot of drainage layer) will be maintained throughout the calculation at 60 psf ($a = \sigma_v \tan \beta = 120 \times \tan(26.5) = 60$). Similarly, the *large displacement shear strength* envelope is developed using a 1.25 factor of safety for shallow slope stability assuming an interface shear strength of 22.6° with no adhesion. Therefore, the shear strength at a normal load of 120 psf (one foot of drainage layer) will be maintained throughout the calculation at 50 psf

Table 8-3, Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight. Pcf	Range of Optimum Moisture, Percent	Typical Value of Compression		Typical Strength Characteristics				Typical Coefficient of Permeabilit y ft./min.	Range of CBR Values	Range of Subgrade Modules k lbs/cu in.
				At 1.4 tsf (20 psi)	At 3.6 tsf (50 psi)	Cohesion (as com- pacted) psf	Cohesion (saturated) psf	ϕ' (Effective Stress Envelope Degrees)	Tan ϕ'			
				Percent of Original Weight								
GW	Well graded clean gravels, gravel-sand mixtures	125-135	11-8	0.3	0.6	0	0	>38	>0.79	5x10 ⁻²	40-80	300-500
GP	Poorly graded clean gravels, gravel-sand mix	115-125	14-11	0.4	0.9	0	0	>37	>0.74	10 ⁻¹	30-60	250-400
GM	Silty gravels, poorly graded gravel-sand-slit	120-135	12-8	0.5	1.1			>34	>0.67	>10 ⁻⁶	20-60	100-400
GC	Clayey gravels, poorly graded gravel –sand-clay	115-130	14-9	0.7	1.6			>31	>0.60	>10 ⁻⁷	20-40	100-300
SW	Well graded clean sands, gravelly sands	110-130	16-9	0.6	1.2	0	0	38	0.79	>10 ⁻³	20-40	200-300
SP	Poorly graded clean sands, sand-gravel mix	100-120	21-12	0.8	1.4	0	0	37	0.74	>10 ⁻³	10-40	200-300
SM	Silty sands, poorly graded sand-silt mix	110-125	16-11	0.8	1.6	1050	420	34	0.67	5x>10 ⁻⁵	10-40	100-300
SM-SC	Sand-silt clay mix with slightly plastic fines	110-130	15-11	0.8	1.4	1050	300	33	0.66	2x>10 ⁻⁶	5-30	100-300
SC	Clayey sands, poorly graded sand-clay mix	105-125	19-11	1.1	2.2	1550	230	31	0.60	5x>10 ⁻⁷	5-20	100-300
ML	Inorganic silts and clayey silts	95-120	24-12	0.9	1.7	1400	190	32	0.62	>10 ⁻⁵	15 or less	100-200
ML-CL	Mixture of inorganic silt and clay	100-120	22-12	1.0	2.2	1350	460	32	0.62	5x>10 ⁻⁷		
CL	Inorganic clays of low to medium plasticity	95-120	24-12	1.3	2.5	1800	270	28	0.54	>10 ⁻⁷	15 or less	50-200
OL	Organic silts and silt-clays, low plasticity	80-100	33-21								5 or less	50-100
MH	Inorganic clayey silts, elastic silts	70-95	40-24	2.0	3.8	1500	420	25	0.47	5x>10 ⁻⁷	10 or less	50-100
CH	Inorganic clays of high plasticity	75-105	36-19	2.6	3.9	2150	230	19	0.35	>10 ⁻⁷	15 or less	50-150
OH	Organic clays and silty clays	65-100	45-21								5 or less	25-100

Notes:

- All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "Modified Proctor" maximum density.
- Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
- Compression values are for vertical loading with complete lateral confinement.
- (>) indicates that typical property is greater than the value shown.
(..) indicates insufficient data available for an estimate.

(NAVFAC DM-7.02, 1986)

($a = \sigma_v \tan \beta = 120 \times \tan(22.6) = 50$). The adhesions for these envelopes were calculated using the following equations while varying the interface shear strength until the factors of safety of 1.50 and 1.25 are achieved.

Equation 8-5, Peak Shear Strength

$$a = 60\text{psf} - \tan(\delta) \times 120\text{psf}$$

Equation 8-6, Large Displacement Shear Strength

$$a = 50\text{psf} - \tan(\delta) \times 120\text{psf}$$

Where

a = adhesion (psf)

δ = interface friction angle

Soils

The example *waste containment facility* is underlain by three *unconsolidated soil units*. None of these soils are listed as being landslide-prone on ODNr's Geo-Factsheet Number 8, but one of them has SPT N_{60} values less than or equal to 8. The upper *soil unit* was *sampled* in three locations with Shelby tubes and tested using one-dimensional consolidation tests. The cohesive *soil unit* with SPT N_{60} values less than or equal to 8 was *sampled* in three separate locations. Two *samples* were taken from each of the three locations using a piston sampler to obtain sufficient material for testing. Then *samples* from each location were tested using one-dimensional consolidation tests and CU tests with pore pressure readings.

Brown CL

This soil will also be used as the recompacted soil liner (RSL), recompacted barrier layer (RSBL), and the cap *protective layer*.

The Brown CL does not appear to be a critical soil, so the *drained* strengths and *undrained* strengths of this material will be estimated using an empirical correlation. Test results showed the in situ wet unit weights of this *soil unit* to be about 120 pcf.

Empirical Drained Strength

The *drained shear strength* (ϕ') is estimated using an empirical correlation between friction angle and Plastic Index.

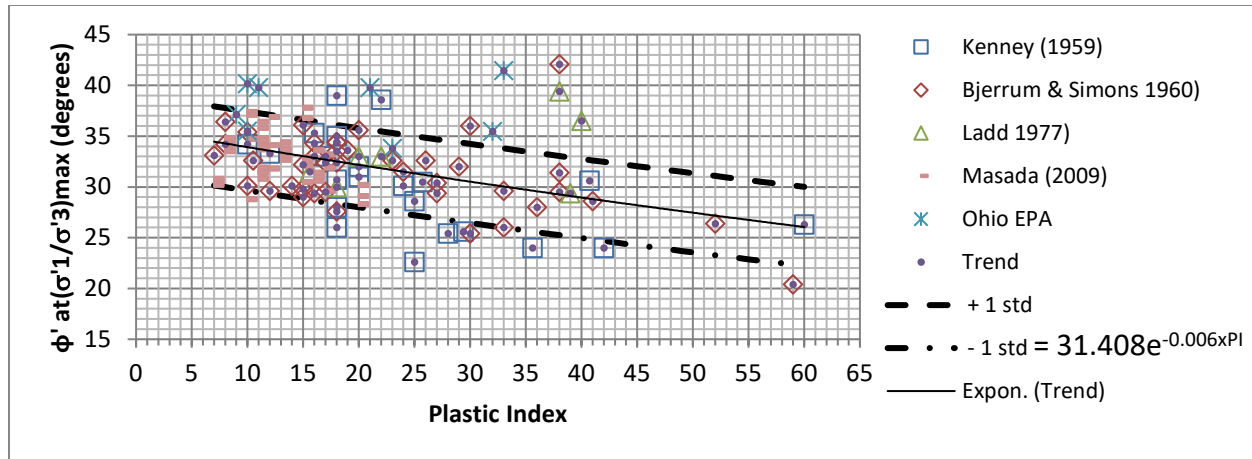


Figure 8-18, Empirical correlation between friction angle and Plastic Index from triaxial tests on normally consolidated clays. (after Holtz, 1981)

Specifically, the following equation from the -1 standard deviation is used to calculate effective stress friction angle.

Equation 8-7

$$\phi' = 31.408e^{-0.006 \times PI}$$

The highest plastic index (PI) determined for the Brown CL is 26. Therefore, the friction angle component of this soil's *drained shear strength* is considered as

$$\phi' = 31.408e^{-0.006 \times 26} = 26.9^\circ$$

Empirical Undrained Strength

The *undrained shear strength* (S_u) will be estimated using an empirical correlation between standard pressure test N values and soil classification.

Table 8-4, Relationships Between Undrain Shear Strength and N values for Cohesive Soils (after Sowers, 1979)

Type of Clay	Undrain Shear Strength (psf)
High plasticity (CH)	$S_u = 260 \text{ N}$
Medium plasticity (CL)	$S_u = 152 \text{ N}$
Low plasticity and clayey silt (SC-ML)	$S_u = 76 \text{ N}$

This soil is classified as a lean clay (CL) and the lowest number of blow counts (N) is 13 therefore:

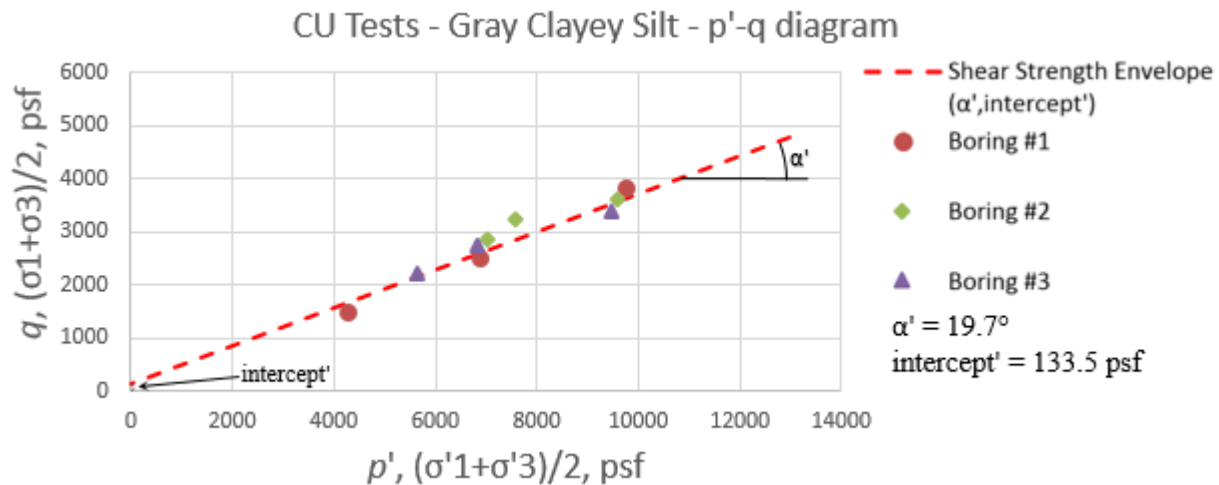
$$S_u = 152 \times 13 = 1976 \text{ psf}$$

Grey Clayey Silt

The Gray Clayey Silt appears to be a critical soil, so this unit was *sampled* in three locations at three different depths (Boring #1 – 31ft, Boring #2 – 35ft, and Boring #3 – 40ft) using a piston sampler. 1D consolidation tests show that the soil is slightly over consolidated with all three *samples* having an over consolidation ratio (OCR) of about 1.3. Two confining pressures used for the CU tests were approximately equal to the effective stress (σ_v) (Boring #1 Sample 1 and Boring #3 Sample 1). Two confining pressures were approximately equal to the preconsolidation stress (σ'_p) as determined by the consolidation tests (Boring #2 Sample 1 and Boring #3 Sample 2). The remaining five *samples* were tested at confining pressures of either 5760 psf (40 psi) (Boring #1 Sample 2 and Boring #2 Sample 2) or 8640 psf (60 psi) (Boring #1 Sample 3, Boring #2 Sample 3 and Boring #3 Sample 3). The resulting CU test data with pore pressure is used to determine the drained and *undrained shear strengths* of this material.

Drained Shear Strengths

The following chart shows the p' – q diagram of the test data with an α' , intercept' shear strength envelope plotted so that 2/3s of the points fall above the line representing the envelope.



$$\phi' = \arcsin(\tan(\alpha')) = \arcsin(\tan(19.7^\circ)) = 21^\circ$$

$$c' = \text{intercept}' / \cos(\phi') = 133.5 / \cos(21^\circ) = 143 \text{ psf}$$

Undrained Shear Strengths

The Gray Clayey Silt at this site was typically found between 28 and 42 feet below the ground surface. The *undrained shear strengths* for this *critical layer* were modeled as preconstruction *undrained shear strength* (S_{uo}) that is the normalized undrained shear strength ratio (S_u/σ'_{vc}) obtained from a CU test run at a confining pressure approximately equal to the effective stress on the *sample* at the time of sampling multiplied by the preconsolidation pressure (σ'_p). CU *sample* Boring #1 Sample 1 (upper) was tested to model the upper portion of the layer and Boring #3 Sample 1 (lower) was tested to model the lower portion of the layer. Using [Equation 8-1](#)

$$S_{uo-upper} = \frac{S_u}{\sigma'_{vc}} \sigma'_p = \frac{1/2 (\sigma_1 - \sigma_3)}{\sigma_3} \sigma'_p = \frac{1/2 (6731 - 3744)}{3744} 4836 = 1929 \text{ psf}$$

$$S_{uo-lower} = \frac{S_u}{\sigma'_{vc}} \sigma'_p = \frac{1/2 (\sigma_1 - \sigma_3)}{\sigma_3} \sigma'_p = \frac{1/2 (9203 - 4752)}{4752} 6240 = 2922 \text{ psf}$$

Where

σ_1 = Major principal stress

σ_3 = Minor principal stress

Therefore

Undrained shear strength for the upper half of this unit is $c = 1929$ psf and a $\phi = 0^\circ$.

Undrained shear strength for the lower half of this unit is $c = 2922$ psf and a $\phi = 0^\circ$.

Sand

The *drained shear strength* of the Sand can be determined by empirical correlations. Using the following chart, the N_{60} of the sand units versus the effective vertical stress on the sand units can be correlated to the effective stress friction angle:

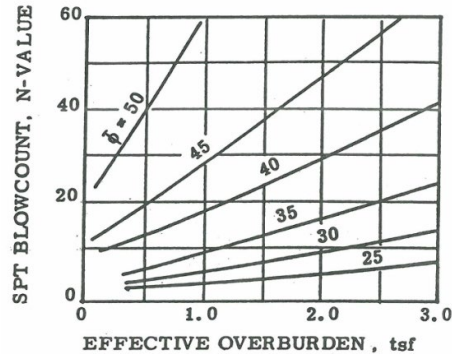


Figure 8-19, N_{60} versus the effective vertical stress correlated to the effective stress friction angle for sand (originally from Schmertmann, 1975, reproduced from Abramson, 1996)

Prior to excavation at this *waste containment facility*, the Sand was covered with a maximum of 45 feet of soil of which 20 feet were below the water table. The Brown CL and the Gray Clayey Silt had in situ wet unit weights of about 120 pcf. The Sand had an in situ wet unit weight of about 127 pcf. Effective vertical stress at the middle of the Sand prior to excavating is calculated as follows:

$$\sigma'_{vo} = 45ft \times 120pcf + \frac{8}{2}ft \times 127pcf - \left(20ft + \frac{8}{2}ft\right) \times 62.4pcf$$

$$\sigma'_{vo} = 4410psf = 2.21tsf$$

Using [Figure 8-19](#) the estimated effective stress at the middle of the Sand layer and the lowest STP N_{60} the shear strength of the sand is determined to be $\phi' = 29^\circ$

	N_{60}	σ'_{vo}	ϕ'
Sand	9	2.21 tsf	29°

Table 8-4, Summary of Material Inputs

Type	Unit weight (pcf)	Drained		Undrained	
		Cohesion or adhesion (psf)	Friction angle (Degrees)	Cohesion (psf)	Friction angle (Degrees)
Liner Soils	120	0	26.9	-	-
Waste	75	313	33	-	-
Drainage Layer	135	0	38	-	-

Type	Unit weight (pcf)	Drained		Undrained	
		Cohesion or adhesion (psf)	Friction angle (Degrees)	Cohesion (psf)	Friction angle (Degrees)
Large Displacement Interface Shear Strength	100	22.8	10	-	-
Peak Interface Shear Strength	100	30.1	14	-	-
Brown Lean Clay	120	0	26.9	1976	0
Gray Clayey Silt Upper	120	143	21	1929	0
Gray Clayey Silt Lower	120	143	21	2922	0
Sand	127	0	29	-	-
Limestone <i>Bedrock</i>	140	5000	0	-	-

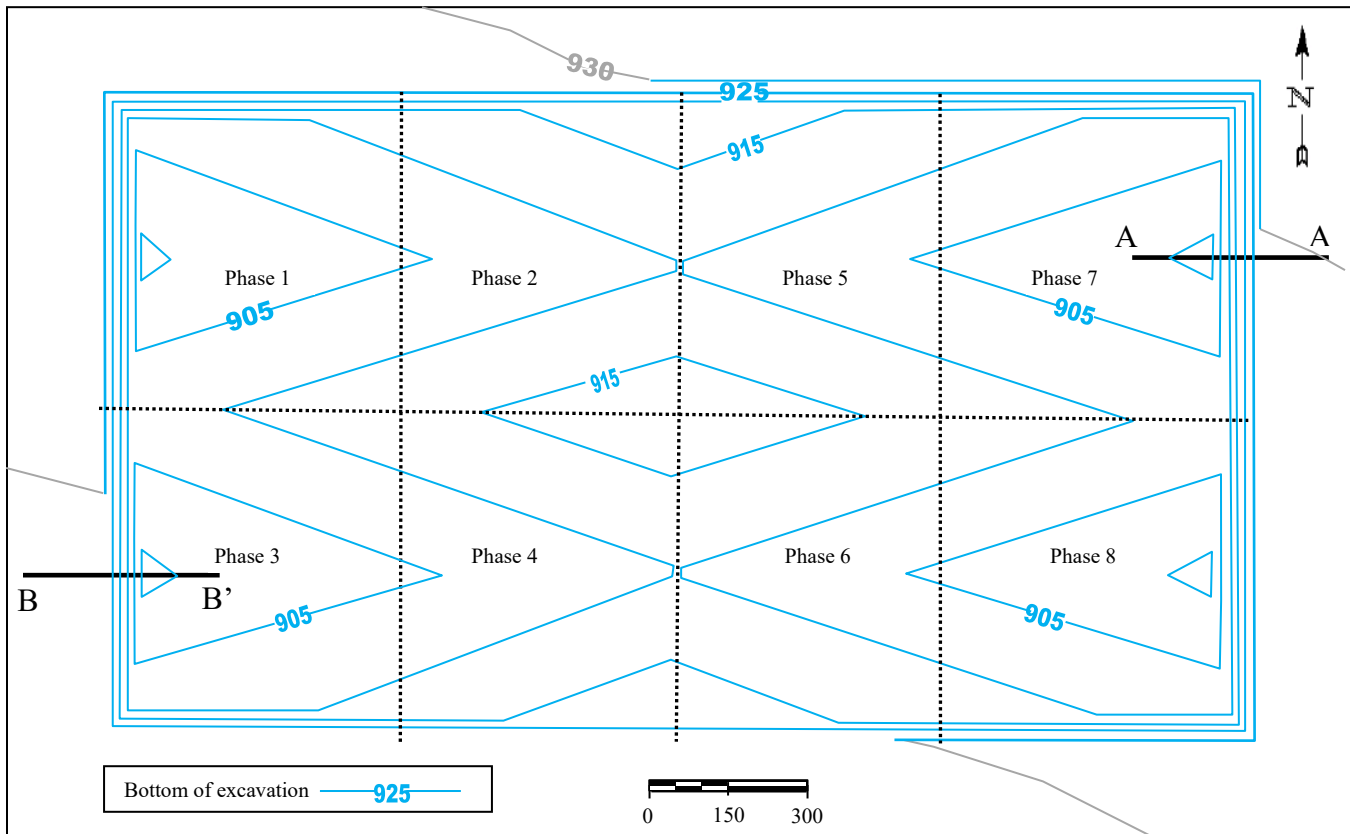


Figure 8-20, Bottom of excavation

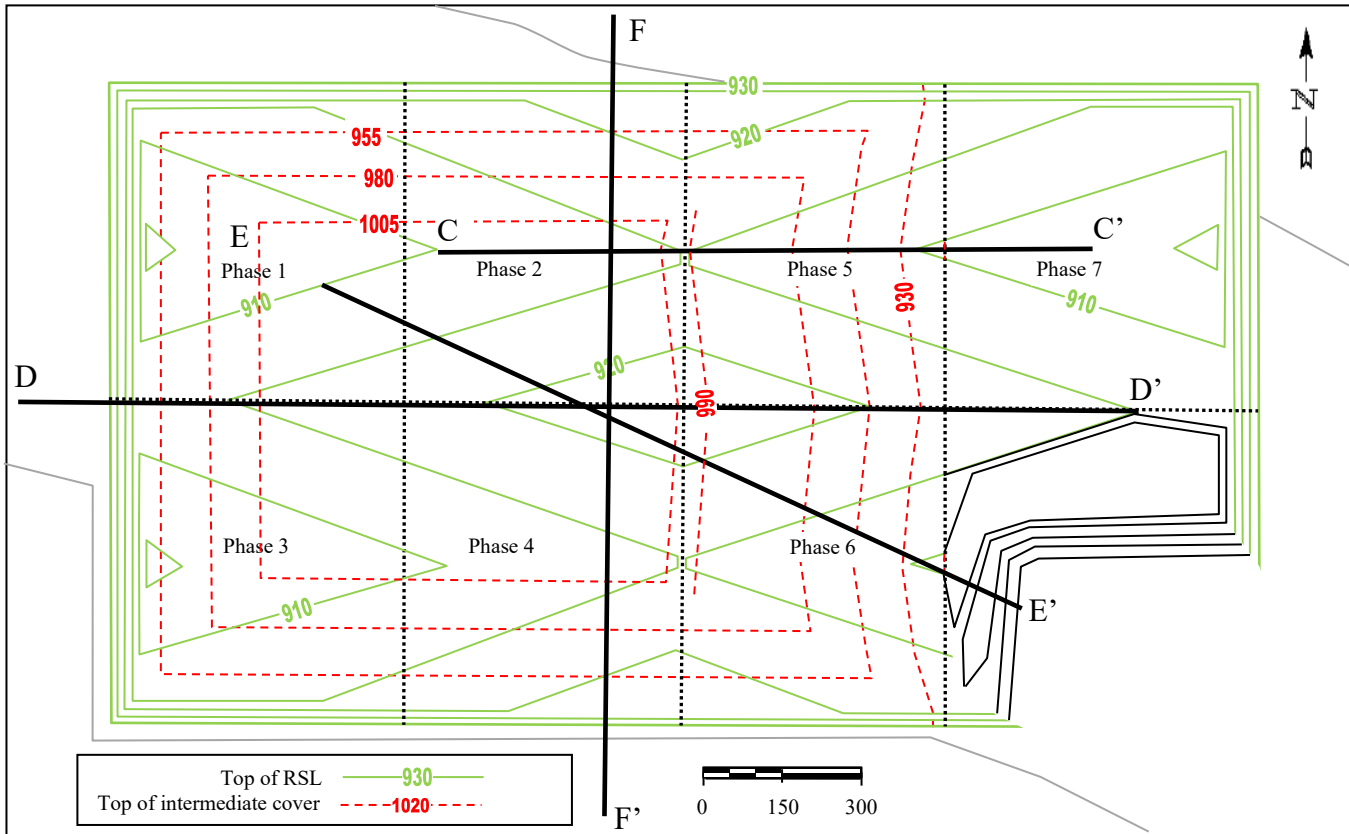


Figure 8-21, Worst-case interim phase development.

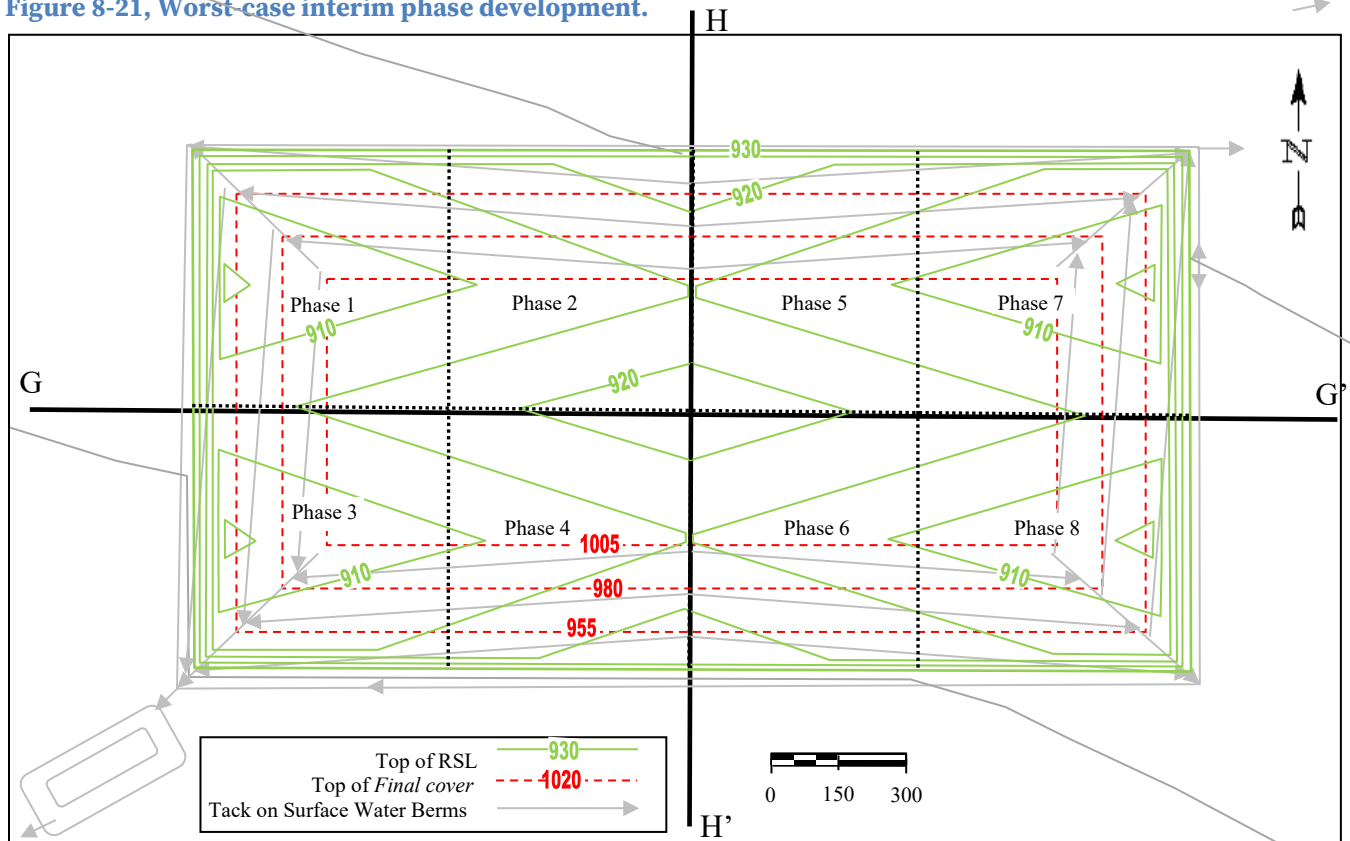


Figure 8-22, Final Slopes.

Table 8-5, End Point Locations of Cross-Sections

Cross-Section	Northing	Easting	Northing'	Easting'
AA'	1,494,406	268,966	1,494,406	268,596
BB'	1,495,006	271,066	1,495,006	270,696
CC'	1,494,406	270,276	1,494,406	269,046
DD'	1,494,706	271,066	1,494,706	268,956
EE'	1,494,476	270,496	1,495,076	269,176
FF'	1,493,966	269,966	1,495,486	269,966
GG'	1,494,706	271,266	1,494,706	268,416
HH'	1,493,816	269,826	1,495,616	269,826

Ohio State Plane North

Table 8-6, Summary of Static Factors of Safety

Failure Type	Cross-Section	Drain Translational	Drained Rotational
Internal	A-A'	N/A	1.51
Internal	B-B'	N/A	1.52
Interim	C-C'	2.05	2.33
Interim	D-D' left to right	2.07	2.50
Interim	D-D' right to left	2.08	2.21
Interim	E-E'	2.08	1.83
Interim	F-F' left to right	2.11	2.62
Interim	F-F' right to left	2.18	2.02
Final	G-G' left to right	2.117	2.68
Final	G-G' right to left	1.98	2.68
Final	H-H' left to right	2.11	2.71
Final	H-H' right to left	2.19	2.61

Notes

- 1) Rotational failure surfaces shallower than 4 ft. were filtered from search to eliminate shallow failures in the *protective layer*.
- 2) All factors of safety presented in this table were determined with Spencer's method performed by the computer program Slide.
- 3) In this example, the "primed" side of the cross-section is the right side of the cross-section.

Table 8-7, Back Calculated Minimum Interface Shear Strength for Approximately a 1.50 Factor of Safety

Type	Drained	
	Adhesion (pcf)	Friction angle (Degrees)
Large Displacement Interface Shear Strength	35.7	6.8
Peak Interface Shear Strength	44.8	7.2

Table 8-8, Back Calculated Factors of Safety

Cross-Section	Drain Translational
C-C'	1.50
G-G' right to left	1.50

Conclusion

This example *waste containment unit* appears to meet the design requirement of having a factor of safety of 1.50 or greater as designed. The *internal slopes* AA' and BB' have a rotational factor of safety just above 1.5. The worst-case *interim slope* is CC' for translational failures and EE' for rotational failures. The worst-case *final slope* is GG' for translational failures and HH' for rotational failures.

Long-Term - Seismic Conditions - Example Calculation

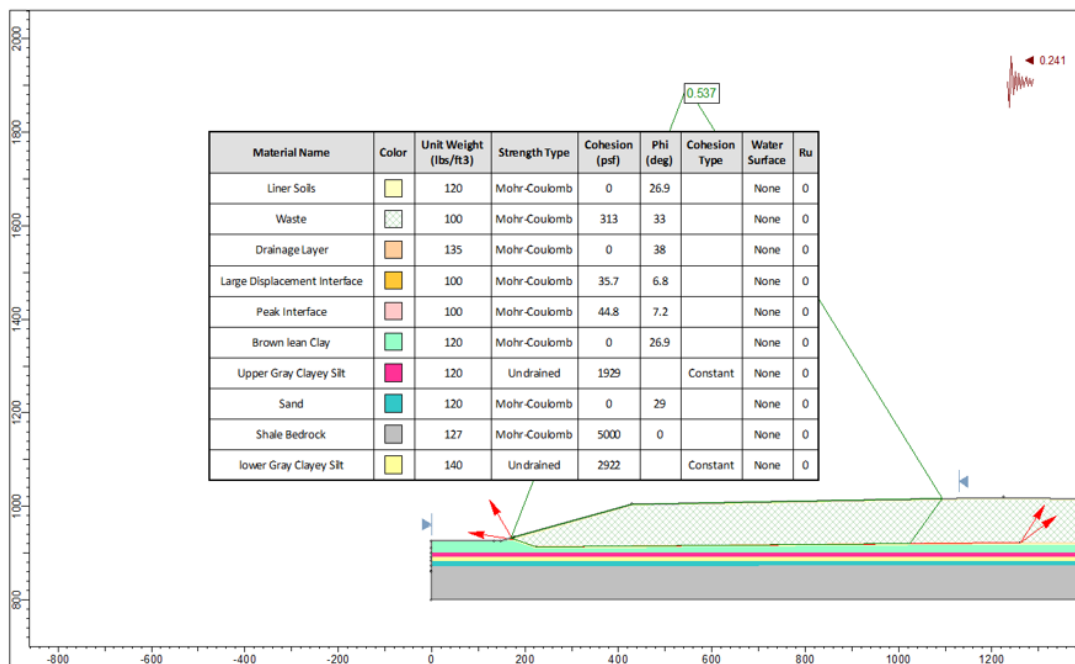
Using the worst-case *final slopes* from the previous static example, perform a seismic evaluation assuming the municipal solid waste landfill is located in southern Allen County, Ohio.

First perform a screening analysis, as using cross-section GG' translational right to left which has the lowest factor of safety and was used to back calculate the proposed interface shear strength of the geosynthetics and the estimated MHA_{Rock} for the *bedrock* beneath the site.

1. Estimate MHA_{Rock} , using site- specific USGS NSHMP's 2023 data for a spectral period of 0.0 seconds i.e. Peak Ground Acceleration based on a probability level of 2% exceedance in 50 years and a V_{s30} of 760 m/s (PGA).

USGS NSHMP's 2023 data indicates a PGA of 0.241 g so $MHA_{Rock} = 0.241$ g

2. Perform a pseudo-static analysis for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* for side slopes and lowest *peak shear strength* for the bottom slopes and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section. It should be noted that in this example the gray clayey silt is *saturated*, so it is being modeled using *undrained shear strengths*, however, when analyzed the worst-case failure surfaces did not pass through this unit. Setting the horizontal seismic load coefficient (k_H) for the base of the *waste containment unit* as $k_H = MHA_{Rock} = 0.241$ g.

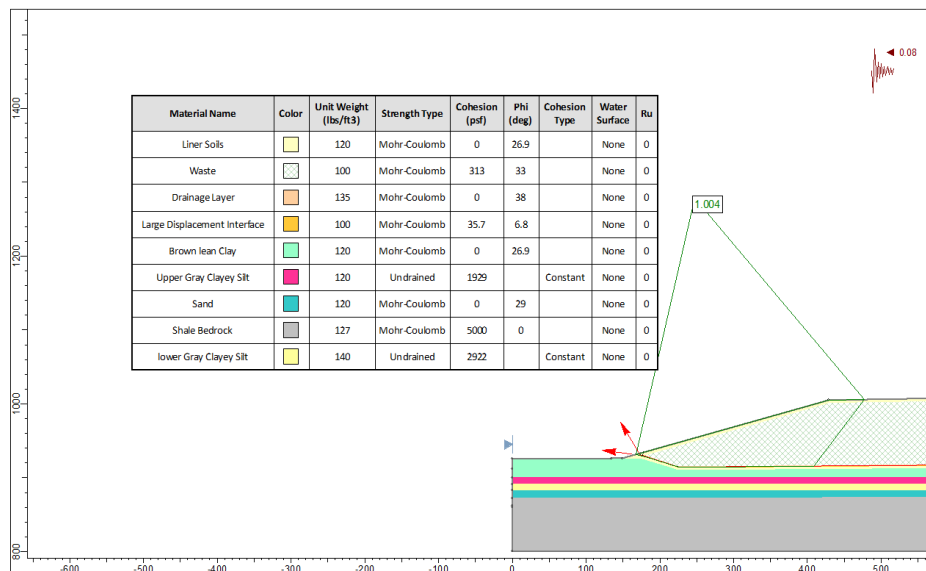


3. The factor of safety for this analysis is 0.537, which is less than 1.0 so a more detailed analysis needs to be performed. The Simplified Procedure outlined in

Bray et al 1998 will be performed and Richardson et al 1995 will be performed. It should be noted that only one of these analysis typically should be performed.

Simplified Procedure outlined in Bray et al 1998

1. Using a limit equilibrium slope stability program which has a method that satisfies full equilibrium, such as Spencer, calculate the yield acceleration coefficient, k_y in the following manner: Perform pseudo-static analyses for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section. While fluctuating the horizontal seismic load coefficient for the base of the *waste containment unit* until a factor of safety of 1.0 is achieved. The horizontal seismic load coefficient where the Factor of safety is equal to 1.0 is considered the yield acceleration coefficient k_y .



$k_y=0.08g$

2. Assign the earthquake magnitude (MHA_{Rock}) and distance using site-specific USGS NSHMP's 2023 data for a spectral period of 0.0 seconds i.e. Peak Ground Acceleration based on a probability level of 2% exceedance in 50 years and a VS30 of 760 m/s (PGA). Available through performing a Disaggregation for the landfill locations currently available at <https://earthquake.usgs.gov/nshmp/>.

USGS NSHMP's 2023 disaggregation indicates a PGA of 0.241 g so $MHA_{Rock} = 0.241g$

USGS NSHMP's 2023 disaggregation indicates a mean (over all sources) r : 26.08 km so use a distance of 26 km

- Estimate the mean period of acceleration-time history for the PGA Earthquake (T_{m-EQ}) based on T_m (s) from either [Figure 8-23](#) or [Figure 8-24](#) and the distance to the likely PGA earthquake epicenter as determine in step 2.

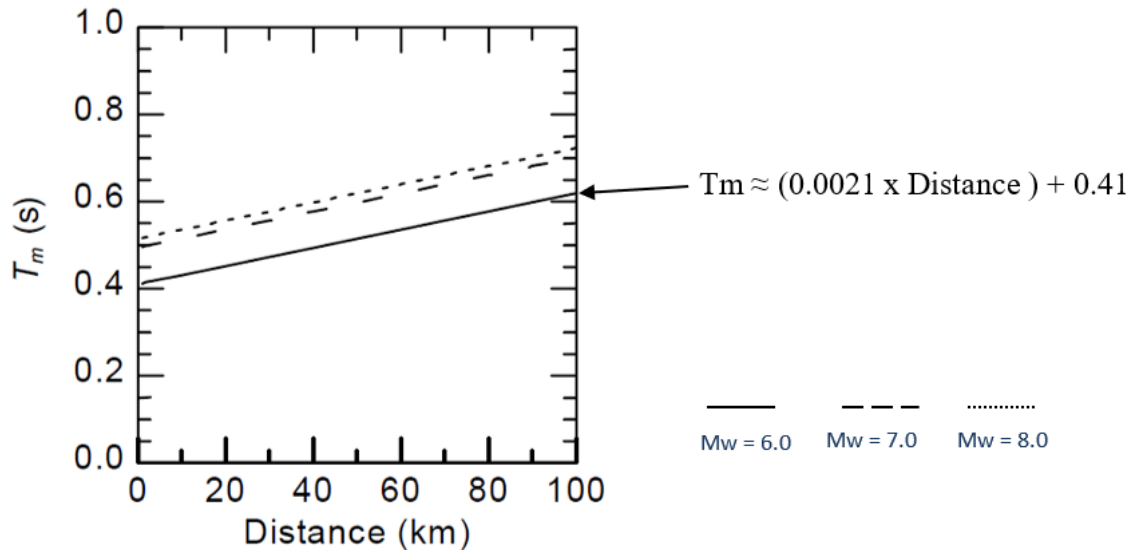


Figure 8-23: Simplified Characterization of Earthquake Rock Motions - Frequency Content, T_m (Bray et al 1998)

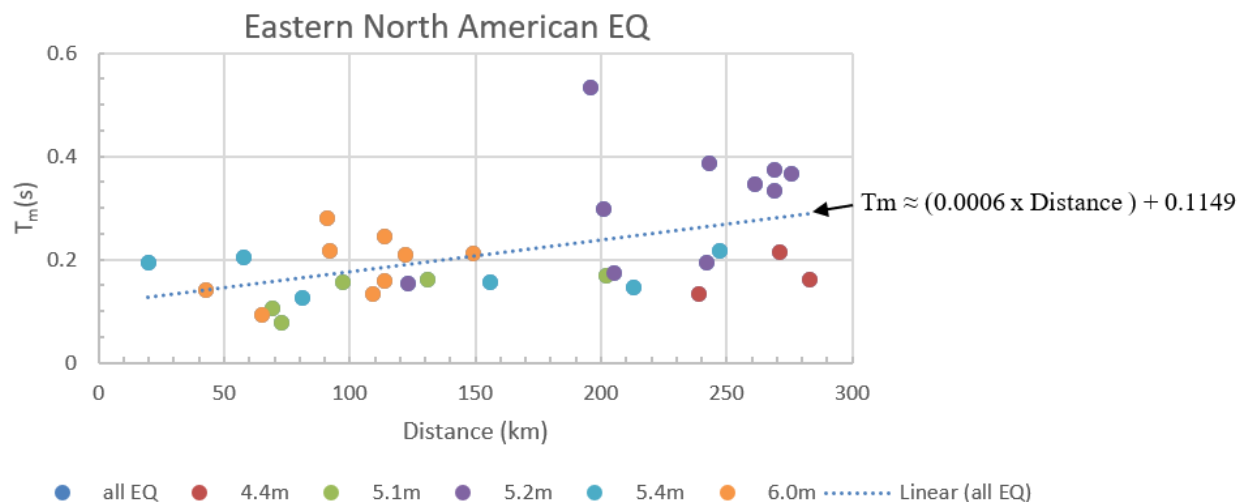


Figure 8-24: Simplified Characterization of Eastern North American Earthquake Rock Motions - Frequency Content, $T_{m-EQ} = 0.465$ s Bray or 0.127 s Eastern North America

4. Estimate the significant duration of acceleration-time history (D_{5-95}) based on either [Figure 8-25](#) or [Figure 8-26](#) and the distance to the likely PGA earthquake epicenter as determine in step 2.

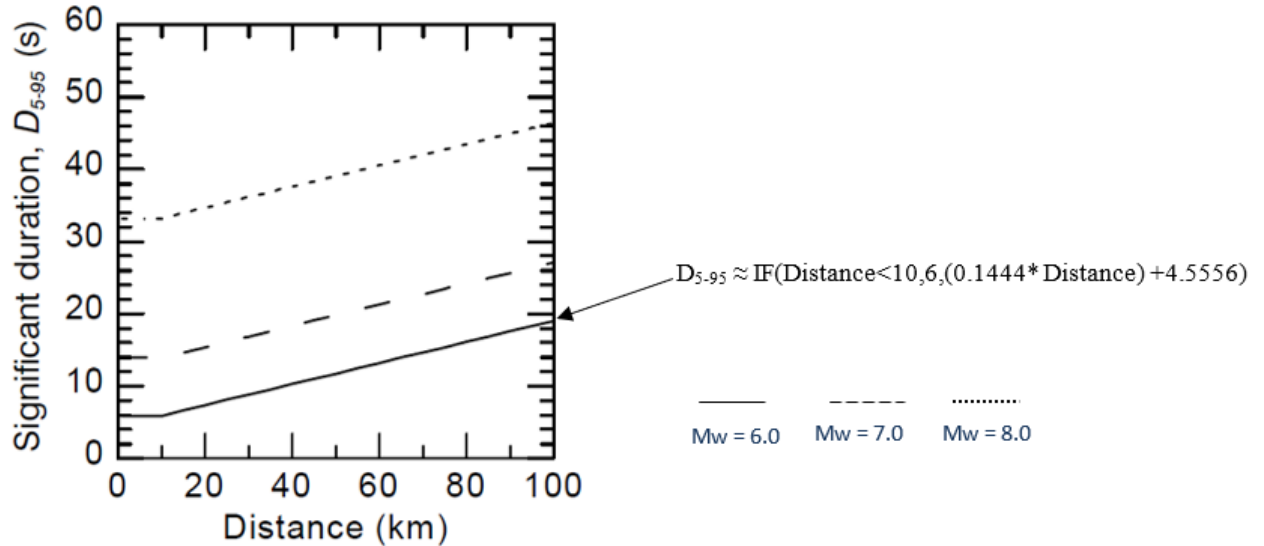


Figure 8-25: Simplified Characterization of Earthquake Rock Motions - Duration D5-95 (Bray et al 1998)

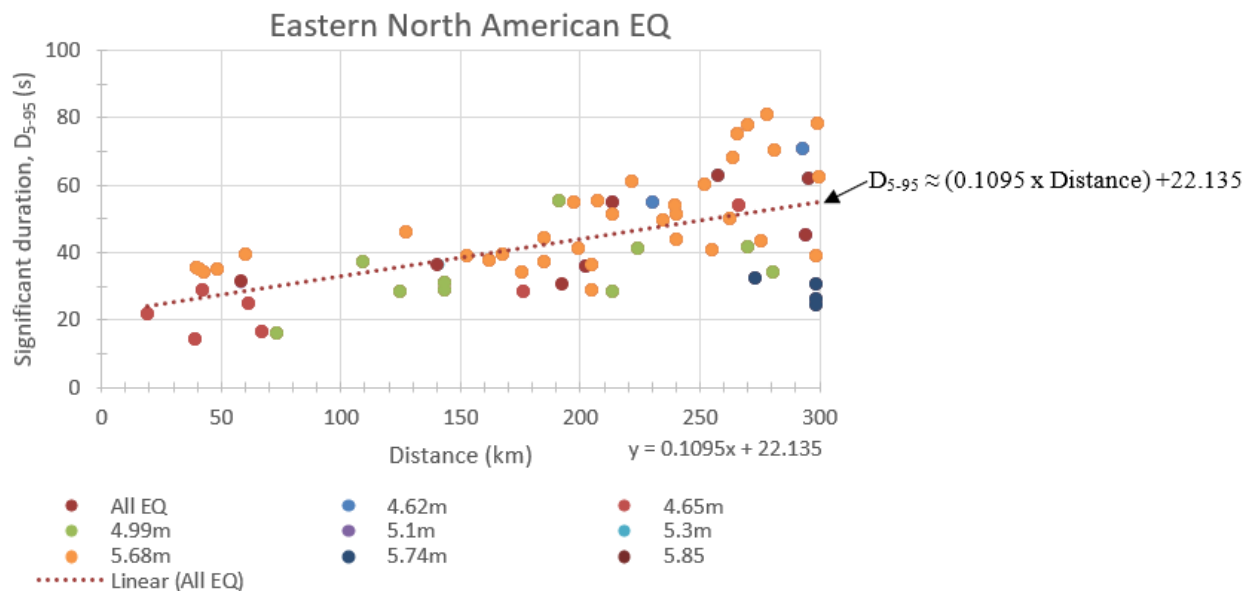


Figure 8-26: Simplified Characterization of Eastern North American Earthquake Rock Motions - Duration D_{5-95} @ 6.0M= 8.3 s Bray or @ 5.5M= 25.0 s Eastern North America

- Calculate the fundamental period of the landfill using the following formula:

Equation 8-8

$$T_{s-waste} = 4H/V_s$$

Where

$T_{s-waste}$ = Fundamental Period (s)

H = Average Waste Height (m) = 113 ft $\approx$ 34.4 m

V_s = Shear Wave Velocity of Waste at Average Waste Height (m/s) using = 251

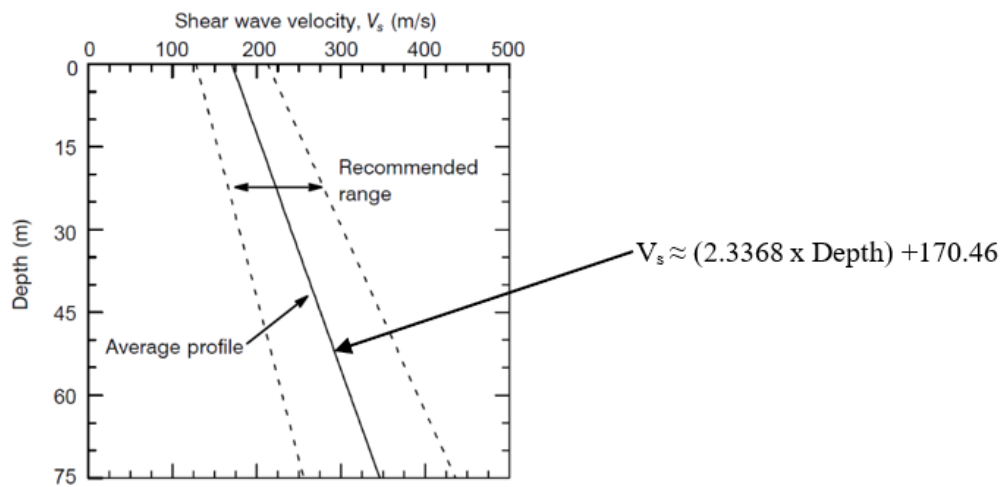


Figure 8-27 = 251

Figure 8-27: Shear wave velocities of municipal solid waste (Bray et al 1998).

$$T_{s-waste} = 4 \times 34.4 / 251 = 0.549s$$

- Calculate $T_{s-waste}/T_{m-EQ}$.

From steps 3 $T_{m-EQ} = 0.465$ s Bray or 0.127 s Eastern North America and from step 5 $T_{s-waste} = 0.457$ s, therefore $T_{s-waste}/T_{m-EQ} = 1.181$ Bray, 4.323 Eastern North America

- Using [Figure 8-28](#) to approximate the $MHEA_{Base}$ considering the 16% probability of exceedance.

$$MHEA_{Base}[(MHA_{Rock}) (NRF)] = 0.623 \text{ Bray}$$

$$MHEA_{Base}[(MHA_{Rock}) (NRF)] = 0.226 \text{ Eastern North America}$$

$$NRF = 1.15 \text{ for } MHA_{Rock} = 0.24g$$

$$MHEA_{Base} = (0.24g)(1.15)(0.52) = 0.173g \text{ Bray}$$

$$MHEA_{Base} = (0.24g)(1.16)(0.22) = 0.063g \text{ Eastern North America}$$

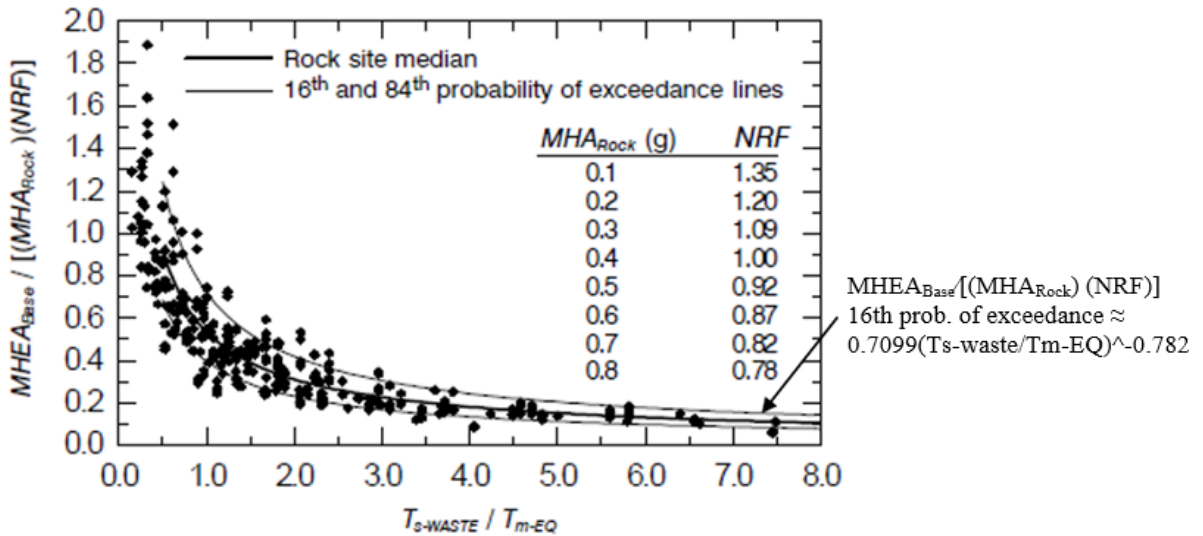


Figure 8-28: Normalized maximum horizontal equivalent acceleration for base sliding versus normalized fundamental period of waste fill (Bray et al, 1998)

- Using Figure 8-29 estimate the seismically induced normalized base liner sliding displacements:

Bray: $k_{max} = MHEA_{base} = 0.173g$, and $k_y = 0.08$, so $k_y k_{max} = 0.462$

Eastern North America: $k_{max} = MHEA_{base} = 0.063g$, and $k_y = 0.08$, so $k_y k_{max} = 1.27$

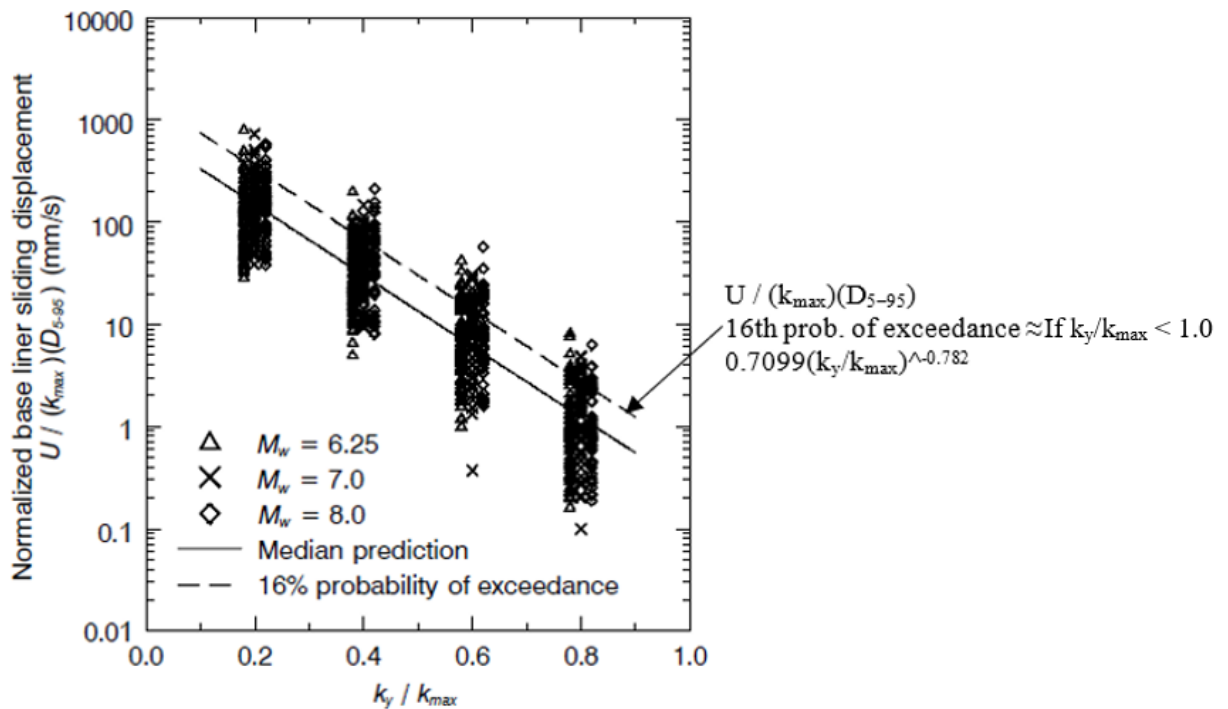


Figure 8-29: Normalized Base Liner Sliding Displacements (Bray et al, 1998)

Bray - $U/(k_{max})(D_{5-95})$ 16th prob= 18.914 mm/s

Eastern North America - $U/(k_{max}) (D5-95)$ 16th prob = 0 mm/s since $k_y/k_{max} > 1.0$

9. Estimate displacement (U) using the following formula

Equation 8-9

$$U = (U/(k_{max}) (D5-95)) \times (k_{max}) \times (D5-95)$$

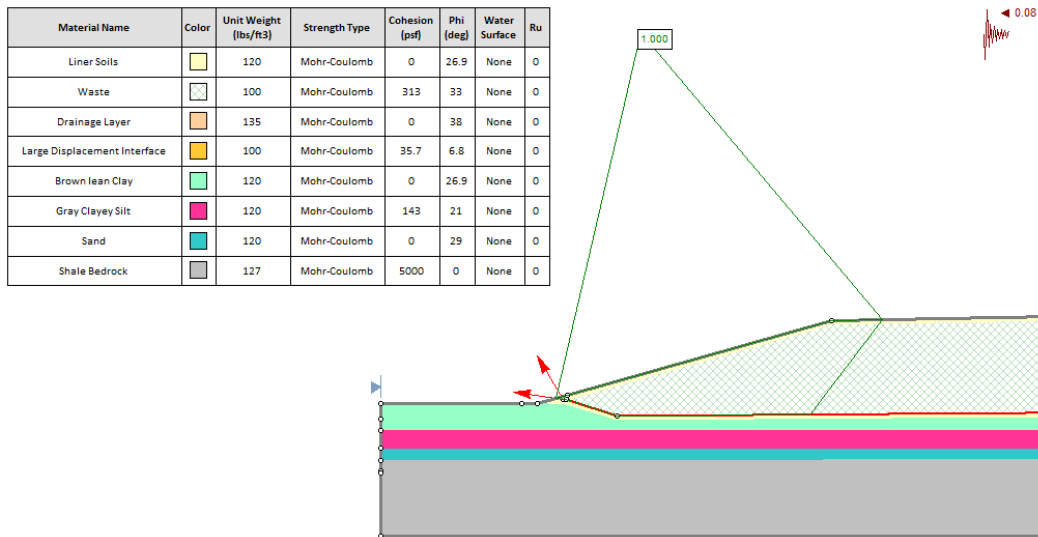
$$U_{16\% \text{ probability of exceedance - Bray}} = (18.914 \text{ mm/s})(0.173g)(8.3 \text{ s}) = 27.16 \text{ mm}$$

Since the deformation based on eastern north American earthquakes indicates no displacement and Bray's set of earthquakes indicates 27.16 mm of deformation are less than 15 cm, the *waste containment facility's* design is acceptable.

Simplified Procedure outlined in Richardson et al 1995

1. Using a limit equilibrium slope stability program which has a method that satisfies full equilibrium, such as Spencer, calculate the yield acceleration coefficient, k_y in the following manner: Perform pseudo-static analyses for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section. While fluctuating the horizontal seismic load coefficient for the base of the *waste containment unit* until a factor of safety of 1.0 is achieved. The horizontal seismic load coefficient where the Factor of safety is equal to 1.0 is considered the yield acceleration coefficient k_y .

Chapter 8 - Deep-Seated Failure Analysis



$$k_y = 0.08g$$

2. Classify the top 100 feet of foundational soils at the site:

- The upper brown lean clay is a stiff soil N_{60s} that ranged from 14 to 20.
- The wet grey clayey silt is medium to stiff N_{60s} that ranged from 6 to 13 with an average of 8.
- The *saturated* sand is loose to medium dense N_{60s} that ranged from 9 to 16 with an average of 11.
- Limestone is a firm *bedrock*.

Since the soils have average N_{60s} of 8 or higher we will classify the soils as medium stiffness for the purpose of this evaluation.

3. Estimate peak outcrop acceleration ($a_{o\max}$) using site-specific USGS NSHMP's 2023 data for a spectral period of 0.0 seconds i.e. Peak Ground Acceleration

based on a probability level of 2% exceedance in 50 years and a V_{s30} of 760 m/s (PGA).

a_{omax} = USGS NSHMP's 2023 Disaggregation data indicates a PGA of 0.241 g

4. Estimate k_{max} for the *waste containment unit* using the recommendations in Richardson, 1995 page 45 step 2.

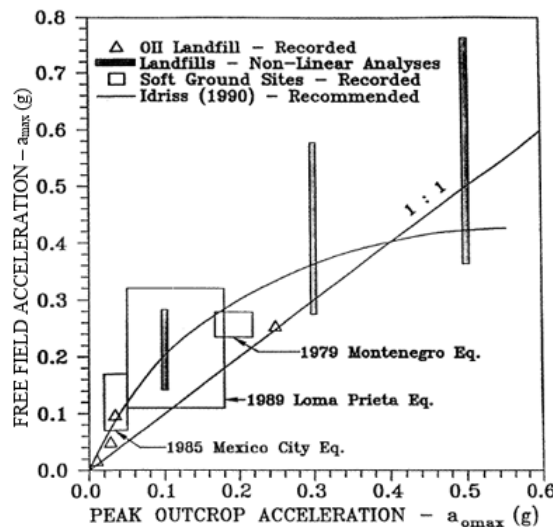


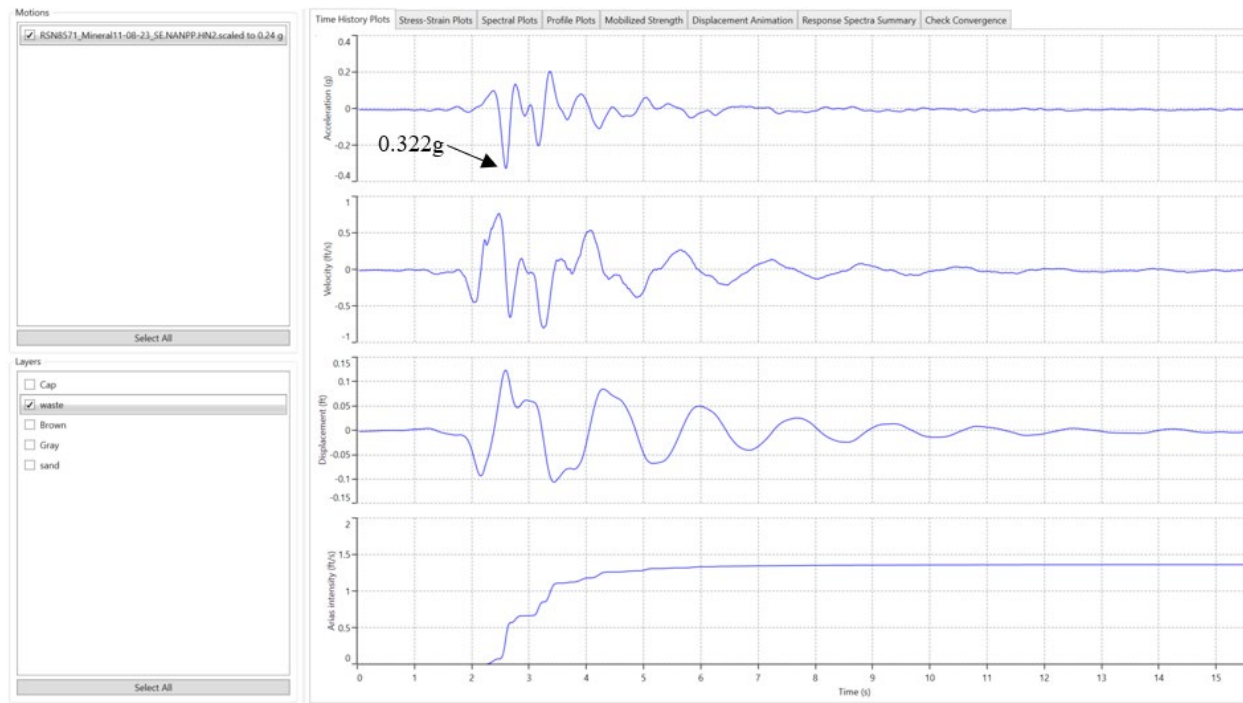
Figure 8-30: Observed Variations of Peak Horizontal Accelerations on Soft Soil and MSW Sites in Comparison to Rock Sites (Kavazanjian and Matasovic, 1994). (Richardson, 1995)

Step 2 in Richardson, 1995 states that “For medium stiff soils, use an acceleration equal to the average of the rock site acceleration and the soft soil site acceleration from Idriss' curve in

[Figure 8-30](#) for peak bedrock accelerations less than or equal to 0.4 g.” The Idriss (1990) soft soil site acceleration curve indicates that the free field acceleration (a_{max}) for a soft soil site would be approximately 0.32 g and the *bedrock* site acceleration is equal to a_{omax} 0.241 g. So, the average of these values is 0.28 g therefore $k_{\text{max}} = 0.28$ g.

Richardson, 1995 also allows for calculating the free field acceleration using a site-specific seismic response analysis. When this was conducted the max free field acceleration for the waste mass was calculated as 0.322g using the site-specific response to the 2011 Mineral Virginia earthquake with data scaled to

0.241 g from North Anna Nuclear Generation Station, Mineral, VA (station SE.NANPP).



5. Calculate $k_y/k_{\max}$

Figure 8-30 analysis $k_y/k_{\max} = 0.08 / 0.28 = 0.286$.

Site-specific seismic response analysis $k_y/k_{\max} = 0.08 / 0.32 = 0.25$.

6. Estimate displacement (U) using Figure 8-31.

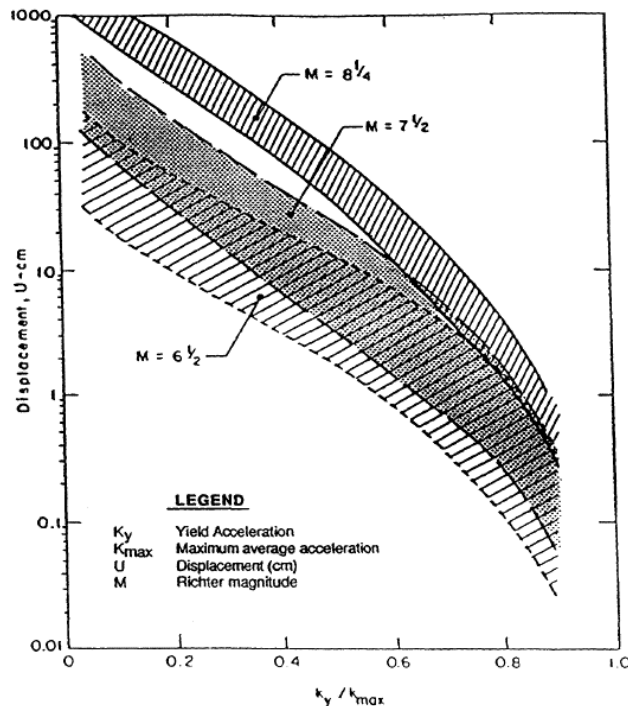


Figure 8-31: Makdisi and Seed Permanent Displacement Chart (Makdisi and Seed, 1978) (Richardson et al, 1995)

Richardson et al, 1995 states to use the upper bound curve appropriate for your region. Since the USGS NSHMP's 2023 Disaggregation data indicates a mean (over all sources) magnitude of 5.54 using the upper bound curve of the $6 \frac{1}{2}$ magnitude, may overestimate displacement for the southern Allen County area. So, we will use the midpoint of the $6 \frac{1}{2}$ magnitude.

Midpoint of the $6 \frac{1}{2}$ magnitude at a $k_y/k_{max} = 0.286 \approx 15$ cm

Midpoint of the $6 \frac{1}{2}$ magnitude at a $k_y/k_{max} = 0.25 \approx 18$ cm

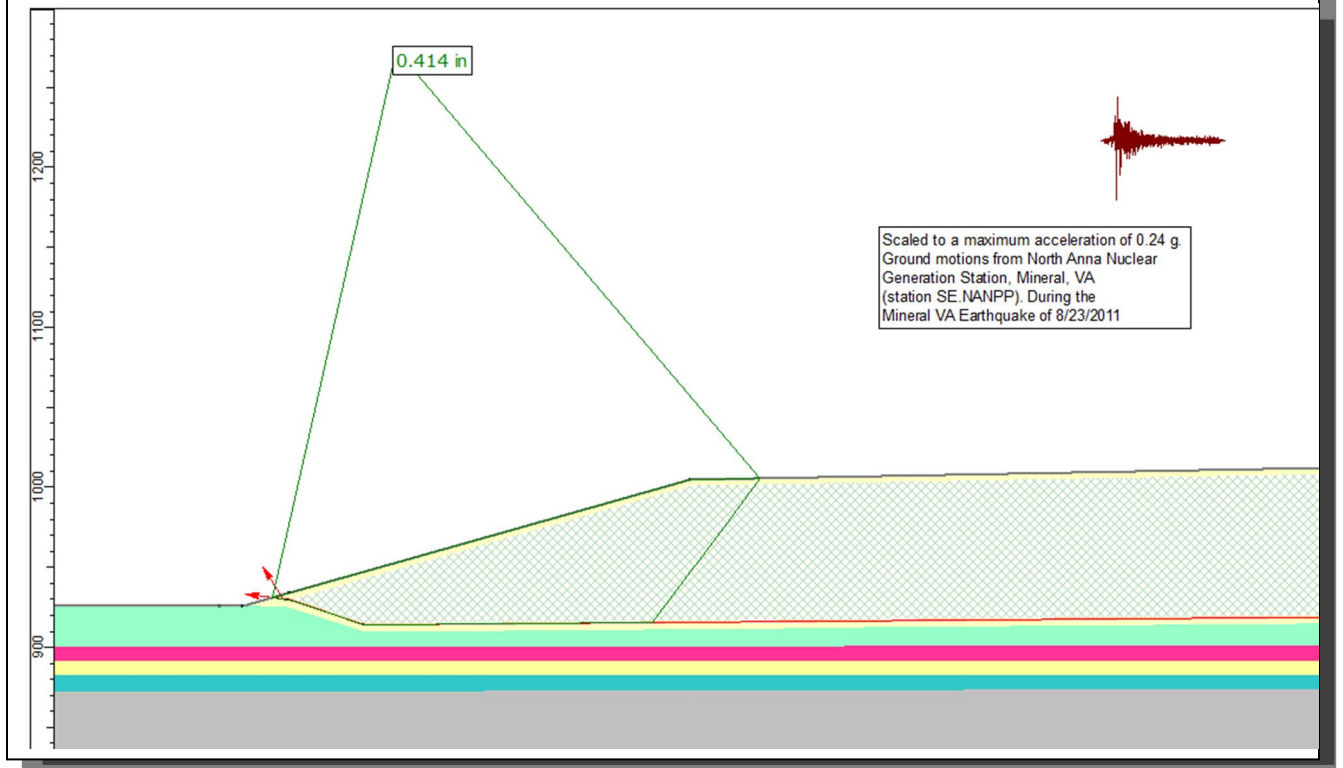
Using Richardson et al, 1995 the predicted deformation is equal to or in excess of 15 cm.

Conclusion

Since Bray's analysis method is more rigorous than Richardson's method, and since Richardson's does not have a curve for a magnitude like our location, we will conclude that Bray's analysis method based on eastern north American earthquakes is the appropriate method and that no displacement will occur. Therefore, the minimum required *large displacement shear strength* needed for an acceptable design ($a = 25.7$ psf,

$\delta = 6.8^\circ$) is equivalent at the interfaces for both seismic and static conditions at this example *waste containment unit*. The quality assurance plan should require that the shear strength of the material interfaces for this example landfill meet the strengths used in these calculations.

Some computer programs can also perform deformation analyses and may also be used. For example, the program Slide 2018 2D Limit Equilibrium Analysis of Slope Stability performs a Newmark displacement analysis. The previous example yields a deformation of 0.414 inches (10.5 mm) which is very similar to Bray's method.



Short-Term - Example Calculation

Short-term deep-seated analyses are used to challenge the in-situ foundation materials under the waste mass to ensure that they provide a static factor of safety of 1.30 under the hypothetical situation of the entire *waste containment unit* being placed all at one time. In this example, only one cross-section is examined. In most cases, however, several *final slope* cross-sections and possibly several *interim slope* cross-sections may need to be examined.

A 100-acre industrial waste landfill is proposed to be located in north central Ohio. The *facility* will be constructed during one construction season and filled in about 31 years. The liner for the *waste containment unit* will be built nearly on grade with only enough material being excavated for construction purposes and to contour the bottom liner for leachate drainage. The *final slopes* will be 3(h):1(v). The liner system comprises 3 feet of recompacted soil liner, a 60-mil textured FML, a geotextile cushion layer, a 1-foot granular drainage layer, and a geotextile filter layer. A few standard penetration tests (SPTs) were performed at the site until it was determined the site is underlain by about 70 feet of soft varved clay with intermittent fully *saturated* sand seams. At that time, the design engineer brought in a Cone Penetrometer Test (CPT) truck to log the changes in strata etc., at a frequency of one CPT per four acres, and a field vane to obtain *undrained shear strengths* of the clay across the site. Numerous piston *samples* were also taken of the clay. The field vane results show that the *undrained shear strength* of the clay increases with depth, but, varies little horizontally across the site.

Assigning Shear Strengths

Waste

Shelby tube *samples* of the waste were taken from three locations at the *waste containment facility* where the waste is currently being disposed. The *samples* from each location were tested using CU tests with pore pressure measurements at three normal loads. The waste has an effective shear strength of $\phi = 35^\circ$ and a cohesion of 250 psf. It was also determined that the waste has a unit weight of 100 pcf.

Soft Varved Clay

The soft varved clay was tested using field vane tests conducted every five feet in five separate *borings* across the site. In five additional *borings*, off set from the field vane by five feet, soil *samples* were taken for Atterberg limits, in situ moistures and odometer tests as well as other laboratory tests.

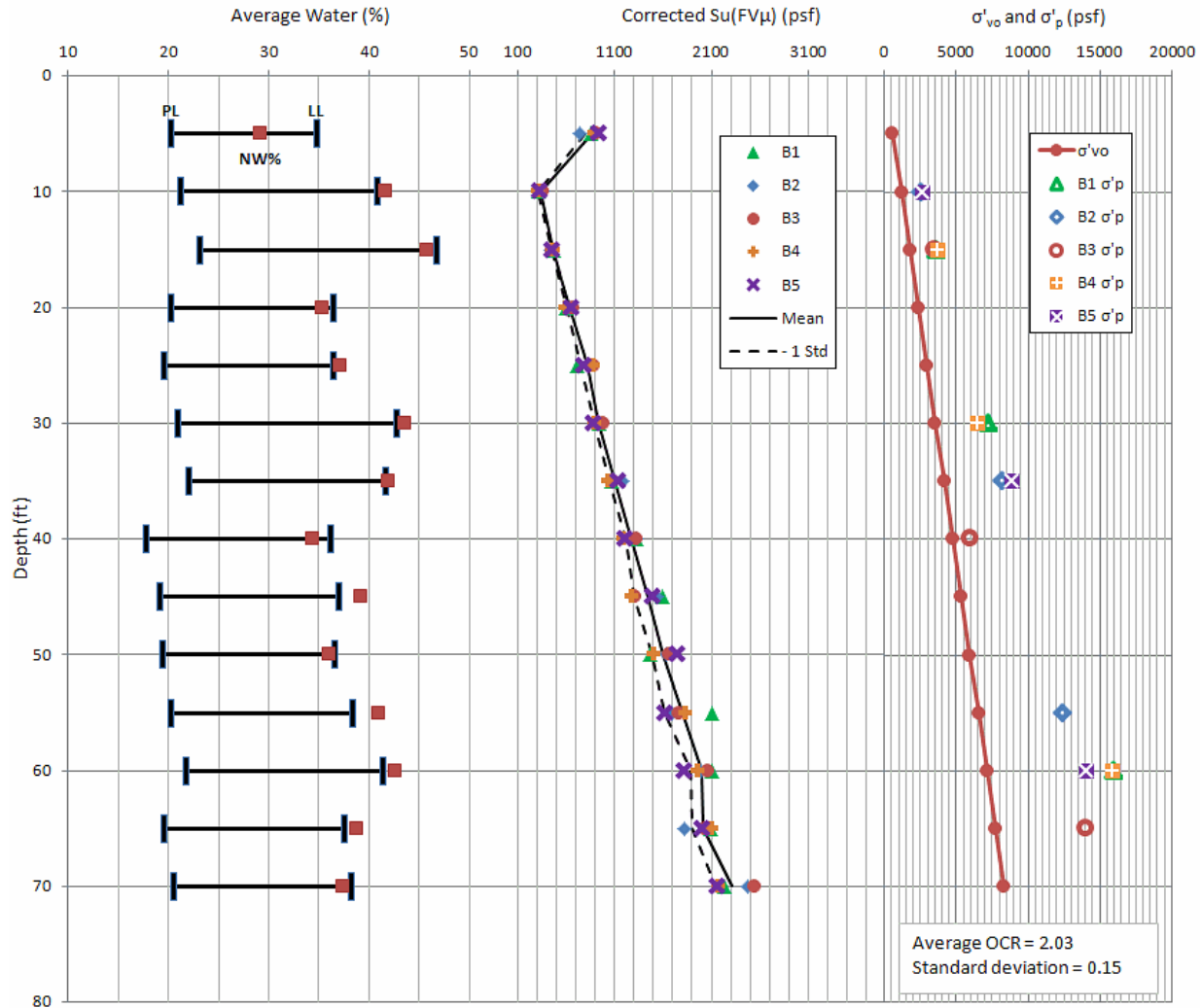


Figure 8-32, Depth vs. Atterberg Limits, Correct $S_u(FV\mu)$ and stress history

As observed from [Figure 8-32](#), a crust of dryer material has formed on the surface of the soft varved clay and the *undrained shear strength* increases with depth. In this example, *undrained shear strengths* from the mean of the $S_u(FV\mu)$ minus one standard deviation are selected and the soft varved clay unit is broken into ten-foot-thick vertical layers (see [Table 8-10](#)). The shear strength from the uppermost portion of the ten-foot-thick layer was inputted into a limit equilibrium stability modeling program and was analyzed using a cross-section of the *waste containment unit* once it was full of waste. This analysis determined that the short-term factor of safety is well below 1.30 (see [Figure 8-33](#)).

Table 8-9, Undrain Shear Strengths of Soft Varved Clay

Depth (ft.)	S _u (FVμ) (psf)
0 to 10	790
10 to 20	305
20 to 30	599
30 to 40	885
40 to 50	1213
50 to 60	1490
60 to 70	1885

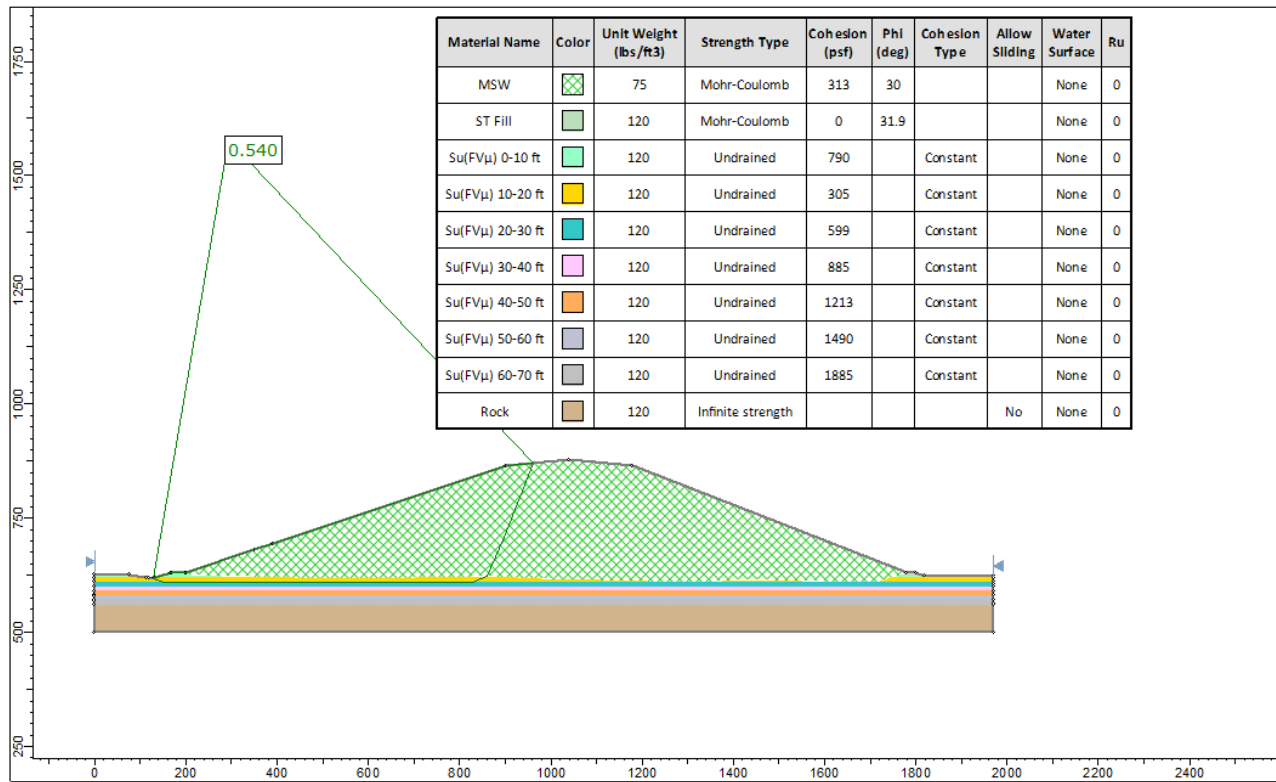


Figure 8-33, Short-term calculation results

Conclusion

Since the resulting factor of safety does not meet the required 1.30, a staged loading calculation will need to be performed. A staged loading calculation example is intentionally not included in this document. Please seek the advice of an expert in soft soils before running a staged loading calculation or just outsource it to them.

REFERENCES

- Abramson, L. W., Lee, T. S., Sharma, S., and Boyce, G. M., 1996, *Slope Stability and Stabilization Methods*. John Wiley and Sons, Inc. New York.
- Benson, C. H., 1998, "Properties of Municipal Solid Waste," *Static and Seismic Slope Stability for Waste Containment Facilities*, April 27 - 29, University of Wisconsin-Madison.
- Blake, T.F., Hollingsworth, R.A., and Stewart, J.P., eds., 2002, *Recommended procedures for implementation of DMG Special Publication 117 guidelines for analyzing and mitigating liquefaction hazards in California*: Los Angeles, Calif., Southern California Earthquake Center, 132 p.
- Bowles, J.E., 1982, "Foundation analysis and design," McGraw Hill.
- Bray, J.D., Augello, A.J., Leonards, G.A., Repetto, P.C. and Byrne, R.J., 1995, "Seismic Stability Procedures for Solid Waste Landfills", *Journal of Geotechnical Engineering*, Vol. 121, No. 2, pp. 139-151.
- Bray, J.D. Rathje, E.M., Augello, A.J. and Merry, S.M., 1998, "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills", *Geosynthetics International*, Vol. 5, Nos. 1-2, pp. 203-235.
- Bray, J. D., and Travarasrou, T. (2007) "Simplified procedure for estimating earthquake-induced deviatoric slope displacements." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 133, No. 4, pp. 381-392.
- Bray, J. D., Zekkos, D., Kavazanjian, E., Jr., Athanasopoulos, G. A., Riemer, M. F. (2009), "Shear strength of Municipal Solid Waste", *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, June 2009, Vol. 135, No. 6, pp. 709-722
- Bjerrum, L. (1973). "Problems of Soil Mechanics and Construction on Soft Clays." *Proc., 8th International Conference Soil Mechanics and Foundation Engineering*, Moscow, 3, 111-159.
- Boussinesq J., (1874), *Annales des Ponts et Chaussées: Mémoires et Documents* 5 Série, Vol. VII, 2nd Semestre, pp169-187.
- Caltrans, 2009, "Geotechnical Services Design Manual, Version 1.0" Caltrans Division of Engineering Services Geotechnical Services.
- Castagna, J. P., Batzle, M. L., and Eastwood, R. L., 1985, "Relationship between compressional-wave and shear wave velocities in clastic silicate rocks", *Geophysics*, 50, 571-581.

DeGroot, D. J., S. E. Poirier and M. M. Landon, 2005, "Sample Disturbance-Soft clays", *Studia Geotechnica et Mechanica* 27(3-4), pg. 91-105.

Duncan, J. M., Wright S. G., 2005, "Soil Strength and Slope Stability", John Wiley & Sons, Inc., Hoboken, New Jersey.

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

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

Gardner, G.H.F., Gardner, L.W., and Gregory, A.R., 1974, "Formation velocity and density – the diagnostic basics for stratigraphic traps", *Geophysics*, 39, 770-780

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

Hansen, M. C., 1995, "GeoFacts No. 8 Landslides in Ohio" Ohio Department of Natural Resourced, Division of Geological Survey.

Hansen, M. C., 2012, "Earthquakes in Ohio" Ohio Department of Natural Resourced, Division of Geological Survey.

Holtz, R. D. and Kovacs, W. D., 1981, *Introduction to Geotechnical Engineering*, Prentice Hall, Inc., Englewood Cliffs, New Jersey, pp. 309 and 390.

Kavazanjian, E., Jr., Matasovic, N. Bonaparte, R. and Schmertmann, G.R. (1995), "Evaluation of MSW Properties for Seismic Analysis," In: *Geoenvironment 2000*, ASCE Geotechnical Special Publication No. 46, Vol. 2, pp. 1126.

Kavazanjian, E., Jr., Matasovic, N., Stokoe, K.H., II, and Bray, J.D. (1996), "In-Situ Shear Wave Velocity of Solid Waste from Surface Wave Measurements," *Proc. 2nd International Congress Environmental Geotechnics*, Osaka, Japan, A.A. Balkema, Vol. 1, pp. 97-104.

Kavazanjian, E., Jr. (2001), "Mechanical Properties of Municipal Solid Waste," *Proc. Sardinia '01 - 8th International Waste Management and Landfill Symposium*, Cagliari, Italy, October, Vol. III, pp. 415-424.

- Koerner, R. M., 1997, "Designing with Geosynthetics," 4th Edition, Prentice Hall Publ. Co., New York. Singh, S., and Sun, J. I., 1995, "Seismic Evaluation of Municipal Solid Waste Landfills," *Geoenvironment 2000*, New Orleans, pp. 1081 - 1096.
- Ladd, C.C., and Foott, R. (1974). "New Design Procedure for Stability of Soft Clays." *Journal of Geotechnical Engineering*, Div., 100(GT7), 763-786.
- Ladd, C.C., 1991. "Stability Evaluation During Staged Construction (22nd Terzaghi Lecture)." *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No 4, pp. 540-615.
- Ladd, C. C. and D. J. DeGroot, 2003, "Recommended practice for soft ground site characterization: Arthur Casagrande Lecture", *Proc. 12th Pan-American Conference on Soil Mechanics and Geotechnical Engineering.*, Massachusetts Institute of Technology, Cambridge, MA.
- Lunne T., Berre T., Strandvik S., 1997 "*Sample Disturbance Effects in Soft Low Plasticity Norwegian Clay*", *Proc. of Conference on Recent Developments in Soil and Pavement Mechanics*, Rio de Janeiro, pg. 81-102.
- Makdisi, F.I., Seed, H.B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," *Journal of Geotechnical Engineering Division*, ASCE Vol. 104, No. GT7, pp. 849-867.
- Manassero, M., Van Impe, W. F., and Bouazza, A., 1997. "Waste disposal and containment." *Proc., 2nd Int. Congress on Environmental Geotechnics*, Vol. 2, Balkema, Rotterdam, The Netherlands, pp. 1425-1474.
- Masada, T., 2009, "Shear strength of Clay and Silt Embankments." *Ohio Research Institute for Transportation and the Environment*, Athens, OH.
- Matasovic, N. and Kavazanjian, E., Jr..1998, "Cyclic Characterization of OII Landfill Solid Waste," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 124, No. 3, pp. 197-210.
- Mesri, G. and Huvaj, N., 2007, "Shear strength mobilized in undrained failure of soft clay and silt deposits," *Proceedings of Sessions of Geo-Denver 2007 Congress: Advances in Measurement and Modeling of Soil Behavior*, Denver, pp. 1-22.
- NAVFAC DM-7.02 (1986). "Foundations and Earth Structures". *Facilities Engineering Command*, U.S. Dept. of the Navy, Alexandria, VA, pg.39
- Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Yuehua, Rezaeian, Sanaz, Harmsen, S.C., Boyd, O.S., Field, Ned, Chen, Rui, Rukstales, K.S., Luco, Nico, Wheeler, R.L., Williams, R.A., and Olsen, A.H., 2014,

Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014-1091, 243 p.

Richardson, G. N., Kavazanjian, E. and Matasovi, N., 1995, EPA/600/R-95/051, RCRA Subtitle D (258) “Seismic Design Guidance for Municipal Solid Waste Landfill Facilities.”

Sahadewa, A., Zekkos, D.P., Lobbestael, A. Woods, R.D., 2011, “Shear Wave Velocity Measurements at Municipal Solid Waste Landfills in Michigan,” Proceedings of the 2011 Pan-Am CGS Geotechnical Conference.

Scalia, J., Bareither, C., and Benson, C., 2019, “Evaluation and Management of High-Moisture Waste Disposal in Municipal Solid Waste Landfills”, Environmental Research & Education Foundation project kickoff evaluation presentation.

Schmertmann, J.H., 1975, “Measurement of the In Situ Shear Strength,” Proceedings: Specialty Conference on In Situ Measurement of Soil Properties, ASCE, Raleigh, NC, State-of-the-Art Report, Vol. 2, pp, 57-138.

Skempton, A. W., 1954, “The Pore Pressure Coefficients, A and B,” *Géotechnique*, 4(4), pp. 143-147.

Spencer, E. 1967. “A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces,” *Géotechnique*, Vol 17, No. 1, pp 11-26.

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

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

Stark, T.D., Martin, J.M., Gerbasi, G.T., Thalhamer, T., Gortner, R.E., 2010, “Aluminum Waste Reaction Indicators in a Municipal Solid Waste Landfill”, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 137, No. 3, March 2012, pp. 252-261

Sowers, G. F., 1979, “Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th ed.”, MacMillan, New York Taylor, D.W., 1948, “Fundamentals of Soil Mechanics,” Wiley, New York.

Terzaghi, K., Peck, R.B., and Mesri, G., 1996, “Soil Mechanics in Engineering Practice – 3rd Edition”. John Wiley and Sons, NY.

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

Turner, A. Keith, and Schuster, Robert L. eds. Landslides—Investigation and mitigation: Transportation Research Board, Special report no. 247, National Research Council, National Academy Press, Washington, D.C.

United States Army Corps of Engineers, 1990, Engineering and Design. Slope Stability. EM 1110-2-1902, Department of the Army, Washington, DC.

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

United States Geological Survey, 2011, National Seismic Hazard Mapping Project, "Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary)."

Zekkos (Zeccos), D. P., 2005. "Evaluation of Static and Dynamic Properties of Municipal Solid-Waste." Ph.D. thesis, Dept. of Civil and Environmental Engineering, Univ. of California, Berkeley, CA.

Zekkos, D. Bray, J. D., Kavazanjian, E. Jr., Matasovic, N., Rathje, E. M., Riemer, M. F., Stokoe, K. H., 2006, "Unit weight of Municipal Solid Waste", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 10, October 2006 pp. 1250-126

Zekkos, D., Bray, J. D., Riemer, M., Kavazanjian, E., and Athanasopoulos, G. A. 2007. "Response of municipal solid-waste from Tri-Cities landfill in triaxial compression." Proc., Sardinia 2007, 11th Int. Waste Management and Landfill Symposium, Paper No. 447.

Zekkos, D., Bray, J. D., and Riemer, M. F., 2008a, "Shear modulus and material damping of municipal solid waste based on large-scale cyclic triaxial testing." Canadian Geotechnical Journal, 45(1), 45-58.

Chapter 9

SHALLOW FAILURE ANALYSIS

Table of Chapter Contents

Introduction	9-1
Reporting	9-2
Factors of Safety	9-4
Assigning Shear Strengths	9-5
Assigning Transmissivities and Permeabilities	9-8
Analysis (Unsaturated , Saturated , Seismic , Equipment Loading , Gas Pressure , Anchoring)	9-11
Appendix 9-A Examples (Shallow Failure , <i>Unsaturated</i> Static Conditions and <i>Peak Shear Strengths</i> - Example Calculation , Shallow Failure , <i>Saturated</i> Static Conditions - Example Calculations , Seismic Conditions - Example Calculation, Equipment loading)	9-21
References	9-46

INTRODUCTION

This chapter provides information to use when analyzing the potential for shallow translational failures of *internal slopes* and *final slopes* (see **Figure f-1**) 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 wastewater 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 loaded with 1,440 psf or less to be a shallow failure.

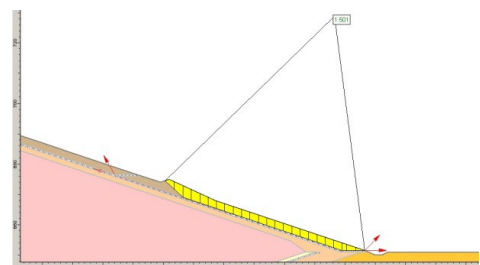


Figure 9-1. Typical translational failure surface.

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 benches, and berms built on top of a cap system that are loaded with less than 1,440 psf above the geomembrane are very rare since roots and soil suction typically provide enough strength to prevent rotational failure in typical cover scenarios. These types of failures are therefore not discussed in this chapter.

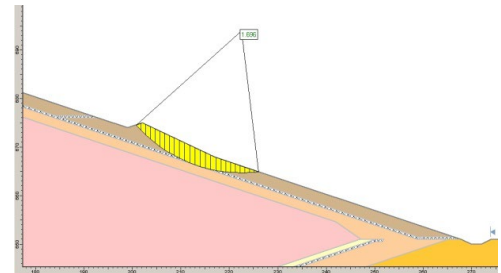


Figure 9-2. Typical rotational failure surface.

REPORTING

This section describes the information that should be submitted to demonstrate that a facility has an acceptable margin of safety with respect to shallow failures. Ohio EPA recommends that the following information be included in its own section of a geotechnical and stability analyses report:

Conclusions

- 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.

Basis of Design

- One or more tables summarizing the input parameters used for the shallow failure analyses including the internal and interface shear strengths of the various components of the internal and *final slopes*, the permeability of the cover soil, and the transmissivity of the geocomposite cap/bottom liner drainage layer or the permeability of the cap/bottom liner drainage layer.
- Plan views of the internal, and *final slope* grading plans, clearly showing the location of each analyzed cross-section and the limits of the *waste containment unit(s)*. Northings and eastings of the cross-section should be indicated on the plan view. These plan views are typically submitted on 11 x 17 paper with a scale of one inch to 300 feet showing the top of drainage layer/ final cover.

- Drawings of the worst-case cross-sections including the slope components (e.g., geosynthetics, cover soil material, drainage layers, compacted soil, waste, drainage structures, temporal high *phreatic* and *piezometric surfaces*). Any drawings or cross-sections referred to in this document 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.

Any figures, drawings, or references relied upon during the analysis should be properly cited. These figures, drawings, or references may be requested separately for review by Ohio EPA.

Calculations

- Stability calculations for *unsaturated internal slopes* and *final slopes* assuming static conditions and *peak shear strengths* i.e. back calculation for a factor of safety of 1.50.
- Stability calculations for *saturated final slopes* assuming static conditions.
 - If the facility has geosynthetics in the final cover calculation, either verify the head in the final cover drainage layer does not exceed the thickness of the drainage layer or calculate the stability of the final cover system when it is fully *saturated*.
- Long-term seismic stability calculations for *final slopes*.
- Any other necessary calculations (some of these calculations are discussed in this document and are listed below).
 - If soils, soil-like waste, or solid waste will be placed directly over geosynthetics on slopes exceeding 3(H) to 1(V) or by equipment exceeding 5.5 psi, then include stability calculations for *internal slopes* and *final slopes* under *unsaturated conditions* using static equipment loads.
 - If surface emissions monitoring indicates that excessive near surface gas production is occurring or the waste will be capped within two years of

waste acceptance in the area of capping, then include stability and sizing calculations for gas pressure relief systems.

- If the designer intends to have the geosynthetics pullout of the anchor instead of ripping, if a failure occurs, then submit geosynthetic anchor pullout calculations.
- Electronic copies of the input files or the spreadsheets used to perform the stability calculations.

FACTORS OF SAFETY

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

Static analysis assuming *unsaturated* conditions: $FS \geq 1.50$

Static analysis assuming *saturated* conditions: $FS \geq 1.10$

Seismic analysis: $FS > 1.00$ or verify that permanent seismic deformations are limited to less than 30 cm.

The factors of safety specified in this document are based on the assumptions contained in this document. 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.

The use of higher factors of safety (e.g. 2.00 and 1.46 respectively) against shallow failures may be warranted whenever:

- A failure would have a catastrophic effect upon human health or the environment.
- Uncertainty exists regarding the accuracy, consistency, or validity of data, and no opportunity exists to conduct additional testing to improve the quality of the data.
- Large uncertainty exists about the effects that changes to the site conditions over time may have on the stability of the facility, and no engineered controls can be implemented that will significantly reduce the uncertainty.

A facility must be designed to prevent shallow failures. If a facility has a 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.

A facility should be designed to prevent excessive permanent seismic deformations. If the calculated seismically induced displacements are estimated as being greater than 30 cm at the composite cap liner interface, then the facility needs to be redesigned.

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

Static Shear Strengths Assuming *Unsaturated* and *Saturated* Conditions

Shear strength values may be estimated from empirical correlations for structural fill, cover soils, drainage layers, recompacted soil components and foundation materials. For interfaces with geosynthetics and for internal shear strengths of GCLs, it is recommended that the shallow failure analysis be used to determine the minimum peak interface shear strengths (and peak internal shear strengths of GCL) that are necessary by back calculating what interface strength will be needed to meet the

required factor of safety. This will provide the maximum flexibility for choosing materials during construction.

Seismic Shear Strengths

The same material and interface shear strength used for the static analysis may be used when performing the seismic screening evaluations. However, if a deformation analysis is necessary, *large displacement interface shear strength* of the geosynthetic with the lowest peak interface shear strength should be used.

Failed Slope Shear Strength

If during the site investigation, slope failures (e.g. landslides) are found within the limits of the facility, Ohio EPA recommends removing the failure mass prior to facility construction. If soils involved in the failure will remain, use the *large displacement shear strength* for the shear strength of the soils. This can be estimated using the back calculated factor of safety of 1.0 for the failure area and verified using the *large displacement shear strengths* determined using the ring shear test or equivalent.

Specifying Shear Strengths in Authorizing Documents

The internal and interface shear strengths of construction materials must be verified before construction (see Conformance Testing in Chapter 4). All geosynthetic interfaces should be tested twice upon initial selection. The material with the lowest *peak shear strength* at each load interval shall be retested prior to each construction event. The compound nonlinear *peak shear strength* envelope developed from these tests shall be greater than or equal to the shear strength envelope determined by back calculating from the required safety factor for interfaces loaded with less than 1,440 psf. The compound nonlinear *large displacement shear strength* envelope developed from the material with the lowest *peak shear strength* at each load interval shall be greater than the shear strength envelope determined as necessary by the seismic back calculations for interfaces loaded with less than 1,440 psf.

Large Displacement shear strengths should be substituted for *peak shear strengths* 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).

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

- During static conditions, the soil portion of a recompacted soil liner (RSL) / geomembrane interface may increase in moisture content due to leachate seeps, migration of groundwater, 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 geomembrane/drainage layer interface.
- At landfills with under designed or broken gas extractions systems, gas can uplift the geomembrane and cause slope failure. Case histories indicate that this is more likely in the portions of landfills that have taken waste with in the last two years.
- During the construction of an internal or *final slope* of a *waste containment facility*, a cover material 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, resulting in a failure. This may also happen when material is placed from the bottom to the top if a piece of equipment is too heavy. Equipment loading calculations may be used to determine risks involved in equipment choice.



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

<http://tstark.net/wp-content/uploads/2012/10/IP91.pdf>

The *responsible party* should ensure that the design and specifications in all authorizing documents and the quality assurance quality control (QA/QC) plans clearly require that the assumptions and specifications used in the 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 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).

ASSIGNING TRANSMISSIVITIES AND PERMEABILITIES

One of the most critical components in performing a *saturated* stability analysis is to estimate the head on top of the geomembrane. To perform this estimate, conservatively assign permeabilities to the materials on the slope of concern. When assigning transmissivities and permeabilities to geocomposite drainage layers, granular drainage layers, and *protective layers*, the following usually apply. (see example page [9-24](#))

- The measured transmissivity (θ_{measured}) of a geocomposite should be determined from *book values* or tested in the laboratory (ASTM D 4716) during the design phase using the expected normal loads, expected boundary conditions and a set time of 100 hours. The θ_{measured} of the geocomposite should be verified at an appropriate frequency before each construction event to assure design assumptions are met.
- The long-term-in-soil permeability of the geocomposite (k_{LTIS}) can be determined with the following formula.

Equation 9-1

$$k_{\text{LTIS}} = \frac{\frac{\theta_{\text{measured}}}{t_{\text{original}}}}{FS \times RF_{CR} \times RF_{IN} \times RF_{PC} \times RF_{CC} \times RF_{BC}}$$

Where

t_{original} = typically 0.00508m or 0.000655m, **original thickness** of the geocomposite core.

FS = 3.0*, **factor of safety**.

RF_{CR} = 1.4 (1.1)* or value determined in 10,000-hour conventional compression creep test (ASTM D7406) or a SIM (Stepped Isothermal Method ASTM D7361) creep test, **reduction factor for creep**. This data is often provided by the manufacturer.

RF_{IN} = 1.2 (1.0)* assuming measured transmissivity was determined using boundary condition similar to the design, **reduction factor for intrusion**.

RF_{PC} = 1.0 (1.0)* assuming that the *protective layer* over the geocomposite meets the specifications discussed in the *protective layer* discussion in the following section, **reduction factor for particulate clogging**.

$RF_{CC} = 1.2 (1.0)^*$, **reduction factor for chemical clogging.**

$RF_{BC} = 1.5 (1.2)^*$, **reduction factor for biological clogging.**

* final cover (short-term *internal slope*) (Giroud et al., 2000a)

- The long-term-in-soil permeability of the granular drainage layer (k_{LTIS}) can be determined with the following formula.

Equation 9-2

$$k_{LTIS} = \frac{k_{measured}}{FS \times RF_{PC} \times RF_{CC} \times RF_{BC}}$$

Where

$FS = 3.0, (1.0)^*$ **factor of safety.**

$RF_{PC} = 1.0 (1.0)^*$ assuming the filter design is appropriate for the *protective layer* in the cap, **reduction factor for particulate clogging.**

$RF_{CC} = 1.2 (1.0)^*$, **reduction factor for chemical clogging.**

$RF_{BC} = 1.5 (1.2)^*$, **reduction factor for biological clogging.**

* final cover (short-term *internal slope*) (Giroud et al., 2000a)

- If the construction specifications limit the soil classification of the *protective layer* to a low plasticity clay (CL) in the "Unified Soil Classification System" described in ASTM D2487. The permeability of the *protective layer* ($k_{measured}$) in the cap system can be considered 1×10^{-5} cm/sec. If the construction specifications provide for the use of other soil classifications, the permeability of the *protective layer* should be conservatively estimated and a specification for a minimum permeability based on these calculations should be added for *protective layer* construction specifications.
- When selecting or specifying a type of material for use as a *protective layer* for a cap system, choose a soil that:
 - Does not easily desiccate; typically soils with a plastic index (PI) below 25, an organic content of 5 % or less, and a maximum silt content of 65 %." (LA DOTD, 2000).

- Will not release particles that will clog the drainage layer; typically soils with a PI above 15 do not have mobile fines and a soil with a PI above five may not have mobile fines but should also be tested to have a double - hydrometer ratio less than 0.5 as determined by ASTM D-4221. Alternatively, the cover soil and geocomposite could be tested with the Hydraulic Conductivity Ratio (HCR) Test (ASTM D5567).
- Has sufficient fertility in the uppermost portion to support vegetation. The permeability of the *protective layer* over a geocomposite on an *internal slope* should be at least one order of magnitude greater than that of the waste material being disposed of and be composed of material that will not clog the geocomposite drainage layer. The following chart adopted from MIRAFI's *Geotextile Filter Design, Application, and Product Selection Guide* can be used to determine the allowable apparent opening size for the geotextile portion of the geocomposite in most cases. Note that the cover soil with a permeability less than 5×10^{-2} cm/sec and the geocomposite could be tested with the Hydraulic Conductivity Ratio (HCR) Test (ASTM D5567), whereas the cover soil with a permeability greater than 5×10^{-2} cm/sec should be tested with the gradient ratio test (ASTM D5101).

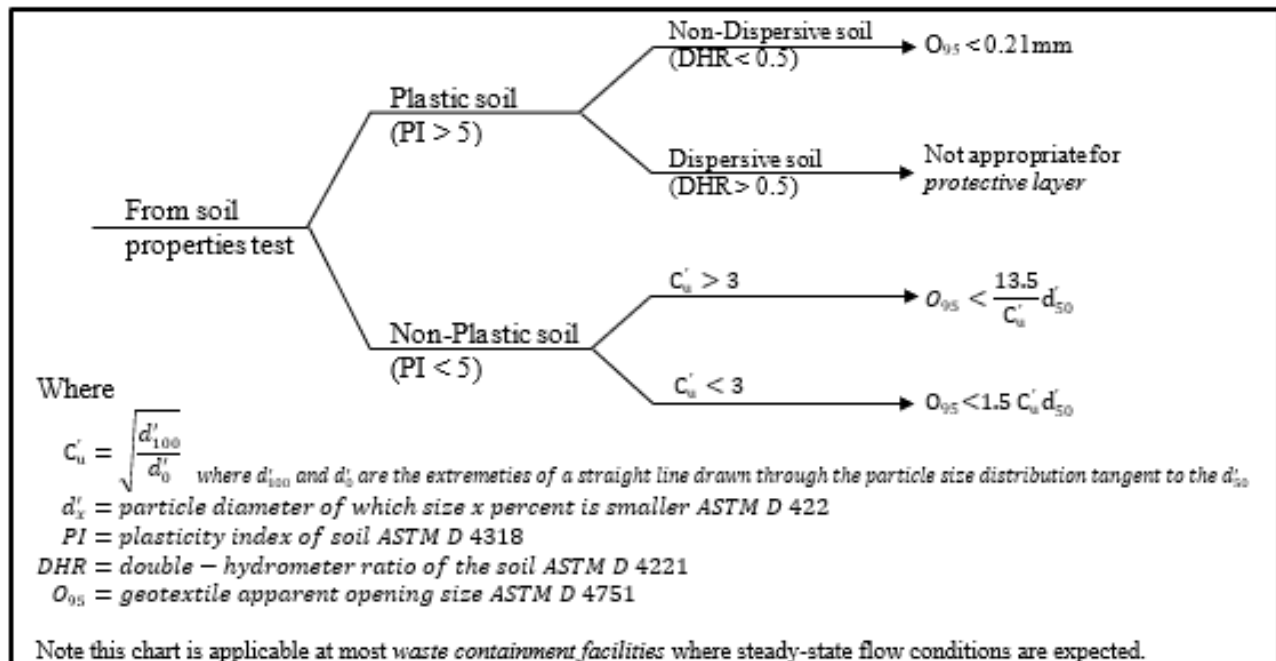


Figure 9-4 Apparent Opening Size Determination Chart

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 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., a fly ash impoundment) and all *final slopes* need to remain stable indefinitely. *Interim slopes* consisting of only soil are generally considered stable for shallow conditions up to a slope of 2.5 to 1 and do not need to be analyzed for shallow stability as long as the design engineer believes the soil used for the slope is stable at the slope being constructed.

There are many different methods to determine the factors of safety when performing a shallow stability analyses. Ohio EPA typically expects a two-dimensional limit equilibrium method such as those presented in the various papers in the reference section written by authors such as Bray Giroud, Soong, and Spenser.

Any method submitted for Ohio EPA review will be reviewed and may be commented on, however, the methods presented in the example section of this chapter have been reviewed as part of preparing this document and are considered acceptable methods for determining factors of safety, head on liners etc.

Static Unsaturated Analysis (see [example](#) in [Appendix 9-A](#))

A translational failure analysis of an *internal slope* or *final slope* should be performed on the longest parallel slopes and the steepest parallel slopes on top of a cap system or bottom liner under *unsaturated* conditions. These analyses should determine the minimum shear strength of the various components of the cap system and the bottom liner to produce an acceptable static *unsaturated* factor of safety i.e. 1.50. If the minimum shear strengths determined by the static *unsaturated* analysis are the lowest of those determined by the shallow failure analyses, then these shear strengths should be incorporated into the specifications of the facility.

Static Saturated Analysis (see [example](#) in [Appendix 9-A](#))

A translational failure analysis of a *final slope* should be performed on the slope with the longest drainage path and on the slope that is greater than 5% with the deepest head on top of liner. Generally *final slopes* with geosynthetics that are stable under dry

conditions are stable during rain events unless the head in the drainage layer exceeds the thickness of the drainage layer. This calculation should be included as part of the static *saturated* stability analysis if the facility has a final cover drainage layer. If the maximum head in the cap drainage layer exceeds the thickness of the cap drainage layer, the water will back up in the drainage layer and either begin flowing through the *protective layer* soil as in the case of highly permeable *protective layer* soil, or cause hydrostatic uplift pressure on low permeable *protective layer* soil, or some combination of the two. This can lead to stability issues like slope failures and/or sand boils. Therefore, design engineers should determine and specify the maximum permeability of the *protective layer* soil, the minimum permeability of the drainage layer and the maximum length between outlet structures for the cap drainage layer required to keep the head in the cap drainage layer less than the thickness of the cap drainage layer.

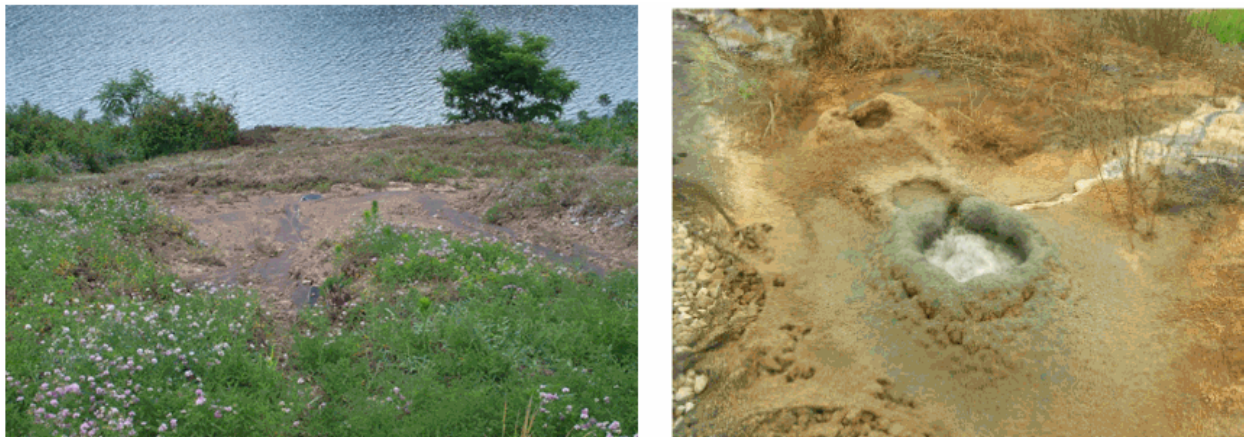


Figure 9-5, Excessive head in final cover drainage layer caused a slope failure (on left) and a sand boil (on right) taken at two different landfills.

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: If the granular drainage layer on the *internal slope* is eroded or suffers a slope failure during a large storm event, the *internal slope* must be repaired as soon as practical to prevent damage to underlying materials through UV degradation, freeze-thaw etc.

- *Saturated Final Slopes*
 - Use the steepest slope and the longest slope length for soil-only final covers or the longest length between outlets for the final covers with a drainage layer and on the slope that is greater than 5% with the deepest

head on top of liner (note; this length may include portions of the flat upper deck of the facility). Assume the final cover is fully *saturated* for soil-only final covers and for final covers with drainage layers where the calculated maximum head in the drainage layer exceeded the thickness of the drainage layer. If calculations show that the calculated maximum head in the final cover drainage layer does not exceed the thickness of the drainage layer, then either the calculated maximum head or the calculated average head may be used in the *saturated* stability calculations. Head calculations should assume that the liquid supply rate to the drainage layer is one of the following:

- 1×10^{-5} cm/sec if the *protective layer* soil will be classified as a low plasticity clay (CL) in the "Unified Soil Classification System" described in ASTM D2487.
- The permeability specified as the minimum permeability in the construction specification for *protective layer*.
- Use the steepest slope and the longest slope length for *internal slopes* with *protective layer* soils over geocomposites. Head calculations should assume that the liquid supply rate to the geocomposite drainage layer is the permeability of the *protective layer* which may be estimated for design purposes as long as a permeability requirement is provided in the specifications for the *protective layer* over the geocomposite.

These static *saturated* analyses should determine if the necessary interface shear strengths of the various components of the cap system meet the recommended factor of safety using the shear strengths determined by the static *unsaturated* calculations. If the factor of safety is not met, the static *saturated* calculations should determine the minimum shear strength of the various components of the cap system needed to produce an acceptable static *saturated* factor of safety. If the minimum shear strengths determined by the static *saturated* analyses are the lowest of those determined by the shallow failure analyses, then these shear strengths should be incorporated into the specifications of the facility.

Seismic evaluation (see example on page [9-29](#))

The United States Geologic Survey (USGS) 2023 version of the National Seismic Hazard Mapping Program (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Peak Ground Acceleration (PGA) based on a probability level of 2% exceedance in 50 years

and a V_{s30} of 760 m/s predicts that over 90% of Ohio has a Maximum Horizontal Acceleration in Rock (MHA_{Rock}) less than 0.10 g. Thus damage to a waste facility due to an earthquake is unlikely in Ohio. Therefore most sites will only need to perform a screening analysis. Most sites that do not pass this screening analysis will need to perform a method similar to the Simplified Procedure outlined in either Bray et al. in *Simplified Seismic design Procedure for Geosynthetic-Lined, Solid-Waste landfills* as published by Geosynthetics International in 1998 or Richardson et al in *RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities* 1995. Sites built on soft clay or materials subject to liquefaction (Uniform Building Code Site Category F) (see Chapter 5 for more information on Liquefaction) should not use the screening analysis or the simplified procedure. These sites should perform a site-specific probabilistic determination of the horizontal acceleration using a one-dimensional site response program followed by a pseudo static slope stability analysis. This evaluation should either be checked by or performed by someone with significant experience in doing this type of analysis.

Ohio earthquakes occur at depths of about 3 to 6 miles along faults in Precambrian rock. “Very few faults in Ohio are visible at the surface, and no surface faults in Ohio are known to be associated with historic earthquakes. Furthermore, none of these faults exhibit evidence of movement during recent (Holocene) time; most of them probably have not been active since the Paleozoic. In fact, fault scarps generated by historic earthquakes have not been clearly demonstrated in the eastern United States. Most faults known in the sub surface of Ohio appear to originate in Precambrian rocks and die out upward in the overlying Paleozoic rocks. Therefore, most of them do not reach the surface where they can be directly mapped and studied. In addition, the *bedrock* surface over about two-thirds of the state is masked by a thick cover of glacial sediments.” (Hansen 2012) It should be assumed that an unknown fault exists near any waste facility in Ohio.

Screening Analysis

1. Determine the peak ground acceleration (PGA) for the location where the facility is located using site-specific USGS NSHMP's data for a spectral period of 0.0 seconds i.e. the peak ground acceleration based on a probability level of 2% exceedance in 50 years and a V_{s30} of 760 m/s. (Link as of January 2025 <https://earthquake.usgs.gov/nshmp/>)

2. Perform pseudo-static analyses for the *final slope* cross-sections that have low static factors of safety. Modeling geosynthetic interfaces using *peak shear strengths* and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section of this chapter. Setting the horizontal seismic load coefficient (k_H) for the base of the facility as $k_H = \text{PGA}$.
3. If the factors of safety determined by these analyses are ³ 1.0 the calculated seismically induced displacements will generally be less than 0 cm for the *final slope* of the facility, and the seismic stability analysis can stop here.

If the factors of safety are < 1.0 designers should perform a method similar to the Simplified Procedure outlined in Bray et al 1998 or Richardson et al 1995. If the Simplified Procedure shows the calculated seismically induced displacement is more than 30 cm the facility should be redesigned.

OTHER CALCULATIONS WHICH MAY BE NECESSARY

Equipment Loading (see [example](#) in [Appendix 9-A](#))

Stability calculations for *internal slopes* and *final slopes* under *unsaturated conditions* shall assume static equipment loads if soils, soil-like waste, or solid waste will be placed directly over geosynthetics on slopes exceeding 3(H) to 1(V) or by equipment exceeding 5.5 psi.

More than seven slope failures have occurred in Ohio after operators have placed materials over geosynthetics (one of these failures occurred within the recompacted clay layer). Most of these failures involved either equipment with a ground pressure exceeding 5.5 psi or the pushing of large amounts of material at one time. Therefore, Ohio EPA recommends only allowing wide tracked low ground pressure equipment (5.5 psi or lighter) be used to deploy material over geosynthetics and that operators be instructed not to push excessive amounts of material at one time. To avoid the cost of damage investigation and reconstruction, if equipment with a ground pressure of 5.5 psi or above will be used, or the slope is greater than 3(H) to 1(V), equipment loading should be calculated. The slope movements associated with equipment loads vary from minor displacement of a few inches to large failures spanning the length of the slope. Although small movements may appear to be minor displacements, the

movement can result in the reduction in liner interfaces shear strengths and possible hidden damage to liner components.

The shear strengths of the cover soil and the granular drainage layer should be conservatively assigned for design purposes and the cohesion of these materials should be zero since they tend to be loose and under low loads.

Ideally a factor of safety of greater than 1.00 would be maintained during material placement. Contractors and owners should be made aware by the design and or construction engineers of the risk of slope failure while placing material over geosynthetics, however since a failure during construction poses little environmental risk, a factor of safety lower than 1.00 may be acceptable if all parties agree to the risks and liabilities. If the contractors and owners do decide to construct using equipment and interfaces that would result in a factor of safety lower than 1.00, Ohio EPA advises adopting a *large displacement shear strength* specification for the cap system component or the bottom liner component with the lowest *peak shear strength* based on a static *unsaturated* factor of safety of 1.25.

Note that a factor of safety greater than 1.00 does not guarantee stability since material interface strengths and actual loading in the field may vary from those determined in testing and used in calculations.

Analyzing designs for potential slope failures due to equipment loading can be performed using 2D limit equilibrium slopes stability programs. The load due to the equipment should be modeled using a distributed load (a.k.a. a surcharge load) on the cross-sectional surface representing the weight of the equipment distributed across its contact with the surface (see examples starting on page [9-41](#)).



Figure 9-6, Stable slope of 1 ft of pea gravel on 3 to 1 slope after being placed with a low ground pressure Caterpillar D6 and IHI crawler carrier (5.8 psi). The calculated factor of safety for equipment loading conditions was 1.041 (interface $\approx a=70$ psf and $\delta=18.3^\circ$, drainage layer $\approx c=0$ psf and $\phi=37^\circ$).



Figure 9-7, Failed slope at an Ohio Landfill between textured geomembrane and GCL covered with 1 ft. of pea gravel on 3 to 1 slope after being placed with a Caterpillar D8 (14.7 psi). The calculated factor of safety for equipment loading conditions was 0.827 ($\approx a=20$ psf, $\delta=23^\circ$) and the calculated factor of safety for static conditions after the equipment was removed using *large displacement shear strengths* was 1.23 ($\approx a=27.9$ psf, $\delta=16.9^\circ$).

Determine if Gas Pressure Relief Systems is Necessary

If surface emissions monitoring indicates that excessive near surface gas production is occurring or the waste was placed within two years of capping, a gas pressure relief system is recommended.

There have been at least four failures in the United States that were most likely caused by landfill gas reducing the effective normal force on the geosynthetic / recompacted soil barrier layer interface. At least half of these failures occurred in areas that had received waste within eighteen months of the failure occurring. Two of these occurred after all or part of the active gas extraction system went down. To avoid such a failure, the area to be capped should be examined for areas with excessive surface gas emissions. These emissions can be detected with methods similar to those used when conducting surface emissions monitoring for the New Source Performance Standards (NSPS). A gas pressure relief layer can be installed as part of the composite cap system over any recently disposed waste and over areas where gas pressure is a concern. For more information on the design of gas pressure relief system see Thiel, R.S., 1998, "Design Methodology for a Gas Pressure Relief Layer Below a Landfill Geomembrane over to Improve Slope Stability", Geosynthetics International, Vol. 5, No. 6, pp. 589-617.

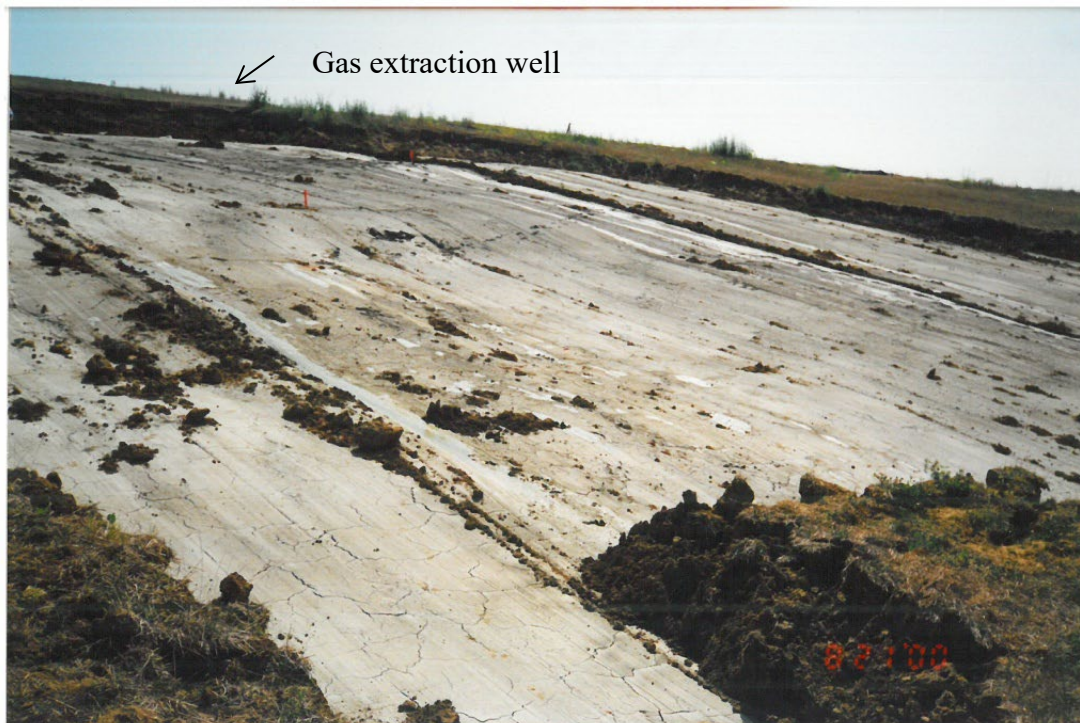


Figure 9-8. Failure at an Ohio landfill between clay barrier layer and textured LLDPE geomembrane, waste was in place less than a year and a nearby active gas well was not functioning due to a lightning strike.

Anchoring Geosynthetics on *Internal Slopes* and Between Phases or Cells

This calculation should be included if the designer intends to have the geosynthetics pullout of the anchor instead of ripping if a failure occurs. It should be noted that if the typical two foot by two-foot anchor trench is used the geomembrane will normally rip if a failure occurs.

A run out is the portion of geosynthetic that extends beyond the crest of a slope or edge of a phase or cell. An anchor is soil or other material placed on top of the geosynthetic to hold it in place.

A run out is the portion of geosynthetic that extends beyond the crest of a slope or edge of a phase or cell. An anchor is soil or other material placed on top of the geosynthetic to hold it in place.

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

The following equations are after Qian et al., 2002, and may be used to determine if the geosynthetics will pull out (Anchorage Ratio (AR) > 1) or if the geosynthetics will tear (AR < 1).

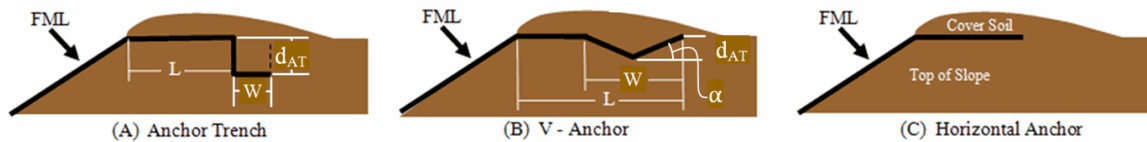


Figure 9-9.

Equation 9-3

$$AR = \frac{T_{GM}}{T_{AT}}$$

Equation 9-4 Anchor Trench

$$T_{AT} = \frac{\gamma \cdot d \cdot L \cdot \tan \delta + 2 \cdot \gamma \cdot [(1 - \sin \phi) \cdot \gamma \cdot (d + 0.5 \cdot d_{AT}) \cdot d_{AT} (d + d_{AT}) \cdot W] \cdot \tan \delta}{\cos \beta - \sin \beta \cdot \tan \delta}$$

Equation 9-5 symmetric V-Anchor

$$T_{AT} = \frac{\gamma \cdot d \cdot L \cdot \tan \delta + 2 \cdot \gamma \cdot (d + 0.5 \cdot d_{AT}) \cdot \tan \delta / \tan \alpha}{\cos \beta - \sin \beta \cdot \tan \delta}$$

Equation 9-6 Horizontal Anchor

$$T_{AT} = \frac{\gamma \cdot d \cdot L \cdot \tan \delta}{\cos \beta - \sin \beta \cdot \tan \delta}$$

Where

AR = **Anchorage Ratio**

T_{GM} = Tensile strength at break (lb/in)

T_{AT} = Anchor trench tension (lb/in)

γ = Soil unit weight (lb/in³)

d = Thickness of the cover soil (in)

L = Embedment length (in)

W = Width of anchor trench (in)

δ = geosynthetics / soil friction angle (°)

β = Side slope angle (°)

d_{AT} = Depth of the anchor trench (in)

α = Slope of the V-shaped anchor trench (°)

ϕ = Soil internal friction angle (°)

APPENDIX 9-A Examples

Shallow Failure - Example Calculations

A 200-ft thick landfill in has 3(H):1(V) (18.43°) *final slopes*. The *final slopes* comprise 1.5 feet of recompacted soil liner; a 40-mil textured geomembrane; a 330 mil (0.008382 m, 0.0275ft) thick geocomposite drainage layer with a measured transmissivity of 1.20×10^{-3} m²/sec and a 2.5-foot thick *protective layer* with a long-term permeability of 1×10^{-4} cm/sec. The geocomposite outlets above built-in surface water control benches at intervals of no greater than 40 feet of vertical drop along the *final slopes*;

The *internal slopes* 3(H):1(V) (18.43°) rise 50 feet above the base of excavation and are comprised of 3-foot recompacted soil liner, a 60-mil textured geomembrane, and a 1-foot granular drainage layer with a permeability of 1 cm/sec. A leachate collection pipe is located at the toe of slope. The site is located in an area with 100 feet of glacial till, and limestone bed rock.

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. The shear strengths of the clay and the glacial till were conservatively estimated since neither appeared to be a *critical layer*. The transmissivity of the geocomposite and the permeability of the *protective layer* would also have to be verified.

Shallow Failure, Unsaturated Static Conditions and Peak Shear Strengths - Example Calculation

Determine the interface 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.

Using equation 39 from Giroud et al. 1995 solved for interface friction angle ([Equation 9-7](#)) determine the required interface friction to achieve a 1.5 factor of safety above the geomembrane for the slopes between the drainage outlets every 40ft of vertical drop.

Equation 9-7

$$\delta = \text{atan} \left(FS \tan \beta - \frac{a \tan \beta}{\gamma_t t \sin \beta} \right)$$

Where

FS = factor of safety (dimensionless) 1.5

a = interface adhesion (psf), 0.0 psf

t = thickness of the soil layer (ft), 2.5 ft. or 1 ft for *internal slopes*

β = slope angle ($^\circ$), 18.43 $^\circ$

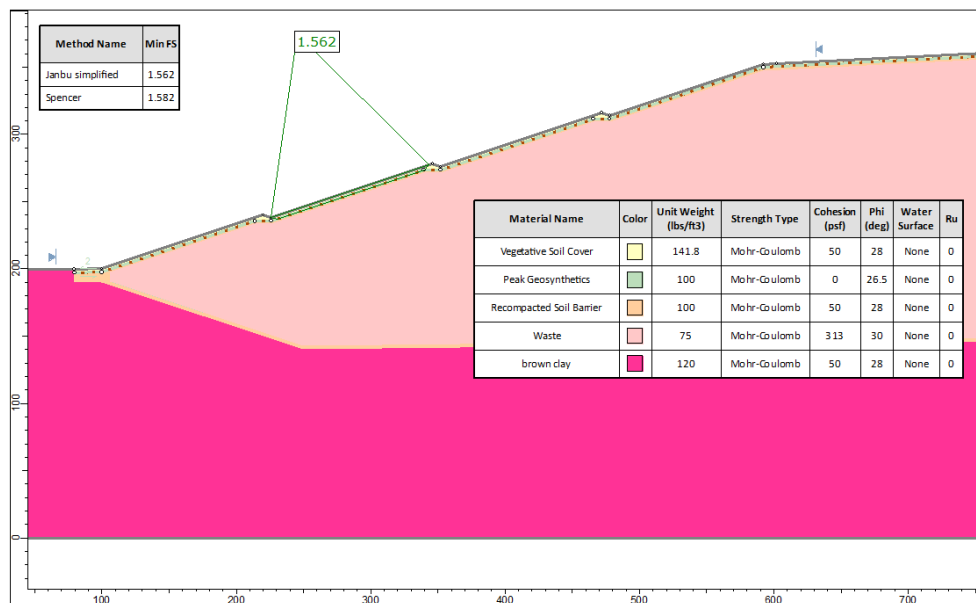
δ = interface friction angle ($^\circ$),

γ_d = dry unit weight of soil (pcf), 120 pcf

γ_t = total unit weight of soil (pcf), $\gamma_d(1 + \%moisture) = 120(1 + 0.16) = 139.2$ pcf

$$\delta = \text{atan} \left(1.5 \tan(18.43) - \frac{0 \tan(18.43)}{139.2 * 2.5 * \sin(18.43)} \right) = 26.5^\circ$$

Using a two-dimensional limit equilibrium method check if the built-in surface water control benches are stable.

**Conclusion**

Based on shallow static conditions, a friction angle of 26.5 $^\circ$ is sufficient for both internal and final slopes.

Shallow Failure, *Saturated* Static Conditions - Example Calculations

Mid-Slope final slope outlet structure configuration.

Determine if the 26.5° friction angle, or a higher friction angle, will be needed to obtain the required 1.10 *saturated* static factor of safety for the *final slopes* using the worst-case cross-sections for the facility. Calculations were performed with equations described in the following papers *Influence of Water Flow on The Stability of Geosynthetic-Soil Layered Systems on Slopes*, Giroud et al. 1995, *Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers*, Giroud et al. 2000a, and *Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers Comprising Two Different Slopes*, Giroud et al. 2000e

In this example we make the following assumptions: water is flowing vertically through the *protective layer* flows at a very conservative rate, the maximum permeability expected for a dozer placed cohesive soil 1×10^{-4} cm/sec, and the calculated average thickness of liquid in the drainage layer controls stability as long as the calculated maximum thickness of liquid in the drainage layer is less than the thickness of the drainage layer. This latter assumption is based on what would likely happen if a 100-year storm event occurred after a period of wet weather.

Using equation 40 from Giroud et al. 2000a ([Equation 9-8](#)), determine if the maximum thickness of liquid in the drainage layer (t_{max}) is less than the thickness of the drainage layer.

Equation 9-8

$$t_{max} = \frac{\sqrt{\tan^2 \beta + 4q_h/k_{LTIS}} - \tan \beta}{2 \cos \beta} \left\{ 1 - 0.12 \exp \left[- \left[\log \left(\frac{8(q_h/k_{LTIS})}{5 \tan^2 \beta} \right)^{5/8} \right]^2 \right] \right\} L$$

Where

- β = slope angle (°), 18.43°
- q_h = rate of liquid supply rate per unit surface area measured horizontally (cm/sec), 1×10^{-4} cm/sec
- k_{LTIS} = hydraulic conductivity of liquid collection layer material (cm/sec), 1.7 cm/sec see below
- L = length (measured horizontally) of rectangular liquid collection layer in direction of flow (cm), $40 \text{ ft} \times 3 = 120 \text{ ft} = 3657.6 \text{ cm}$

Using Equation 9-1

$$K_{LTIS} = \frac{\frac{1.20 \times 10^{-3}}{0.008382}}{3 \times 1.4 \times 1.2 \times 1.0 \times 1.2 \times 1.5} = 1.70 \times 10^{-2} m/s = k_{LTIS} = 1.70 c m/s$$

$$t_{max} = \frac{\sqrt{(\tan(18.43))^2 + \frac{4 \times 0.0001}{1.70}} - \tan(18.43)}{2 \times \cos(18.43)} \times \left\{ 1 - 0.12 \exp \left[- \left[\log \left(\frac{8 \times (0.0001/1.70)}{5 \times (\tan(18.43))^2} \right)^{5/8} \right]^2 \right] \right\} 3657.6$$

$$t_{\max} = 0.678\text{cm} = 0.0222\text{ft} = 0.267\text{in} < 0.330\text{in}$$

Therefore, liquid head remains within the thickness of the drainage layer.

Using equation 58 from Giroud et al. 1995 ([Equation 9-9](#)), determine the factor of safety above the geomembrane for the slopes between the drainage outlets every 40ft of vertical drop.

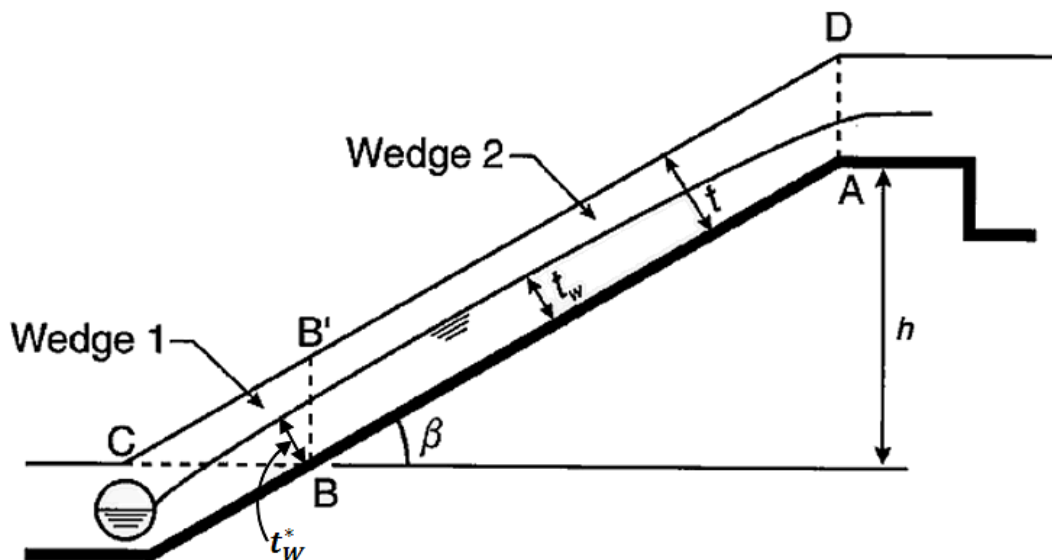


Figure 9-10, "Flow thickness for the case of a slope of finite height Giroud et al. 1995

Equation 9-9

$$\begin{aligned}
 FS_A = & \frac{\gamma_t(t - t_w) + \gamma_b t_w}{\gamma_t(t - t_w) + \gamma_{sat} t_w} \frac{\tan \delta_A}{\tan \beta} + \frac{a_A / \sin \beta}{\gamma_t(t - t_w) + \gamma_{sat} t_w} \\
 & + \frac{\gamma_t(t - t_w^*) + \gamma_b t_w^*}{\gamma_t(t - t_w) + \gamma_{sat} t_w} \frac{t}{h} \frac{\sin \phi}{2 \sin \beta \cos \beta \cos(\beta + \phi)} \\
 & + \frac{c \ t/h}{\gamma_t(t - t_w) + \gamma_{sat} t_w} \frac{\cos \phi}{\sin \beta \cos(\beta + \phi)} + \frac{T/h}{\gamma_t(t - t_w) + \gamma_{sat} t_w}
 \end{aligned}$$

Where

- FS_A = **factor of safety** if the slip surface is above the geomembrane (dimensionless)
 a_A = **interface adhesion** along a slip surface located above the geomembrane (psf), 0.0 psf
 c = **cohesion of soil** above the geomembrane (psf), 50 psf
 ϕ = **internal friction angle** of the soil above the geomembrane ($^\circ$), 28°
 d = **thickness of drainage layer** (ft), 0.0275 ft (330 mil geocomposite)
 h = **height of slope** (ft), 40 ft
 T = **geosynthetic tension** (pf), 0.0 pf
 t = **thickness of the soil layer** above interface (ft), 2.5 ft
 t_w = **average water flow thickness** (ft), 0.012 ft. see below
 t_w^* = **water flow thickness in Wedge 1**, $\approx t_{\max} = 0.015583$ ft see above
 β = **slope angle** ($^\circ$), 18.43°
 δ_A = **interface friction angle** along a slip surface located above the geomembrane ($^\circ$), 26.5°
 γ_b = **buoyant unit weight** of soil (pcf), $\gamma_{sat} - \gamma_w = 141.84 - 62.4 = 79.44$ pcf
 γ_d = **dry unit weight** of soil (pcf), 120 pcf
 γ_{sat} = **saturated unit weight** of soil (pcf), $\gamma_d + n\gamma_w = 120 + 0.35 \times 62.4 = 141.84$ pcf
 γ_t = **total unit weight** of soil (pcf), $\gamma_d(1 + \%moisture) = 120(1 + 0.16) = 139.2$ pcf
 γ_w = **unit weight of water**, 62.4 pcf
 n = **soil porosity** (dimensionless), 0.35

Average water flow thickness (t_w) can be calculated with one of the following formulas from Schroeder et al. 1994:

Equation 9-10

$$t_w = \cos \beta \frac{q_h L / k_d}{2(\sin \beta)(\cos \beta)} \text{ for } q_h L / k_d < 0.4 \sin^2 \beta$$

or

Equation 9-11

$$t_w = \cos\beta \frac{\pi\sqrt{q_h L/k_d}}{4(\cos\beta)} \text{ for } q_h L/k_d > 0.4\sin^2\beta$$

See Schroeder et al. 1994 for $q_h L/k_d = 0.4\sin^2\beta$ case.

$$q_h L/k_d = \frac{0.0001 \times 3657.6}{1.70} = 0.215 > 0.04 = 0.4\sin^2\beta$$

$$t_w = \cos\beta \frac{\pi\sqrt{q_h L/k_d}}{4(\cos\beta)} = \cos(18.43) \frac{3.14\sqrt{0.0001 \times 3657.6/1.70}}{4 \times \cos(18.43)} = 0.364 \text{ cm} = 0.012 \text{ ft}$$

$$\begin{aligned} FS_A &= \frac{139.2 \times (2.5 - 0.012) + 79.44 \times 0.012}{139.2 \times (2.5 - 0.012) + 141.84 \times 0.012} \times \frac{\tan(26.5)}{\tan(18.43)} \\ &+ \frac{0/\sin(18.43)}{139.2 \times (2.5 - 0.012) + 141.84 \times 0.012} \\ &+ \frac{139.2 \times (2.5 - 0.0222) + 79.4 \times 0.0222}{139.2 \times (2.5 - 0.012) + 141.84 \times 0.012} \\ &\times \frac{2.5}{40} \frac{\sin(28)}{2 \times \sin(18.43) \times \cos(18.43) \times \cos(18.43 + 28)} \\ &+ \frac{50 \times (2.5/40)}{139.2 \times (2.5 - 0.012) + 141.84 \times 0.012} \times \frac{\cos(28)}{\sin(18.43) \times \cos(18.43 + 28)} \\ &+ \frac{0/40}{139.2 \times (2.5 - 0.012) + 141.84 \times 0.012} \\ &= 1.60 \end{aligned}$$

Conclusion

The *final slopes* comprise 1.5 feet of recompacted soil liner; a 40-mil textured geomembrane; a 330-mil thick geocomposite drainage layer with a measured transmissivity of $1.2 \times 10^{-3} \text{ m}^2/\text{sec}$; and a 2.5-foot thick *protective layer* with a long-term permeability of $1 \times 10^{-4} \text{ cm/sec}$. The geocomposite outlets above built-in surface water control benches berms at intervals of no greater than 40 feet of vertical drop along the *final slopes*, is stable under *saturated* conditions, and a friction angle of 26.5° is acceptable under this scenario.

Crest of final slope outlet structure configuration.

Using equation 20 from Giroud et al. 2000e ([Equation 9-12](#)) determine the maximum thickness of liquid in the drainage layer in the lower slope ($t_{\text{down max}}$) considering there are two slopes that are not provided with a drain at their intersection (see image below).

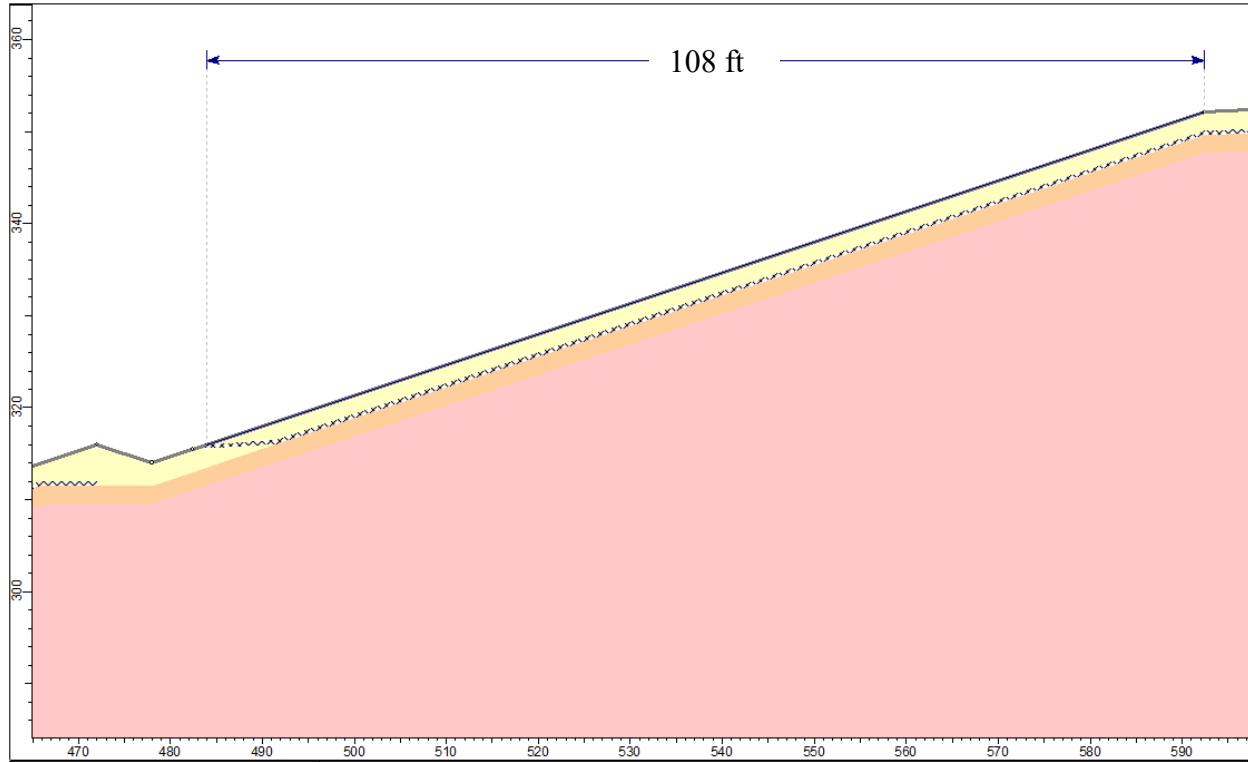


Figure 9-11: Crest of final slope outlet structure.

Equation 9-12

$$t_{\text{down max}} = \frac{q_h(L_{\text{up}} + L_{\text{down}})}{k_{\text{down}} \sin \beta_{\text{down}}} = \frac{0.0001 \times (15240 + 3292)}{1.70 \times \sin(18.43)} = 3.45 \text{ cm} = 1357.2 \text{ mil}$$

Where

β_{down} = slope angle ($^{\circ}$), 18.43°

q_h = rate of liquid supply rate per unit surface area measured horizontally (cm/sec), 1×10^{-4} cm/sec

k_{down} = hydraulic conductivity of the material of downstream section of liquid collection layer (cm/sec), 1.70 cm/sec

L_{down} = horizontal length of downstream section of liquid collection layer (cm),
3292 cm

L_{up} = horizontal length of upstream section of liquid collection layer (cm), 15240
cm

Since $t_{\text{down max}}$ (1357.2 mil) is greater than the thickness of the drainage layer (330 mil) use (thickness of drainage layer + thickness of cover soil) = $2.5 + 0.0275 = 2.53\text{ft}$ for both t_w and t_{max}

Using [Equation 9-9](#) and the interface friction angle 26.5, the factor of safety above the geomembrane for the slope described above is:

$$FS_A = 0.93$$

Conclusion

This is an unacceptable factor of safety

This slope could be redesigned so the upper slope is *drained* and does not flow into the lower slope. When this new configuration is analyzed the upper slope still has a maximum liquid height greater than the thickness of the drainage layer but since it is only a 5% slope it is still stable with a $FS_A = 6.18$. The lower slope has a head less than the thickness of the drainage layer $t_{\text{max}} = 0.019\text{ ft}$ and a $FS_A = 1.62$.

An alternative redesign is to increase the minimum interface friction angle to 31° to obtain a $FS_A = 1.10$.

Note: The length where the head in the drainage layer equals the thickness of the drainage layer (x_m) can be calculated using the following formula after Giroud et al 2000a:

$$x_m = \frac{k_d \sin \beta t_d}{q_h}$$

Shallow Failure, *Unsaturated* Seismic Conditions - Example Calculation

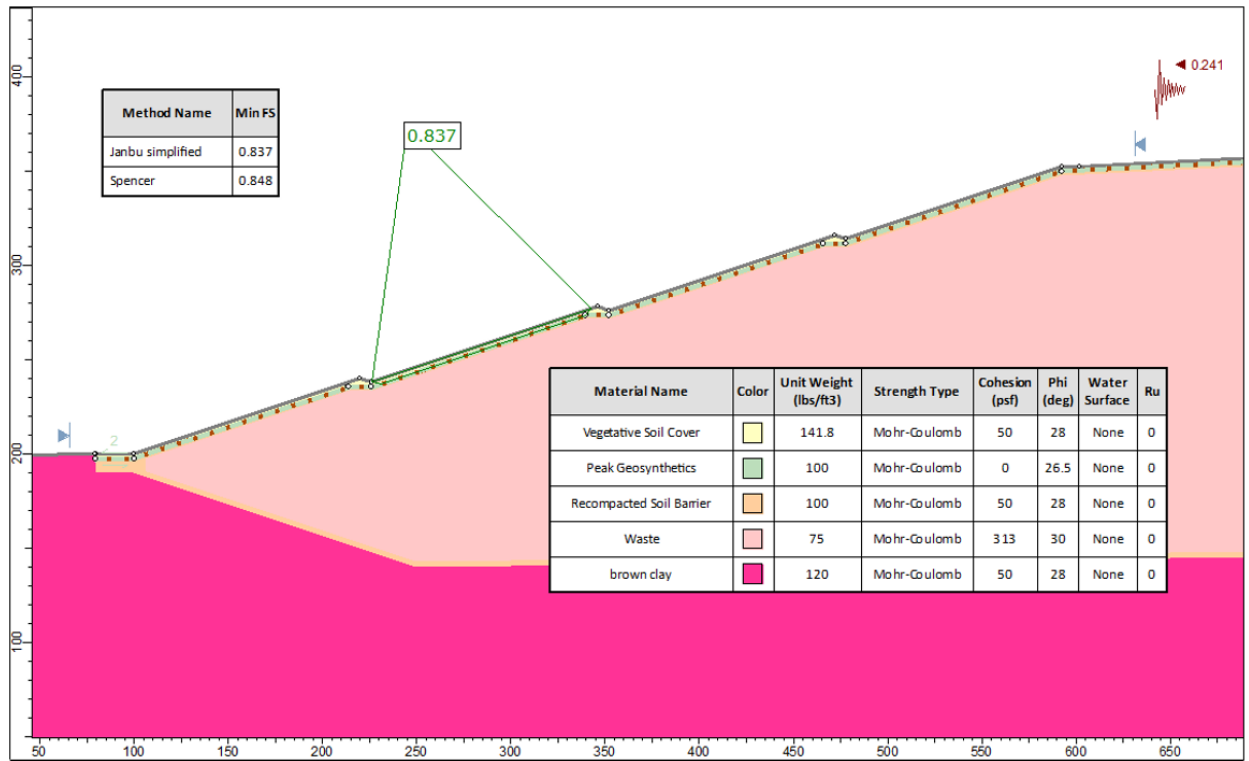
Using the *large displacement interface shear strength* ($\delta=20.0'$ and $a = 0.0$ psf) and assuming the cover soil is 2.5 ft thick has a cohesion of 50 psf and a wet unit weight of 120pcf, perform a seismic evaluation assuming the municipal solid waste landfill is located in southern Allen County, Ohio.

Screening analysis: using the proposed interface shear strength of the geosynthetics and the estimated MHA_{Rock} for the *bedrock* beneath the site.

1. Estimate MHA_{Rock} using site-specific USGS NSHMP's 2023 data for Peak Ground Acceleration based on a probability level of 2% exceedance in 50 years and a V_{s30} of 760 m/s (PGA).

USGS NSHMP's 2023 data indicates a PGA of 0.241 g so $MHA_{Rock} = 0.241$ g

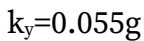
2. Perform a pseudo-static analysis for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* for side slopes and lowest *peak shear strength* for the bottom slopes and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section of this chapter. It should be noted that in this example the gray clayey silt is *saturated*, so it is being modeled using *undrained shear strengths*, however, when analyzed the worst-case failure surfaces did not pass through this unit. Setting the horizontal seismic load coefficient (kH) for the base of the facility as $kH = MHA_{Rock} = 0.241$ g.



- The factor of safety for this analysis is 0.808, which is less than 1.0 so a more detailed analysis needs to be performed. The Simplified Procedure outlined in Bray et al 1998 will be performed and Richardson et al 1995 will be performed. It should be noted that only one of these analyses typically should be performed.

Simplified Procedure outlined in Bray et al 1998

- Using a limit equilibrium slope stability program which has a method that satisfies full equilibrium, such as Spencer, calculate the yield acceleration coefficient, k_y in the following manner: Perform pseudo-static analyses for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section of this chapter. While fluctuating the horizontal seismic load coefficient for the base of the facility until a factor of safety of 1.0 is achieved. The horizontal seismic load coefficient where the Factor of safety is equal to 1.0 is considered the yield acceleration coefficient k_y .



- USGS NSHMP's 2023 disaggregation indicates a mean (over all sources) r: 26.08 km so use a distance of 26 km

- 9-31

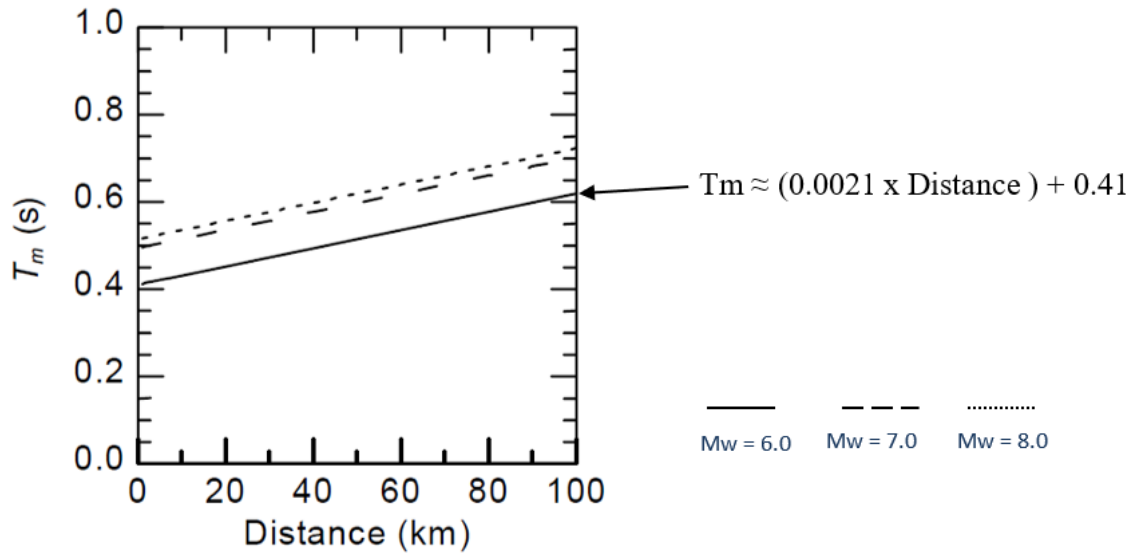


Figure 9-12: Simplified Characterization of Earthquake Rock Motions - Frequency Content, T_m (Bray et al 1998)

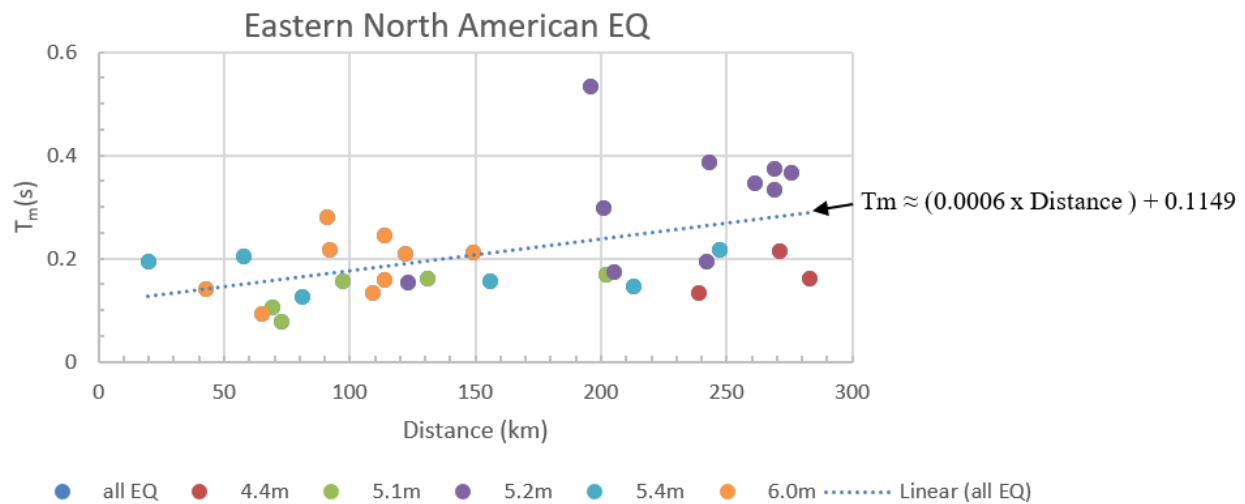


Figure 9-13: Simplified Characterization of Eastern North American Earthquake Rock Motions - Frequency Content, $T_{m-EQ} = 0.465$ s Bray or 0.131 s Eastern North America

4. Calculate the fundamental period of the landfill using the following formula:

Equation 9-13

$$T_{s-waste} = 4H/V_s$$

Where

$T_{s-waste}$ = Fundamental Period (s)

H = Average Waste Height (m) = 113 ft $\approx$ 34.4 m

V_s = Shear Wave Velocity of Waste at Average Waste Height (m/s) using Figure 9-17 = 251

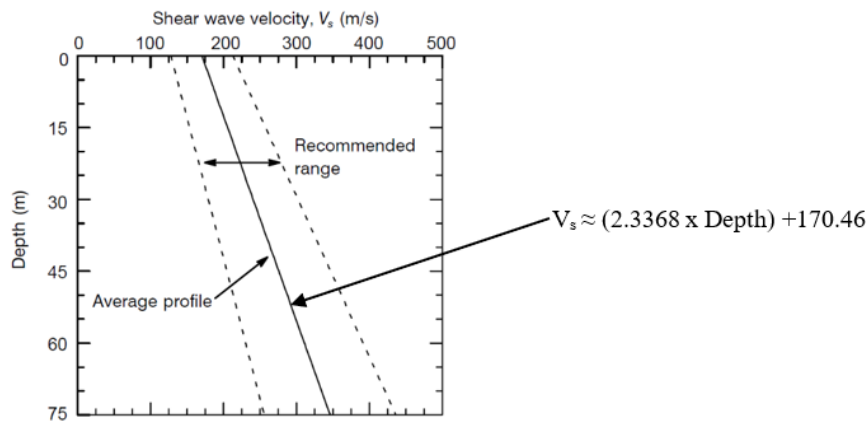


Figure 9-14: Shear wave velocities of municipal solid waste (Bray et al 1998).

$$T_{s-waste} = 4 \times 34.4 / 251 = 0.549s$$

5. Calculate $T_{s-waste}/T_{m-EQ}$.

From steps 3 $T_{m-EQ} = 0.465$ s Bray or 0.131 s Eastern North America and from step 4 $T_{s-waste} = 0.549$ s, therefore $T_{s-waste}/T_{m-EQ} = 1.18$ Bray, 4.19 Eastern North America

6. Using Figure 9-18 to approximate the MHA_{Top} considering the 16% probability of exceedance.

$$MHA_{Top} / [(MHA_{Rock}) (NRF)] = 1.16 \text{ Bray}$$

$$MHA_{Top} / [(MHA_{Rock}) (NRF)] = 0.677 \text{ Eastern North America}$$

$$NRF = 1.15 \text{ for } MHA_{Rock} = 0.241g$$

$$MHA_{Top} = (0.241g)(1.15)(1.16) = 0.321g \text{ Bray}$$

$$MHA_{Top} = (0.241g)(1.15)(0.678) = 0.188g \text{ Eastern North America}$$

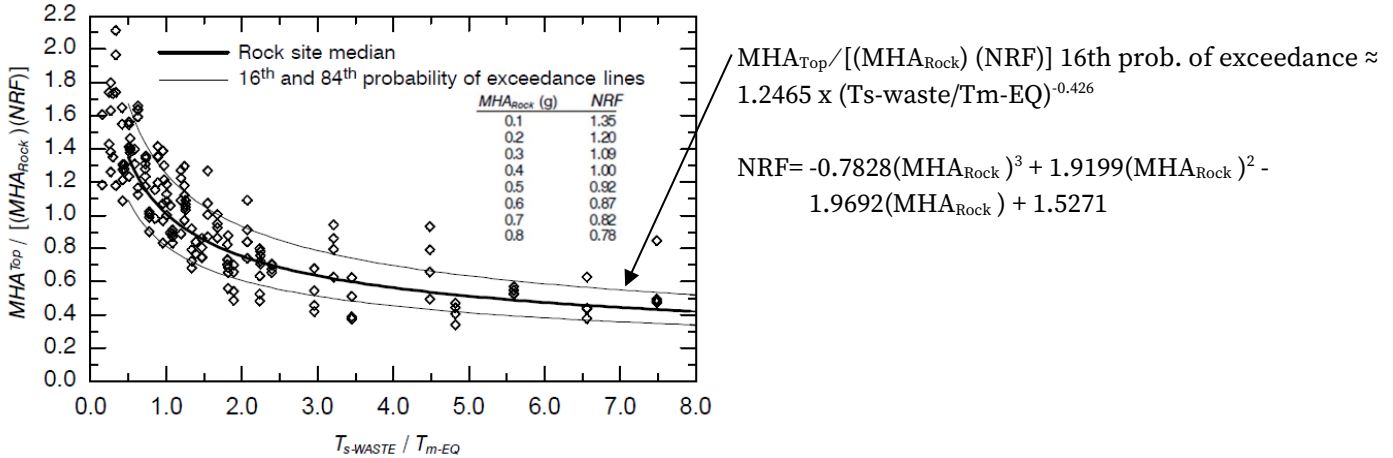


Figure 9-15: Normalized maximum horizontal acceleration at the top versus the normalized fundamental period of the waste fill (from Bray and Rathje 1998) (Bray et al, 1998)

7. Estimate the maximum horizontal acceleration along the entire side slope ($MHEA_{cover}$) using the following formula:

Equation 9-144

$$MHEA_{cover} = 0.65(MHA_{top})$$

$$MHEA_{cover-bray} = 0.209g$$

$$MHEA_{cover-Eastern\ North\ American} = 0.122g$$

8. Using Figure 9-19 estimate the seismically induced normalized Cover liner sliding displacements:

Bray: $k_{max} = MHEA_{cover} = 0.209g$, and $k_y = 0.055$, so $k_y/k_{max} = 0.263$

Eastern North America: $k_{max} = MHEA_{cover} = 0.122g$, and $k_y = 0.055$, so $k_y/k_{max} = 0.451$

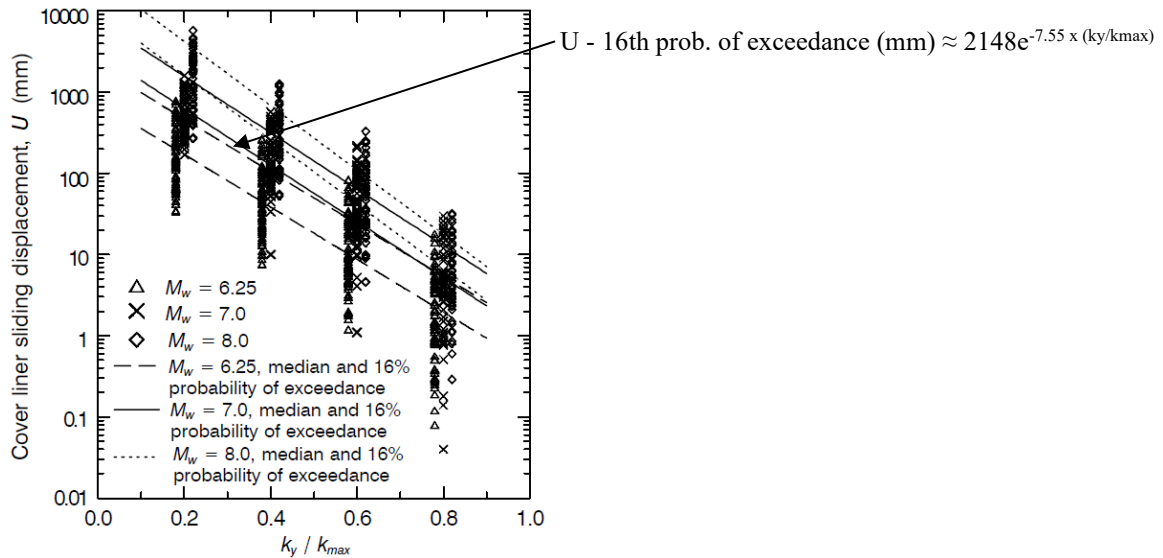


Figure 9-16: Cover liner sliding displacements (Bray et al, 1998)

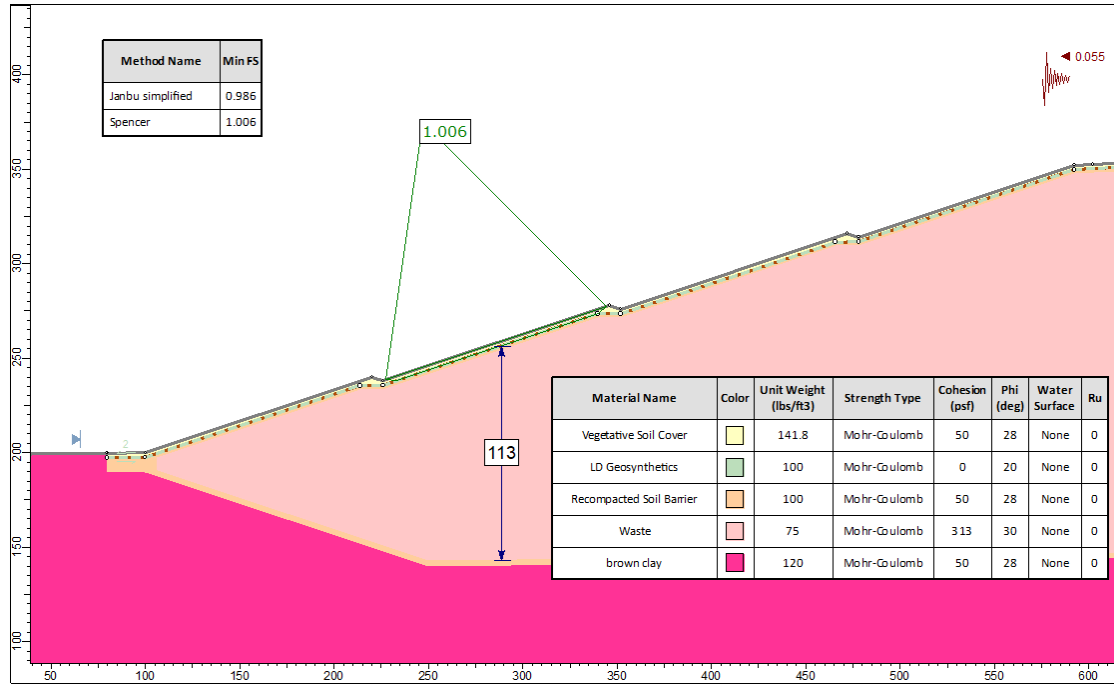
Bray – $U \approx 295$ mm

Eastern North America - $U \approx 71$ mm

Since the deformation based on eastern North American earthquakes is appropriate for a landfill located in Ohio a deformation of 71 mm would be expected. Which is less than 30 cm so the facilities design is acceptable.

Simplified Procedure outlined in Richardson et al 1995

1. Using a limit equilibrium slope stability program which has a method that satisfies full equilibrium, such as Spencer calculate the yield acceleration coefficient, k_y in the following manner: Perform pseudo-static analyses for the *final slope* cross-sections that has low static factors of safety. Modeling geosynthetic interfaces as the *large displacement shear strength* of the geosynthetic with the lowest *peak shear strength* and the other materials with the shear strengths as described in the [Seismic Shear Strengths](#) section of this chapter. While fluctuating the horizontal seismic load coefficient for the base of the facility until a factor of safety of 1.0 is achieved. The horizontal seismic load coefficient where the Factor of safety is equal to 1.0 is considered the yield acceleration coefficient k_y .



$$k_y = 0.055g$$

2. Classify the top 100 feet of foundational soils at the site:

- The upper brown lean clay is a stiff soil N_{60s} that ranged from 14 to 20.
- The wet grey clayey silt is medium to stiff N_{60s} that ranged from 6 to 13 with an average of 8.
- The *saturated* sand is loose to medium dense N_{60s} that ranged from 9 to 16 with an average of 11.
- Limestone is a firm rock.

Since the soils have average N_{60s} of 8 or higher we will classify the soils as medium stiffness for the purpose of this evaluation.

3. Estimate peak outcrop acceleration ($a_{o\max}$) using site-specific USGS NSHMP's 2023 data for a spectral period of 0.0 seconds i.e. Peak Ground Acceleration based on a probability level of 2% exceedance in 50 years and a VS30 of 760 m/s (PGA).

$a_{o\max}$ = USGS NSHMP's 2023 data indicates a PGA of approximately 0.241 g

4. Estimate $k_{\max}$ for the facility using the recommendations in Richardson, 1995 page 45 step 2.

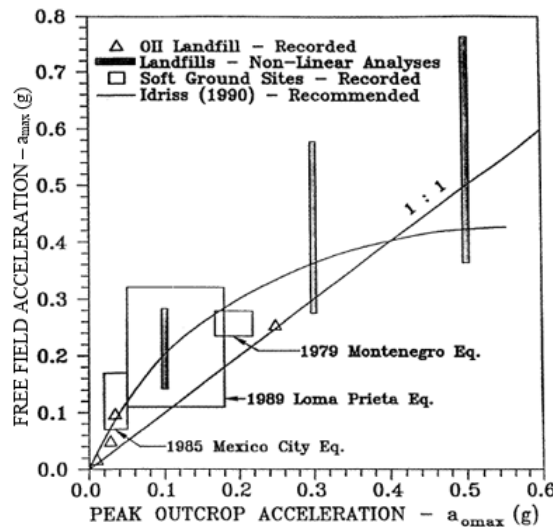


Figure 9-17: Observed Variations of Peak Horizontal Accelerations on Soft Soil and MSW Sites in Comparison to Rock Sites (Kavazanjian and Matasovic, 1994). (Richardson, 1995)

Step 2 in Richardson, 1995 states that “For medium stiff soils, use an acceleration equal to the average of the rock site acceleration and the soft soil site acceleration from Idriss' curve in (Figure 9-20) for peak *bedrock* accelerations less than or equal to 0.4 g.” The Idriss (1990) soft soil site acceleration curve indicates that the free field acceleration ($a_{\max}$) for a soft soil site would be approximately 0.32 g and the rock site acceleration is equal to $a_{\max}$ 0.241 g. The average of these values is 0.28 g so the max acceleration at the base of the landfill will be 0.28 g.

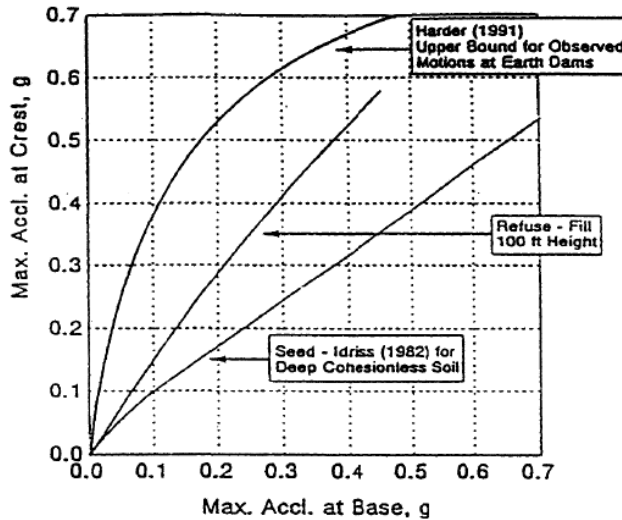
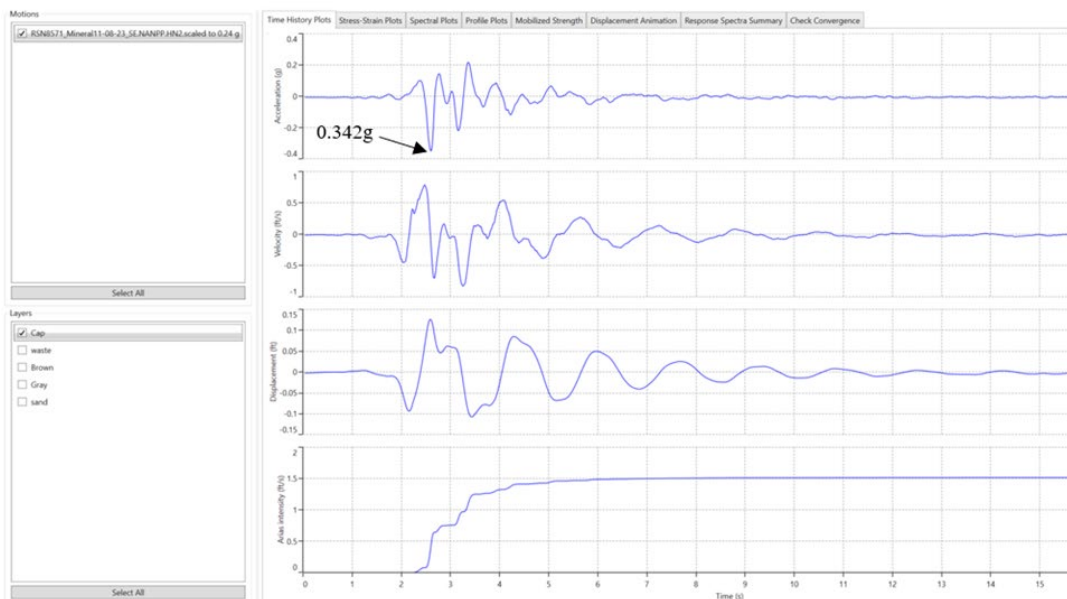


Figure 9-18: Approximate Relationship Between Maximum Accelerations at the Base and Crest for Various Ground Conditions (Singh and Sun, 1995) (Richardson 1995)

Then using Figure 9-21 determine what the maximum acceleration at crest ($a_{\max \text{ crest}}$) would be for the cap interface. $a_{\max \text{ crest}} = 0.385 \text{ g}$ therefore $k_{\max} = 0.385 \text{ g}$.

Richardson, 1995 also allows for calculating the free field acceleration using a site-specific seismic response analysis. When this was conducted the free field acceleration for the cap was calculated as 0.342 g using the site-specific response to the 2011 Mineral Virginia earthquake with data scaled to 0.24 g from North Anna Nuclear Generation Station, Mineral, VA (station SE.NANPP).



5. Calculate $k_y/k_{\max}$

Figure 9-20 and Figure 9-21 analysis $k_y/k_{\max} = 0.055 / 0.385 = 0.143$.

Site-specific seismic response analysis $k_y/k_{\max} = 0.055 / 0.342 = 0.161$.

6. Estimate displacement (U) using [Figure 9-22](#).

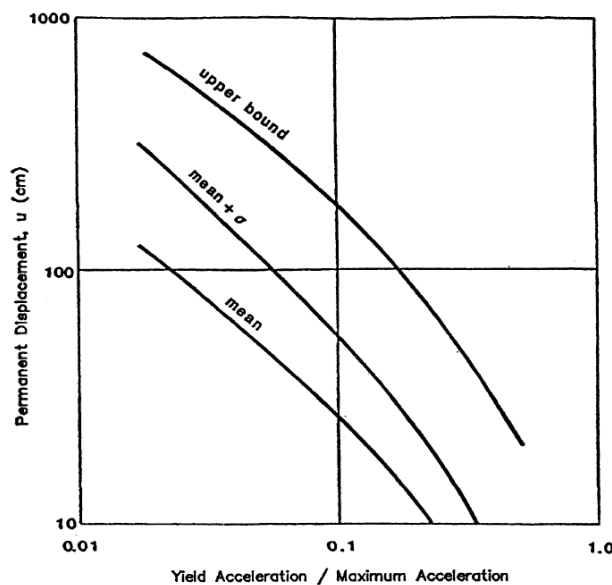


Figure 9-19: Hynes and Franklin Permanent Seismic Displacement Chart (Hynes and Franklin, 1984). (Richardson et al, 1995)

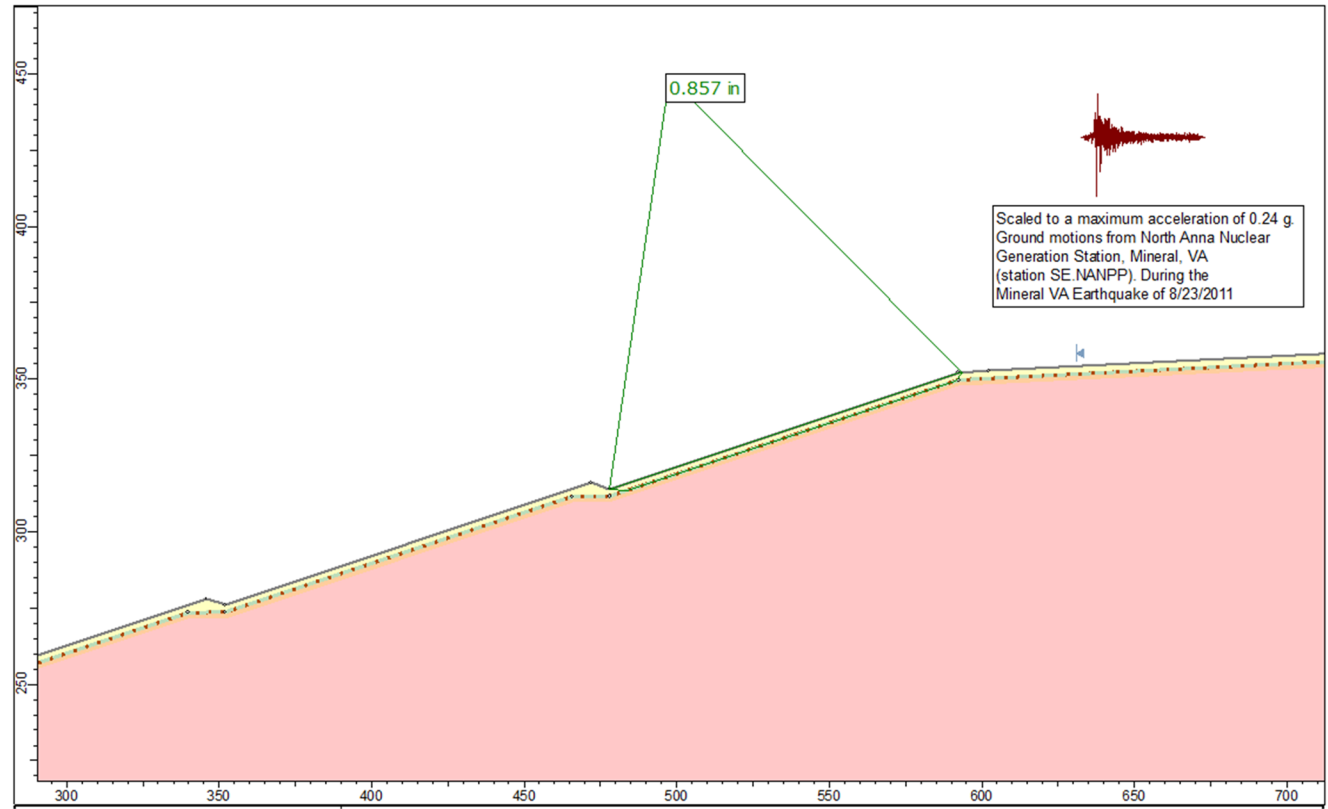
Richardson et al, 1995 states to use “the mean + σ curve from Hynes and Franklin (Figure 9-22) is recommended for simplified permanent seismic deformation analysis of the cover system, which predicts a displacement of approximately 36 cm using Figure 9-20 and Figure 9-21 and 31 cm using a site-specific seismic response analysis. Using Richardson et al, 1995 the predicted deformation is in excess of 30 cm.

Conclusion

Since Bray’s analysis method using eastern North American earthquakes is appropriate and is more rigorous than the Richardson method. We will conclude that about 7.1 cm of displacement will occur, which is less than 30 cm and is therefore acceptable. Therefore, the minimum required *large displacement shear strength* for the interfaces at this example facility needed for an acceptable design for both seismic conditions and static condition are equivalent. The quality assurance plan should require the shear

strength of the material interfaces for this example landfill meet the strengths used in these calculations.

Some computer programs can also perform deformation analyses and may also be used. For example, the program Slide 2018 2D Limit Equilibrium Analysis of Slope Stability performs a Newmark displacement analysis. The previous example yields a deformation of 0.857 inches (21.8 mm) which is less than the results of the previous methods.



Shallow Failure, Equipment loading Conditions - Example Calculation

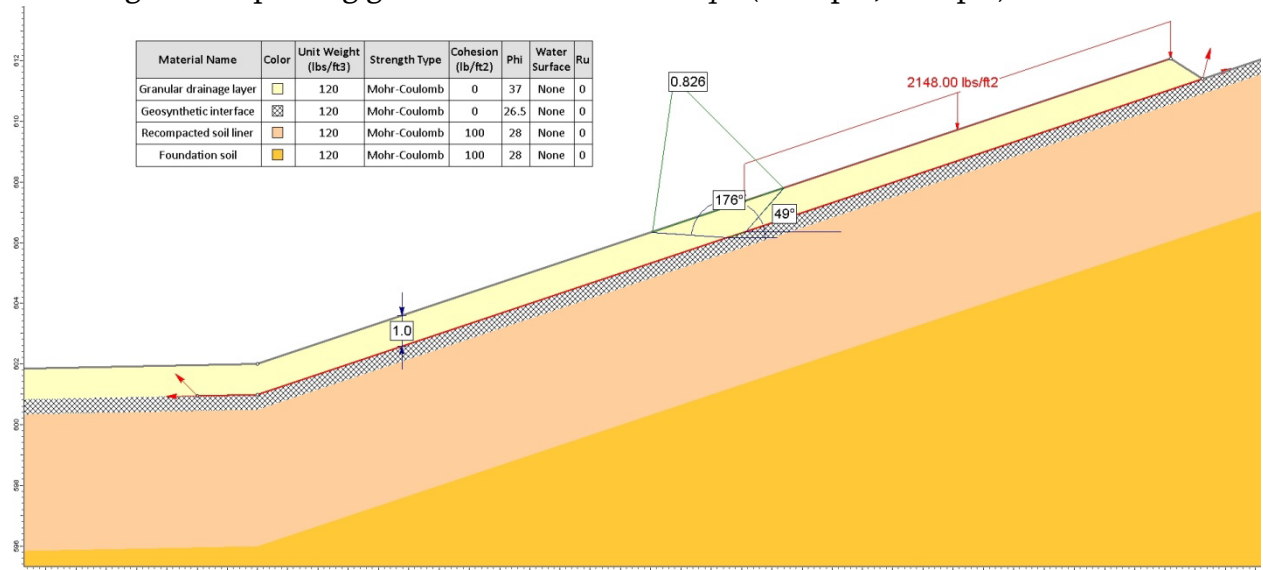
Determine if the 26.5° friction angle or a higher friction angle will be needed to obtain a 1.10 static factor of safety for the following scenarios on 3(H) to 1(V) (3 to 1) slopes:

1. a large dozer placing gravel on the *internal slope* (2148 psf, 14.9 psi)
2. a small low ground pressure dozer placing gravel or crawler carrier delivering gravel on an *internal slope* (792 psf, 5.5psi)
3. an articulated rock truck rated for 43.5 tons on a five-foot-thick road of gravel perpendicular to the *internal slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
4. an articulated rock truck rated for 43.5 tons on a road of gravel parallel to the *internal slope* with a minimum thickness of three feet (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
5. a large dozer placing 1st 18 inches of final cover (2148 psf, 14.9 psi)
6. a large dozer final cover in one lift (2148 psf, 14.9 psi)
7. a small dozer placing 1st 18 inches of final cover 792 psf, 5.5psi)
8. a small dozer placing final cover in one lift or crawler carrier delivering final cover (792 psf, 5.5psi)
9. an articulated rock truck rated for 43.5 tons on a five-foot-thick road of soil perpendicular to the *final slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
10. an articulated rock truck rated for 43.5 tons on a road with a minimum thickness of three feet of soil parallel to the *final slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf).

For this example, granular drainage layers were given a friction angle of 37° and 0 psf cohesion and final cover soil was given a friction angle of 28° and 0 psf cohesion since cohesion under low normal loads is difficult to justify.

Cross-section IS-EQ1

1. a large dozer placing gravel on the *internal slope* (2148 psf, 14.9 psi)



Cross-section IS-EQ2

2. a small dozer placing gravel or crawler carrier delivering gravel on an *internal slope* (792 psf, 5.5 psi)

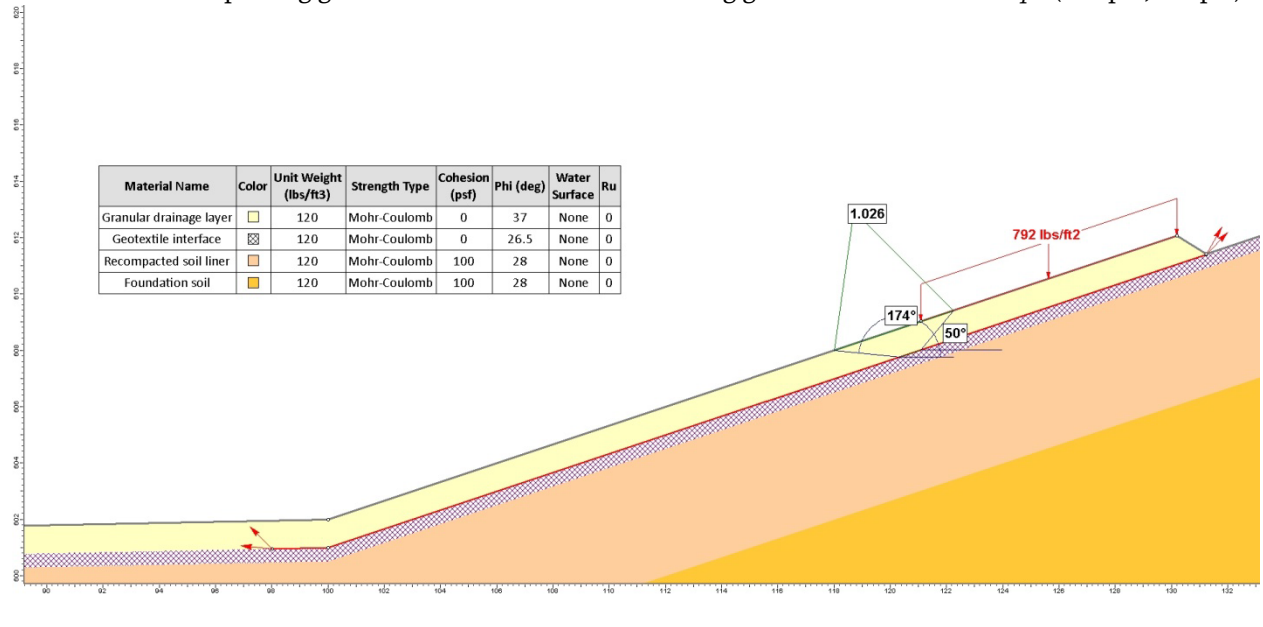


Table 1: Equipment loading results.

Equipment Type	Slope	Search Method	FOS with $\phi=26.5^\circ$	FOS = 1.00 w/ ϕ of	FOS = 1.10 w/ ϕ of
Large Dozer	Internal	Translational	0.83	>35°*	>35°*
Small Dozer	Internal	Translational	1.03		>35°*
Articulated Truck	Internal Perpendicular	Translational	1.27		
Articulated Truck	Internal Parallel Road	Translational	0.87	32°	35°
Large Dozer	Final 1 st 18"	Translational	0.89	>35°	>35°*
Large Dozer	Final	Translational	0.96	31.5°	>35°*
Small Dozer	Final 1 st 18"	Translational	0.85	29.6°	>35°*
Small Dozer	Final	Translational	1.06		28.9°
Articulated Truck	Final road Perpendicular	Translational	0.99	27.4°	>35°*
Articulated Truck	Final Parallel Road	Translational	0.93	28.8°	32.4

*It is unlikely that materials could be found that can achieve the necessary interface strengths.

Conclusion

Based on shallow static equipment loading conditions, a friction angle of 26.5° is sufficient to obtain a factor of safety of 1.00 for 3(H) to 1(V) slopes during the following scenarios:

- a small dozer placing gravel or crawler carrier delivering gravel on an *internal slope* (792 psf, 5.5psi)
- an articulated rock truck rated for 43.5 tons on a five-foot-thick-road of gravel perpendicular to the *internal slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
- a small dozer placing final cover in one lift or crawler carrier delivering final cover (792 psf, 5.5psi)

Based on shallow static equipment loading conditions a friction angle of 26.5° is **NOT** sufficient to obtain a factor of safety of 1.00 for 3 to 1 slopes during the following scenarios:

- a large dozer placing gravel on the *internal slope* (2148 psf, 14.9 psi)
- an articulated rock truck rated for 43.5 tons on a road of gravel parallel to the *internal slope* with a minimum thickness of three feet (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
- a large dozer placing 1st 18 inches of final cover

- a large dozer final cover in one lift
- a small dozer placing 1st 18 inches of final cover
- an articulated rock truck rated for 43.5 tons on a five-foot-thick road of soil perpendicular to the *final slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf)
- an articulated rock truck rated for 43.5 tons on a road with a minimum thickness of three feet of soil parallel to the *final slope* (axial load front 995 psf, center 3282 psf, and rear 3282 psf).

At facilities where large failures have occurred, either before or after waste placement, back calculations have generally shown the failure occurred along an interface that had reached *large displacement shear strengths*. One possible explanation for the occurrence of *large displacement shear strengths* is the small movements that appear to occur during granular material placement. To be reasonable and allow for material variability for facilities that cannot demonstrate a 1.00 factor of safety during static equipment loading conditions, Ohio EPA recommends a factor of safety greater than 1.25 for *internal slopes* that will be buttressed within a year assuming *unsaturated* conditions and *large displacement shear strengths*. *Internal slopes* and *final slopes* that will remain unbuttressed indefinitely will be evaluated on case-by-case bases and may necessitate increasing post closure financial assurance funding if facilities cannot demonstrate a 1.00 factor of safety during static equipment loading conditions.

REFERENCES

- Bray, J.D. Rathje, E.M., Augello, A.J. and Merry, S.M., 1998, "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills", *Geosynthetics International*, Vol. 5, Nos. 1-2, pp. 203-235.
- Caltrans, 2009, "Geotechnical Services Design Manual, Version 1.0" Division of Engineering Services Geotechnical Services.
- Dickenson, S.E., 1994, "Dynamic Response of Soft and Deep Cohesive Soils During the Loma Prieta Earthquake of October 17, 1989": Dissertation for Doctor of Philosophy in Civil Engineering at the University of California at Berkley, pp. 104-112.
- Eid, H. T., Stark, T. D., Evans, W. D., and Sherry, P., 2000, "Municipal Solid Waste Landfill Slope Failure I: Foundation and Waste Properties," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 126, No. 5, pp. 397 - 407.
- Fox, P. J., Rowland, M. G., and Scheithe, J. R. (1998). "Internal shear strength of three geosynthetic clay liners," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124, No. 10, pp. 933-944.
- Fox, P. J., Stark, T. D., and Swan, Jr. R. H. (2004). "Laboratory measurement of GCL shear strength," *Advances in Geosynthetic Clay Liner Technology: 2nd Symposium*, STP 1456, Mackey, R. E. and von
- Gilbert, R. B., 2001, "Peak versus Residual Strength for Waste Containment Systems," *Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II* (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.
- Giroud, J. P., Bachus, R. C. and Bonaparte, R., 1995, "Influence of Water Flow on the Stability of Geosynthetic-Soil Layered Systems on Slopes," *Geosynthetics International*, Vol. 2, No. 6, pp. 1149 - 1180.
- Giroud, J. P., Zornberg, J. G., and Zhao, A., 2000a, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers," *Geosynthetic International*, Vol. 7, Nos. 4 - 5.
- Giroud, J. P., Zhao, A., and Richardson, G. N., 2000d, "Effect of Thickness Reduction on Geosynthetic Hydraulic Transmissivity," *Geosynthetic International*, Vol. 7, Nos. 4 - 5.
- Giroud, J. P., Zornberg, J. G., and Beech, J.F., 2000e, "Hydraulic Design of Geosynthetic and Granular Liquid Collection Layers Comprising Two Different Slopes," *Geosynthetic International*, Vol. 7, Nos. 4 - 5.

Hansen M.C., 2012, "Earthquakes in Ohio, Educational Leaflet No.9," Ohio Department of Natural Resources.

Harder, L.F. Jr., 1991, "Performance of earth dams during the Loma Prieta earthquake," Proc. 2nd Int'l. Conf. on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, Vol. II, 1613-1629.

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

Kavazanjian, E., Jr., and Matasovic, N. 1995a. Seismic Analysis of Solid Waste Landfills. Geoenvironment 2000. ASCE Geotechnical Special Publication No. 46, Volume 2, pp. 1066-1080.

Kavazanjian, E., Jr., Matasovic, N., Bonaparte, R., and Schmertmann, G.R. 1995b. Evaluation of MSW Properties for Seismic Analysis. Geoenvironment 2000. ASCE Geotechnical Special Publication No. 46, Volume 2, pp. 1126-1141.

Kavazanjian, E., Jr., Stokoe, K., and Bray, J., 1996, "In Situ Wave Velocity of Solid Waste from Surface Wave Measurements", Proceedings of the second international congress on Environmental Geotechnics, Osaka Japan.

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

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

Louisiana Department of Transportation and Development (LA DOTD), 2000, 2000 Standard Specifications for Roads & Bridges Manual

Makdisi, F.I., Seed, H.B. (1978), "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations," Journal of Geotechnical Engineering Division, ASCE Vol. 104, No. GT7, pp. 849-867.

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

Matasovic, N., and Kavazanjian, E., Jr., 1998a, "Cyclic Characterization of OII Landfill Solid Waste", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 124 No. 3

Matasovic, N., Kavazanjian, E., Jr. and Giroud, J.P., 1998b, "Newmark

Seismic Deformation Analysis for Geosynthetic Covers”, *Geosynthetics International*, Vol.5, Nos. 1-2, pp. 237-264.

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

Mirafi “Geotextile Filter Design, Application and Product Selection Guide”

Ohta, Y., and Goto, N., 1978, "Empirical Shear Wave Velocity Equations in Terms of Characteristic Soil Indexes." *Earthquake Engineering and Structural Dynamics*. 6, pp. 167-187.

Qian, Xuede, Koerner, R. M., and Gray, D. H., 2002, “Geotechnical Aspects of Landfill Design and Construction.” Text and Technical Reference Book, Prentice Hall, Inc., Upper Saddle River, New Jersey, 18 chapters, 717 pages.

Richardson, G.N., Kavazanjian, E., and Matasovic, N., 1995, “RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities,” EPA/600/R-95/051, Figure 4.13.

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

Richardson, G.N., Smith, S. A., and Scheer, P. K., 2008, “Active LFG control: An unreliable aid to veneer stability”, *Geosynthetics Magazine*, Industrial Fabrics Association International.

Sahadewa, A., Zekkos, D., Lobbestael, A., Woods, R., 2011, “Shear Wave Velocity Measurements at Municipal Solid Waste Landfills in Michigan”, 2011 Pan-Am CGS Geotechnical Conference, Paper 342.

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

Schroeder, P., Dozier, T., Zappi, P., McEnroe, B., John W. Sjostrom, J. and Peyton, R. 1994, “The Hydrologic Evaluation of Landfill Performance (Help) Model Engineering Documentation for Version 3” EPA/600/R-94/168b, September 1994, U.S. Environmental Protection Agency Office of Research and Development, Washington, DC.

- Soong, T. Y. and Koerner, R. M., 1997, "The Design of Drainage Systems over Geosynthetically Lined Slopes," GRI Report #19.
- Spencer, E., 1967. "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces," *Geotechnique*, Vol. 17, pp. 11-26.
- Spencer, E., 1973. "Thrust Line Criterion in Embankment Stability Analysis," *Geotechnique*, Vol. 23, pp.85-100.
- Stark, T. D., Eid, H. T., Evans, W. D., and Sherry, P., 2000, "Municipal Solid Waste Landfill Slope Failure II: Stability Analyses," *Journal of Geotechnical and Geoenvironmental Engrg.*, ASCE, Vol. 126, No. 5, pp. 408 - 419.
- Sykora, D. W., and Stokoe, K. H., II. 1983. "Correlations of In Situ Measurements in Sands With Shear Wave Velocity," *Geotechnical Engineering Report GR83-33*, The University of Texas at Austin, Austin, Tex.
- Sykora, D.W., 1987, "Examination of Existing Shear Wave Velocity and Shear Modulus Correlations in Soils." Department of the Army, Waterways Experiment Station, Corps of Engineers, Miscellaneous Paper GL-87-22.
- Terzaghi, K., Peck, R.B., and Mesri, G., 1996, "Soil Mechanics in Engineering Practice," John Wiley and Sons, Inc.
- Thiel, R.S., and Stewart, M. G., 1993, "Geosynthetic Landfill Cover Design Methodology and Construction Experience in the Pacific Northwest," *Geosynthetics '93 Conference Proceedings*, IFAI, St. Paul, Minnesota, pp. 1131 - 1144.
- Thiel, R.S., 1998, "Design Methodology for a Gas Pressure Relief Layer Below a Landfill Geomembrane Cover to Improve Slope Stability", *Geosynthetics International*, Vol. 5, No. 6, pp. 589-617.
- Thiel, R., 2001, "Peak vs Residual Shear Strength for Landfill Bottom Liner Stability Analyses," *Proceedings of the 15th GRI Conference on Hot Topics in Geosynthetics - II (Peak/Residual; RECMs; Installation Concerns, December 13 - 14.*
- Thiel, R., and Narejo, D., 2005 "Update on designing with geocomposite drainage layers in landfills Part 2 of 4: geocomposites on bioreactor landfill side slopes to control seeps and gas", *GRF magazine*.
- Triplett, E. J. and Fox, P. J. 2001. "Shear strength of HDPE geomembrane/geosynthetic clay liner interfaces," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 6, pp. 543-552.

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

United States Geological Survey, 2008, National Seismic Hazard Mapping Project, “Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years (site: NEHRP B-C boundary).”

Zekkos, D., Bray, J.D., and Riemer, M.F., 2008a, “Shear Modulus and Material Damping of Municipal Solid Waste Based on Large-Scale Cyclic Triaxial Testing.” *Can. Geotech. J.*, 45: 45-58

Zekkos, D., Matasovic, N., El-Sherbiny, R., Athanasopoulos-Zekkos, A., Towhata, I., and Maugeri, M., 2008b, Chapter 4, “Dynamic Properties of Municipal Solid Waste” *Geotechnical Special Publication 209, International Symposium on Waste Mechanics 2008*, ASCE.